सत्यमेव जयते

भारत सरकार
जल शक्ति मंत्रालय
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग
केन्द्रीय जल आयोग
राष्ट्रीय जल अकादमी, पुणे

Serving the Nation since 1945

केन्द्रीय जल अभियांत्रिकी सेवा के नव नियुक्त अधिकारियों का इकत्तीसवां प्रवेशन प्रशिक्षण कार्यक्रम

19 August 2019 – 07 February 2020

डिजाइन और अनुसंधान

Module III: Hydel Civil Designs

पुणे
दिसंबर 2019

Government of India
Central Water Commission
National Water Academy

केन्द्रीय जल अभियांत्रिकी सेवा के नव नियुक्त अधिकारियों का इकत्तीसवां प्रवेशन प्रशिक्षण कार्यक्रम

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Module- III

HYDEL CIVIL DESIGNS

02-08 December 2019

Module Co-ordinator

Shri S N Pande, Director (Designs)

Pune

December 2019

Module – III

Hydel Civil Designs

INDEX

INTRODUCTION TO HYDROPOWER

Rakesh Kumar,
Director (HPP&I)
CEA

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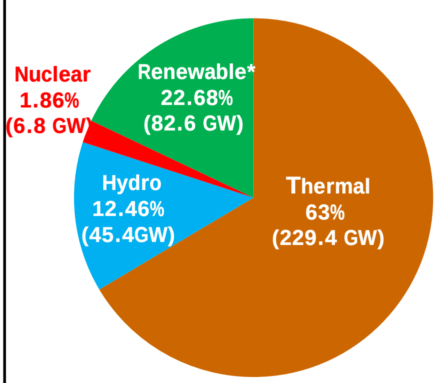
POWER SCENARIO IN THE COUNTRY
Sector–wise Installed Capacity & Energy Generation

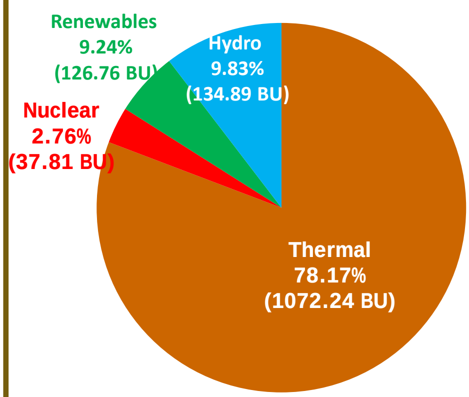
INSTALLED CAPACITY (MW)
(As on 31.10.2019)



Total : 364.2 GW

*(Renewable IC is as on 30.09.2019)

ENERGY GENERATION (BU)
2018-19



Total : 1371.70 BU*

*(Excluding 4.41 BU imports from Bhutan)

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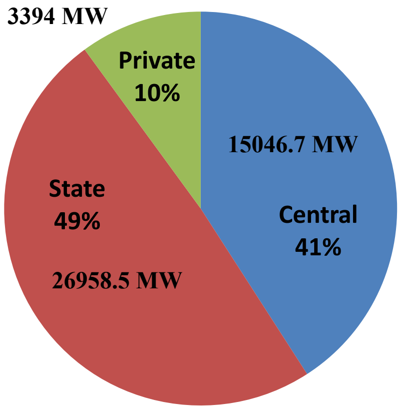
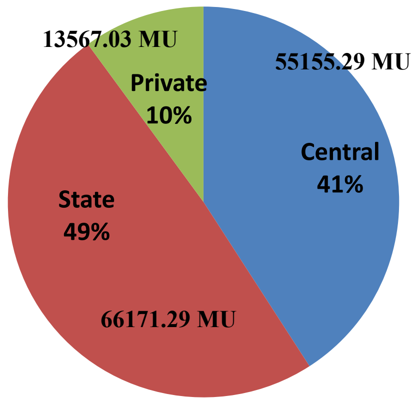
SHARE OF CENTRAL, STATE AND PRIVATE SECTOR

INSTALLED CAPACITY (MW)
45399 MW



GENERATION (BU) for 2018-19*
134.89 BU



Installed capacity as on 31st Oct. 2019

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HYDRO POWER DEVELOPMENT IN INDIA
Some Facts

First Hydro Power Station	1897, Sidrapong (130 kW) in Darjeeling
First Major Hydro Station	1902, Sivasamudram (4500 kW) in Mysore
First Private Hydro Station	1915, Tata-Khopoli (32 MW) in Maharashtra
Largest Tunnel Diameter	15m, Srisailam (770 MW) in Andhra Pradesh
Longest Tunnel	27 km, Nathpa Jhakri (1500 MW) in H.P.
Highest Dam	260m, Tehri (1000 MW), Uttarakhand
Highest Head	1026m, Pykara Ultimate (150 MW), T.Nadu
Hydro Capacity in 1947	508 MW
Present Hydro Capacity	45399 MW (As on October, 2019)

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ADVANTAGES OF HYDROPOWER

- **Renewable, Clean and Green Source of Energy**
- **Ancillary support to Grid**
- **Fastest Ramping Source : Full Load in less than 60 sec.**
- **Social Sustainability**
- **Environmental Sustainability**
- **Economic Sustainability**

Source of Power	GHG Emissions/kWh (Gram Equivalent CO_2)
Thermal – Coal	**957**
Thermal – Gas	422
RES – Solar	38
Hydro – Storage	**10**
RES – Wind	9
Nuclear	6
Hydro - RoR	**4**

Source: Hydro Quebec , Canada

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REGULATORY PROVISIONS

Section 8. (Hydro-electric generation) of Electricity Act, 2003 provides that :

- Notwithstanding anything contained in section 7, any generating company intending to set-up a hydro-generating station shall prepare and submit to the Authority for its concurrence, a scheme estimated to involve a capital expenditure exceeding such sum, as may be fixed by the Central Government, from time to time, by notification.

- The Authority (CEA) shall, before concurring in any scheme submitted to it under sub-section (1) have particular regard to, whether or not in its opinion,
  a. the proposed river-works will prejudice the prospects for the best ultimate development of the river or its tributaries for power generation, consistent with the requirements of drinking water, irrigation, navigation, flood-control, or other public purposes, and for this purpose the Authority shall satisfy itself, after consultation with the State Government, the Central Government, or such other agencies as it may deem appropriate, that an adequate study has been made of the optimum location of dams and other river-works;
  b. the proposed scheme meets the norms regarding dam design and safety.
- Where a multi-purpose scheme for the development of any river in any region is in operation, the State Government and the generating company shall co-ordinate their activities with the activities of the person responsible for such scheme in so far as they are inter-related.

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REGULATORY PROVISIONS

Central Electricity Regulatory Commission (CERC)

It regulates the Tariff and Operation & Maintenance (O&M) of Hydro Power projects of Central Sector and Inter-State Stations.

State Electricity Regulatory Commissions (SERCs)

They regulate the Tariff and Operation & Maintenance (O&M) of Hydro Power projects of State Sector.

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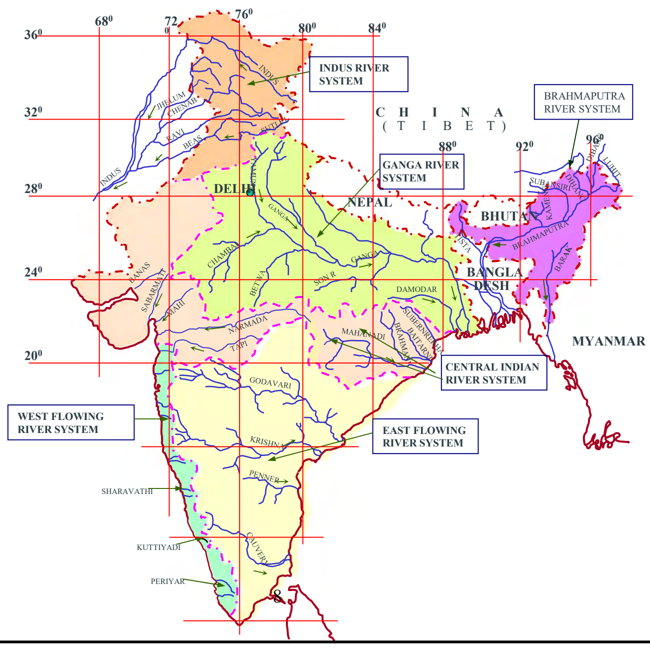
RIVER BASINS IN INDIA



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PLAN-WISE HYDRO CAPACITY ADDITION

Sector	Five Year Plan		
	10th Plan	11th Plan	12th Plan
Central	4495	1550	2584
State	2691	2702	2276
Private	700	1292	619
Total	7886	5544	5479

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GROWTH OF INDIAN HYDRO POWER SECTOR

S. No.	Year	Central Sector (MW)	State Sector (MW)	Private Sector (MW)	Total (MW)
1	2014-15	736	-	-	736
2	2015-16	480	610	426	1516
3	2016-17	80	1555	24	1659
4	2017-18	390	200	205	795
5	2018-19	110	30	-	140
Grand Total		1796	2395	655	4846

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ISSUES INVOLVED IN SLOW PACE OF HYDRO DEVELOPMENT

- Environment and Forest issues
- Land Acquisition & R&R Issues
- Inadequate Infrastructural facilities
- Law & Order / Local issues
- Geological Surprises
- Natural Calamities
- Inter-State Issues
- High Tariff of Hydro Projects
- Levying of Water Cess
- Other Emerging Issues

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HYDRO -ELECTRIC PROJECT

- A project which harnesses power from water flowing under pressure through a prime mover called Water Turbine.
- May be conceived exclusively for power generation; OR
- Power could be one of the benefits along with flood control, irrigation, drinking water supply etc. (multi-purpose project).

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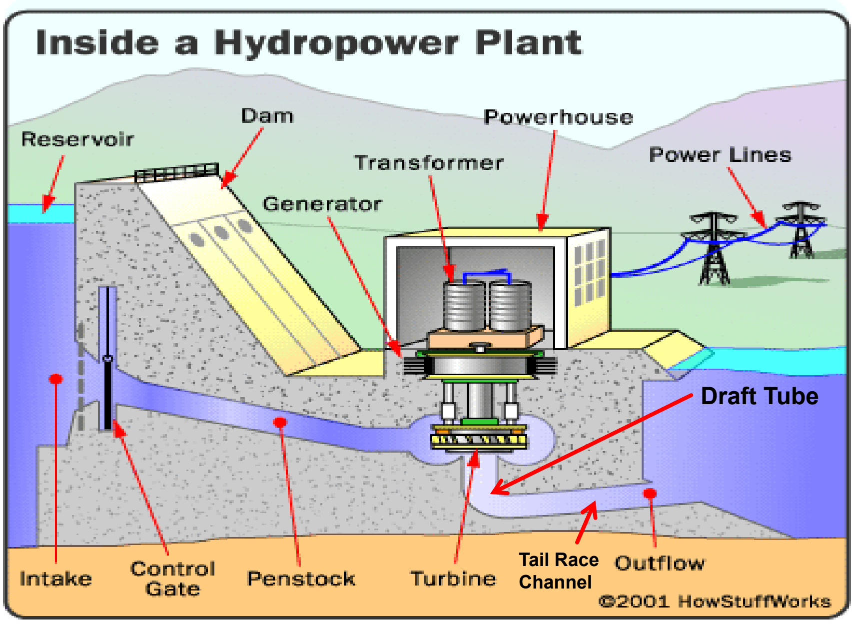
TYPICAL HYDRO STATION



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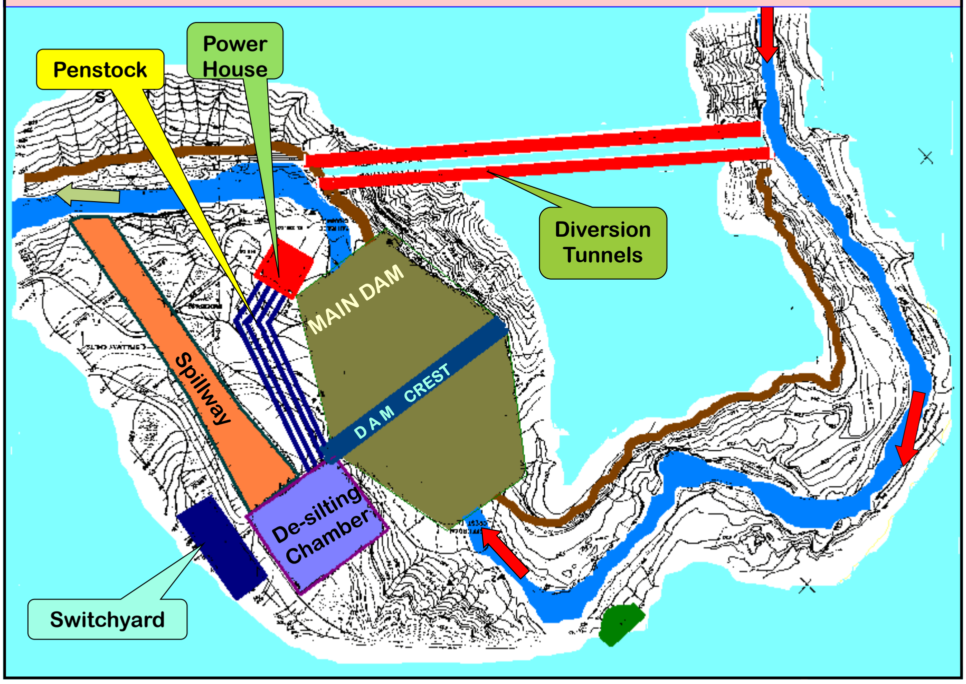
HYDRO POWER PROJECT LAYOUT



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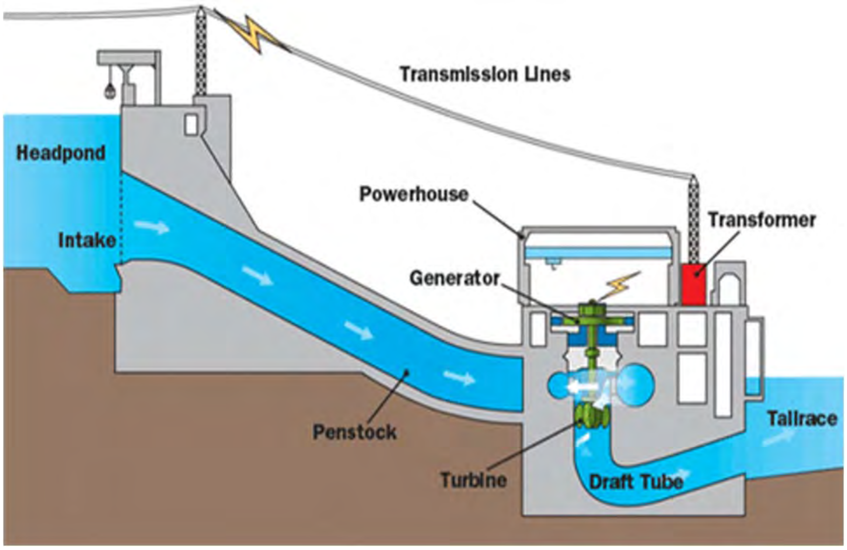
TYPICAL HYDRO STATION



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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Hydroelectric Project (HEP) : The project which harnesses power from water flowing under pressure through the prime mover known as water turbine which rotates generator.

Run-of-River Schemes (ROR) : Run-of-River schemes are the schemes either having pondage sufficient to meet diurnal variation of power demand or no upstream pondage.

Storage based HE Schemes : Schemes with reservoir to store excess water in high flow period for utilisation during low flow periods.

Pumped Storage HE Schemes : Schemes with two reservoirs, upper & lower wherein water flows from upper reservoir to lower reservoir during generation and vice versa during pumping

Multi-purpose Reservoir: Multi-purpose purpose means a reservoir capable of and intended for use for more than one purpose like Irrigation, Power Generation, Navigation and Flood Control etc.

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Full Reservoir Level (FRL) : The normal highest reservoir level that is utilised for power generation.

Minimum Drawdown Level (MDDL) : The level below which the reservoir will not be drawn down in power projects.

Maximum Water Level (MWL) : It is the full reservoir level including Flood moderation component and Freeboard level.

Gross Storage : It is the total storage in the reservoir corresponding to FRL.

Live Storage : Live Storage means all storage above Dead Storage between FRL and MDDL.

Dead Storage : Dead Storage means that portion of the storage which is not used for power generation purposes i.e. the storage below MDDL.

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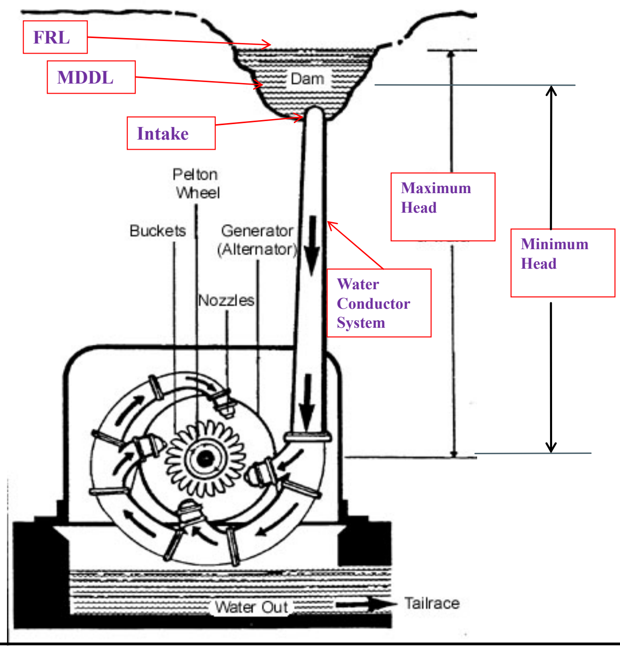
DIAGRAM OF HYDRO ELECTRIC SCHEME



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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

E-flow : Environmental flows are water flows required to sustain freshwater and estuarine ecosystems and the human livelihoods and well being that depend on these ecosystems. For example: Minimum e-flow notification of MoWR for upper reaches of River Ganga are 20%, 25% & 30% of Monthly Average Flow observed during each of preceding 10-daily period in Dry, Lean & High Flow Season respectively.
Also, MoEF&CC notifies e-flows based on project to project basis.

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

Gross Head : The difference of elevations between water surfaces of the forebay/ dam and tailrace under specified conditions.

Net Head : The head chargeable to the turbine less all hydraulic losses in water conductor system.

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

90% Dependable Year: The 90 percent dependable year is termed as the year in which the annual generation has the probability of being equal to or exceeds 90 per cent of the time on annual basis during the expected period of operation of the scheme. For determination of 90% dependable year, the total energy generation in all the years for which hydrological data is available is arranged in descending order and the $(N+1) \times 0.9^{th}$ year would represent the 90 percent dependable year where N is the number of days.

Firm Power : Firm Power in case of storage based hydro projects means the power which can be generated continuously in 90% of the years for which discharge data is available. In case of ROR projects, it is the power corresponding to the minimum mean discharge at the site of the plant in 90% dependable year.

Water Conductor System (WCS): It is the system through which water is carried from the dam to power house. It may consist of tunnels, canals, forebays, pressure shafts/ penstocks, surge tank and inlet valves etc.

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Intake: Intake structure is a structure which collects the water from the reservoir/ forebay and directs it into the HRT/ penstocks. There are different types of intake structures available and selection of type of intake structure depends on various local conditions.

Head Race Tunnel (HRT): A channel/tunnel, which carries the water for all units from Dam/ Desilting Chamber to the powerhouse for power generation.

Penstock: A closed conduit or pipe for conducting water of each unit to the powerhouse.

Butterfly Valve: A valve consisting of a rotating circular plate or a pair of hinged semicircular plates, attached to a transverse spindle and mounted inside a pipe in order to regulate or prevent flow in water conductor system.

Main Inlet Valve: Main inlet valve works as the gate valve/isolating valve in the water conductor system. It is located before turbine and allows water flow from penstock to turbine. MIV acts as closing valve and cuts the flow of water during an emergency trip

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Tail Race Tunnel (TRT): A channel or tunnel that carries the water from the turbine exit (draft tube) back to the river.

Minimum Mean Discharge (MMD) : In case of HE Projects in Indus Basin, the average discharge for each 10-day period (1^{st} to 10^{th}, 11^{th} to 20^{th} and 21^{st} to the end of the month) is worked out for each year for which discharge data, whether observed or estimated, are proposed to be studied for purpose of design. The mean of the yearly values for each 10-day period will then be worked out. The lowest of the mean values thus obtained is taken as the Minimum Mean Discharge.

Pondage: Pondage means Live Storage of only sufficient magnitude to meet fluctuations in the discharge of the turbines arising from variations in the daily and the weekly loads of the plant.

Power House: Power House is a building housing the turbines, generator, and control and protection equipment etc. including auxiliaries for operating the machines.

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Design Energy: Energy generated in 90% dependable year with 95% availability of installed capacity.

Secondary Energy : Secondary Energy represents the additional energy generation in any year over and above the Design Energy.

Carry Over Operations : When good flows available are stored in the reservoir for use in lean flow years.

Annual Operations : Also called Year to Year operation in which reservoir is brought down to MDDL at the end of each hydrological year.

Installed Capacity (MW/ MVA) : The total of the capacities shown on the name-plates of the generating units in a hydropower plant

Annual Load Factor: The ratio of number of units actually generated in a year to number of units which would have been generated for the given installed capacity is called as Annual Load Factor for the station.

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BASIC TERMINOLOGY OF HYDRO POWER ENGINEERING

Daily Load Factor: The ratio of number of units actually generated in a day to number of units which would have been generated for the given installed capacity during the day is called as Daily Load Factor for the station

Power Factor : The ratio of the actual electrical power dissipated by an AC circuit to the product of the r.m.s. values of current and voltage. The difference between the two is caused by reactance in the circuit and represents power that does no useful work.

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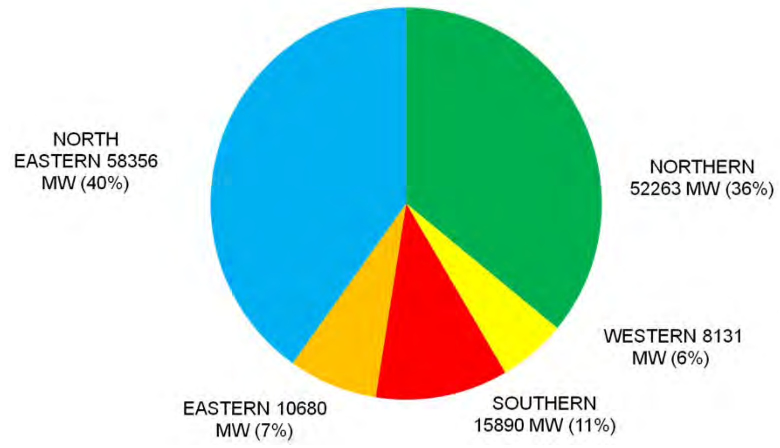
REGION-WISE DISTRIBUTION OF H.E. POTENTIAL
TOTAL: 145320 MW (ABOVE 25 MW)



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HYDRO-ELECTRIC POTENTIAL - REGION-WISE

Region	Potential Capacity (MW)		Capacity Developed		Capacity under Construction	
	Total	> 25 MW	Conv. (MW)	%	Conv. (MW)	%
Northern	53395	52263	19023	36.40	5516	10.55
Western	8928	8131	5552	68.28	400	4.92
Southern	16458	15890	9689	60.97	1060	6.67
Eastern	10949	10680	4923	46.09	1253	11.73
NE Region	58971	58356	1427	2.45	2600	4.45
Total	148701	145320	40614*	27.95	10829**	7.45

*In addition 4785 MW of Pumped Storage Capacity is also in operation
** In addition 1580 MW Pumped Storage is under construction

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PUMPED STORAGE POTENTIAL AND DEVELOPMENT

	Nos.	Capacity (MW)
Identified potential in 1980s	63	96,529.6*
Not found feasible	15	29,930
Additional Schemes identified subsequently	23	15,820
Revised PSP potential	71	82,419.6

Region	Potential (MW)	Developed (MW)	Under Constn. (MW)
Northern	8,185 (5 Nos.)	0	1000 (1 No.)
Western	32,209 (31 Nos.)	1840 (4 Nos.)	80 (1 No.)
Southern	12780.6 (13 Nos.)	2005.6 (3 Nos.)	500 (1No)
Eastern	12345 (12 Nos.)	940 (2 Nos.)	0
North Eastern	16900 (10 Nos.)	0	0
Total	82419.6 (71 Nos.)	4785.6 (9 Nos.)**	1580 (3 Nos.)

*Original identified potential was 96,524 was enhanced due to revised capacity of Nagarjun Sagar PSP (705.6 MW).
*In addition 2 Nos. of schemes namely Paithan(12 MW) and Ujjaini(12 MW) in Aurangabad and Solapur in Maharashtra are also under operation.

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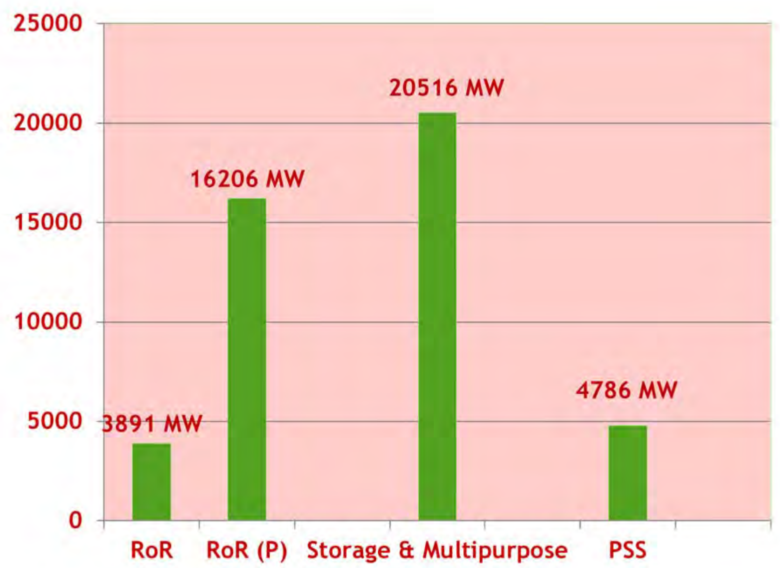
TYPE-WISE SUMMARY OF HYDRO PROJECTS IN OPERATION (45399 MW)



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CLASSIFICATION OF HYDRO POWER PROJECTS BASED ON INSTALLED CAPACITY

MAJOR SCHEMES

Schemes with installed capacity > 100 MW.

MEDIUM SCHEMES

Schemes with installed capacity - 25 MW to 100 MW.

SMALL SCHEMES

Schemes with installed capacity - 2 MW to 25 MW.

MINI SCHEMES

Schemes with installed capacity - 100 KW to 2 MW.

MICRO SCHEMES

Schemes with installed capacity upto 100 KW.

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CLASSIFICATION BASED ON STORAGE

RUN-OF-RIVER SCHEMES

Schemes with very little or no storage (Daily/ Weekly)

STORAGE SCHEMES

Schemes with reservoir to store excess water in high flow period for utilization during low flow periods.

PUMPED STORAGE SCHEMES

Schemes with two reservoirs ,upper & lower, water flows from upper reservoir to lower reservoir during generation and vice versa during pumping

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CLASSIFICATION OF WORKS IN HYDRO ELECTRIC PROJECTS

- **CIVIL WORKS**
  Dam works, Intake structure, Diversion Tunnel, Head Race Tunnel, Power House building, Tail Race Tunnel

- **HYDRO-MECHANICAL WORKS**
  Gates, Valves etc.

- **ELECTRO-MECHANICAL WORKS**
  Turbines, Generators, Control, Protection equipment and other auxiliaries

- **BALANCE OF PLANT (BOP)**
  Switchyard, POT Head Yard, SCADA, Fire Fighting System, HT (above 650 V) Cables, LT (below 650 V) Cables, Protection & Metering System (Relay & Meters), Unit Auxiliary Transformers (UATs), Station Auxiliary Transformers (SATs)

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MAJOR COMPONENTS OF HYDRO SCHEMES

- **DAM WITH CONTROL WORKS**
  Dams are constructed for storage of water and to create head.

- **STORAGE RESERVOIR**
  Water available from a catchment area is stored in reservoir for producing power according to the requirement throughout the year.

- **DESILTING CHAMBER**
  It is a chamber in which the sediment particles up to a specified grain size and above would settle (by slowing down their velocity) thereby allowing relatively silt-free water to flow into the head race tunnel.

- **WATER CONDUCTOR SYSTEM (HRT AND PENSTOCK)**
  The system through which the water is carried from the dam to the power house.

- **GATES**
  A barrier that regulates water released from a reservoir to the power generation unit.

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MAJOR COMPONENTS OF HYDRO SCHEMES

- **VALVES**

The sluicing valves control the water flowing to the downstream and automatic isolating valves stop the water flow when the electrical load is suddenly thrown off from the plant. Automatic isolating valve only operates during emergency to protect the system from burst out.

- **POWER HOUSE**

Building housing the turbines, generator, control, protection equipment and other auxiliaries .

- **TAIL RACE**

Water way to carry water back to the river

- **SWITCHYARD**

Power generated is pooled in switchyard & transmitted to load centers.

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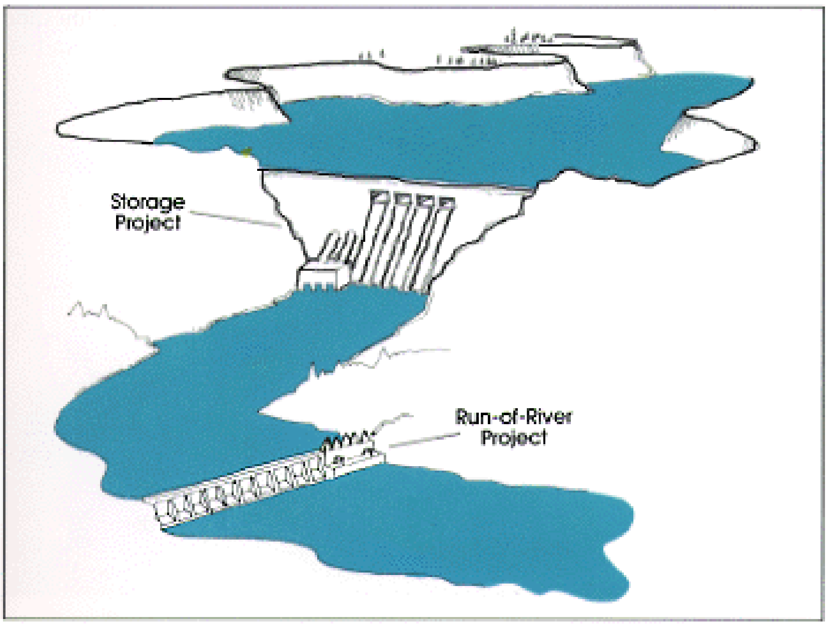
STORAGE AND RUN-OF-RIVER PROJECTS



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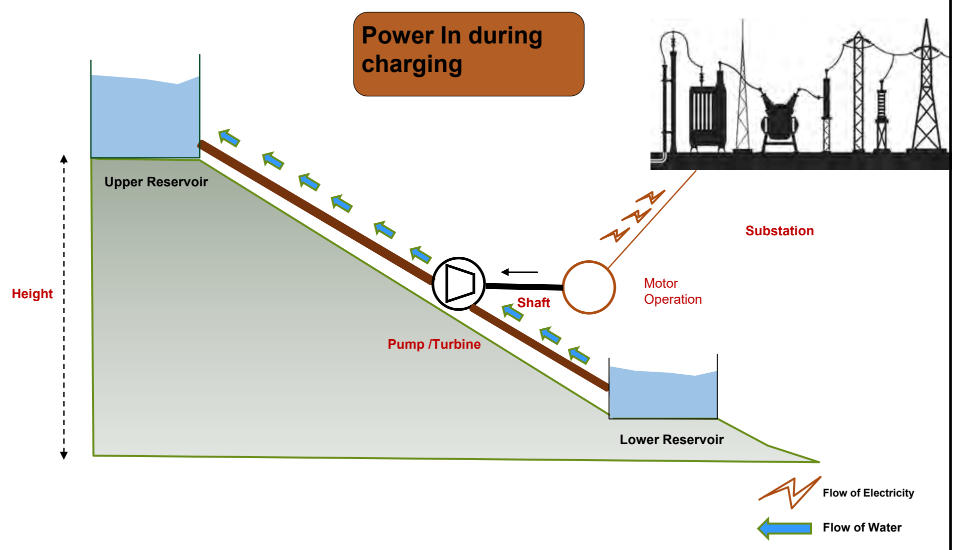
PUMPED STORAGE PROJECTS



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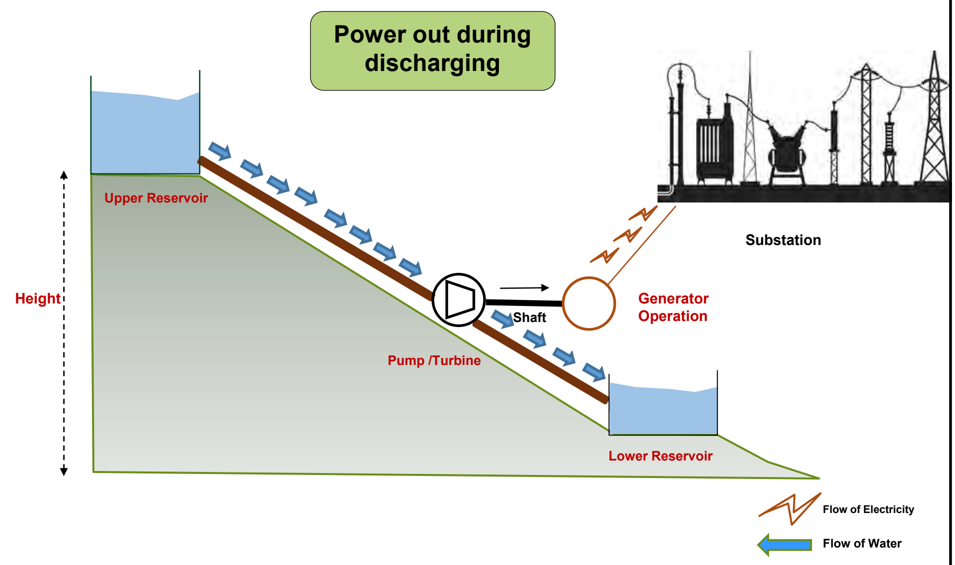
PUMPED STORAGE PROJECTS



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CLASSIFICATION OF ROR PROJECTS

Run-of-River Scheme without Pondage
A hydro electric power generating scheme that uses the flow of the stream as it occurs.

Run-of-River Scheme with Pondage
A hydro electric power generating scheme with sufficient pondage for meeting daily or weekly variation of power demand.

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CLASSIFICATION OF STORAGE PROJECTS BASED ON FUNCTIONS

Multi-purpose Project
Power may be one of the benefits along with Flood Control, Irrigation, Navigation, Drinking Water Supply etc.

Purely Hydro-electric Project
Project is conceived exclusively for power generation.

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BASICS OF POWER POTENTIAL STUDIES

Power Potential Studies are carried out for assessment of the available power potential and likely benefits from a scheme based on:

- a certain set of inflows and head conditions
- corresponding to fulfillment of certain objectives/ constraints
- ensuring optimum development of river/basin potential.

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POWER POTENTIAL STUDIES OF R-O-R PROJECTS

Power is calculated using the relation:-

P=9.81 *Q*Hn*n

where

P is Power generated in kW
Q is Discharge in Cumecs
n is overall efficiency of TG units
Hn is Net Head acting on turbine

Net Head= Gross Head – Losses in Water Conductor System
Gross head= Reservoir level-Tail Water Level (Francis)
= Reservoir level-Centre Line of Turbine (Pelton)

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SELECTION OF FRL & MDDL

FRL is selected mainly based on :

- Techno-economic considerations
- Submergence in the Reservoir Area
- Tail Water Level of Upstream Developments
- Geological Constraints in raising Dam Height

MDDL is selected mainly based on:

- MDDL in case of ROR schemes is selected mainly from consideration of Pondage Requirements for Peaking Operation of the Proposed Station.
- In case of Storage Schemes, techno-economic and other aspects like (New Zero Elevation) & turbine operational limits are also considered.

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SELECTION OF 90% DEPENDABLE YEAR

- **Flow data is arranged, hydrological year-wise**
- **Unrestricted Energy Potential is worked out for all the years (N) and is arranged in Descending Order.**
- **If flow data are Available for 'n' no. of years, then Nth year would constitute 90% Dependable Year :**

Nth Year = (n+1) * 0.9

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POWER POTENTIAL STUDIES
(R-O-R SCHEMES WITHOUT PONDAGE)

- Work Out 90% dependable year.
- For 90% dependable year, work out energy benefits corresponding to alternative of installed capacities
- Select installed capacity after carrying out "*Incremental Analysis*" corresponding to the most attractive alternative
- Select unit size by considering:
  - Pattern of generation in various periods
  - Transportation constraints, if any

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POWER POTENTIAL STUDIES
(R-O-R SCHEMES WITH PONDAGE)

- Work Out 90% dependable year.
- For 90% dependable year, work out energy benefits corresponding to alternative of installed capacities
- Select installed capacity after carrying out "Incremental Analysis" corresponding to the most attractive alternative with due consideration to lean period power output and load factor.
- Select unit size by considering Transportation constraints, if any
- Hourly operation of the pondage is carried out to work out pondage requirements for daily/ weekly peaking

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REQUIREMENT OF PONDAGE

"Pondage may be defined as the **holding back and releasing later of water** at the dam of a water power development project to:

- equalize daily or weekly **fluctuations in river flow** or
- permit irregular hourly use of water by the wheels to accord with **fluctuations in load demand**.

Fluctuations in river flow may be : (1) natural, due to rainfall or snow melting or (2) artificial, due to pondage of water at other plants upstream".

Source : Water Power Engineering by H.K. BARROWS (Third edition 1943-reprinted in 1980) :- Tata McGraw-Hill (Pg. 180)

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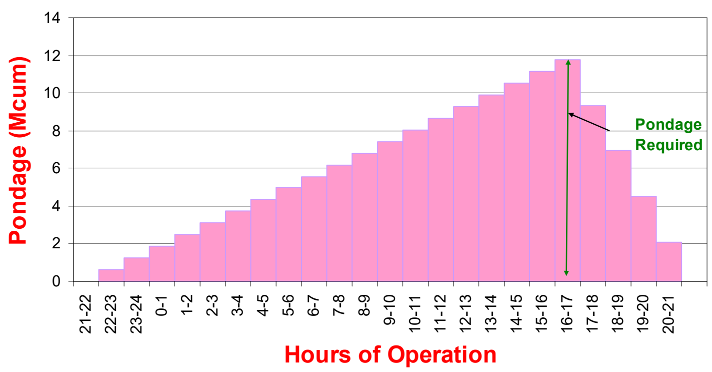
ADVANTAGES OF PONDAGE & OPERATING PATTERN OF DAILY PONDAGE

- **Flexibility in operation of the Plant**
- **Higher Capacity**
- **Greater Energy Benefits**



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CALCULATIONS & FACTORS AFFECTING PONDAGE

CALCULATIONS

- Work out minimum flows in 90% dependable year.
- Estimate design discharge corresponding to installed capacity.
- Estimate required Pondage capacity based on hourly operation of the Plant based on the desired load demand pattern.

FACTORS AFFECTING

- Number of blocks of peak operation (Higher the number of blocks, lesser is the Pondage required)
- Quantum of natural inflows
- Designed Outflows from the power house.

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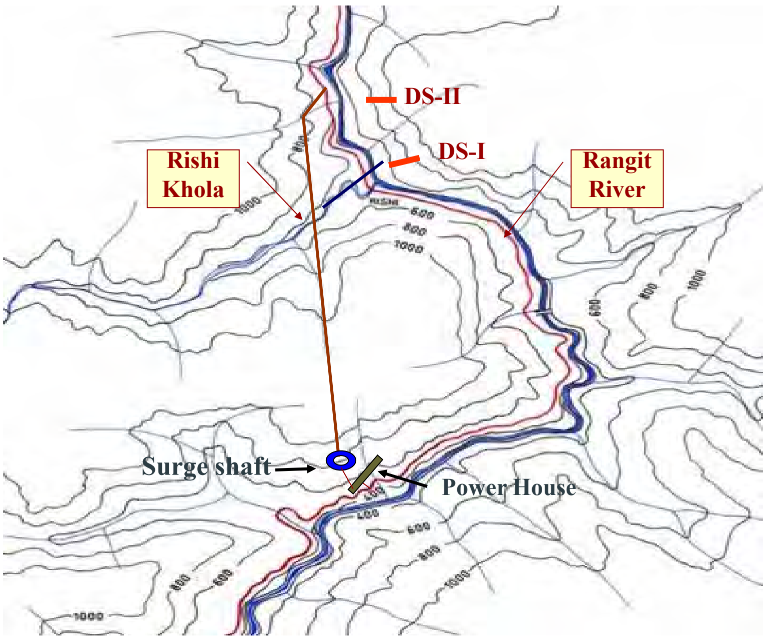
CASE STUDY FOR R-O-R PROJECT



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SELECTION OF DAM SITE & THE FRL

For location of dam, the following two options were studied:

- **Option - I** - Near Rishi restricting the FRL to El. 468 m for safeguarding the religious structures at Tatopani and Hot Spring.
- **Option - II** - about 250m upstream of Tatopani with FRL of 493 m to avoid submergence of religious structures at Tatopani and Hot Spring. This alternative would have exploited full potential available downstream of Rangit Stage III project.

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RANGIT-IV - ISSUE OF TATOPANI

Tatopanai Dam

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GUIDELINES FOR HYDROLOGY CHAPTER FOR PREPARATION OF DPR (As per CWC Guidelines for preparation of DPR for Irrigation and Multipurpose Projects,2010)

A- Classification by storage behind the structures

A-I : Diversion projects without pondage

A-2 : Diversion projects with pondage

A-3 : Within the year storage projects

A-4 : 'Over the year' storage projects

A-5 : Complex system involving combinations of 1 to 4 above mentioned.

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GUIDELINES FOR HYDROLOGY CHAPTER FOR PREPARATION OF DPR

Type of project	Description of 'A'	Minimum length of data for use
Diversion projects without pondage	A1	10 years
Diversion projects with pondage	A2	10 years
Within the year storage projects	A3	25 years
'Over the year' storage projects	A4	40 years
Complex system involving combinations of 1 to 4 above mentioned.	A5	depending upon the predominant element (A 1 to A4)

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DATA/ PARAMETERS OF STUDY

- Hydrological Year - June to May
- Inflow Data - 26 Years
- FRL - 468 m
- MDDL - 458 m
- Avg. Gross Head - 118.67 m
- Head losses in WCS - 15 m
- Net Head - 103.67 m
- Overall TG Efficiency - 92%
- Avg. Tail Water Level - 346 M

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SELECTION OF 90% DEPENDABLE YEAR

Year	Unrestricted Energy Potential (MU)
1-2	743.58
2-3	634.90
3-4	747.53
4-5	676.14
5-6	865.54
6-7	750.79
7-8	868.48
8-9	765.86
9-10	766.51
10-11	814.11
11-12	649.20
12-13	618.42
13-14	687.43

Year	Unrestricted Energy Potential MU)
14-15	782.52
15-16	867.34
16-17	934.22
17-18	1063.34
18-19	989.59
19-20	856.40
20-21	1098.16
21-22	1319.62
22-23	1286.65
23-24	1217.90
24-25	1761.46
25-26	905.00
26-27	703.24

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SELECTION OF 90% DEPENDABLE YEAR

S. No.	Year	Annual Unrestricted Energy Potential (MU)	S. No.	Year	Annual Unrestricted Energy Potential (MU)
1	24-25	1761.46	14	10-11	814.11
2	21-22	1319.62	15	14-15	782.52
3	22-23	1286.65	16	9-10	766.51
4	23-24	1217.90	17	8-9	765.86
5	20-21	1098.16	18	6-7	750.79
6	17-18	1063.34	19	3-4	747.53
7	18-19	989.59	20	1-2	743.58
8	16-17	934.22	21	26-27	703.24
9	25-26	905.00	22	13-14	687.43
10	7-8	868.48	23	4-5	676.14
11	15-16	867.34	24	11-12	649.20
12	5-6	865.54	25	2-3	634.90
13	19-20	856.40	26	12-13	618.42

90% Year = (n + 1) * 0.9th Year

= 24.30 (24th Year)

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SUMMARY OF POWER POTENTIAL STUDIES FOR 90% DEPENDABLE YEAR

	INSTALLED CAPACITY (MW)								
	70	80	90	100	110	120	130	140	150
Annual Energy MU)	376.9	412.7	446.8	479.6	511.0	540.8	567.4	590.9	608.1
Potential Exploited (%)	58.1	63.6	68.8	73.9	78.7	83.3	87.4	91.0	93.7
Incremental Energy (MU)	-	35.8	34.1	32.8	31.4	29.8	26.6	23.5	17.2
Incremental Energy (kWh/kW)	-	3577	3408	3280	3148	2984	2623	2356	1715

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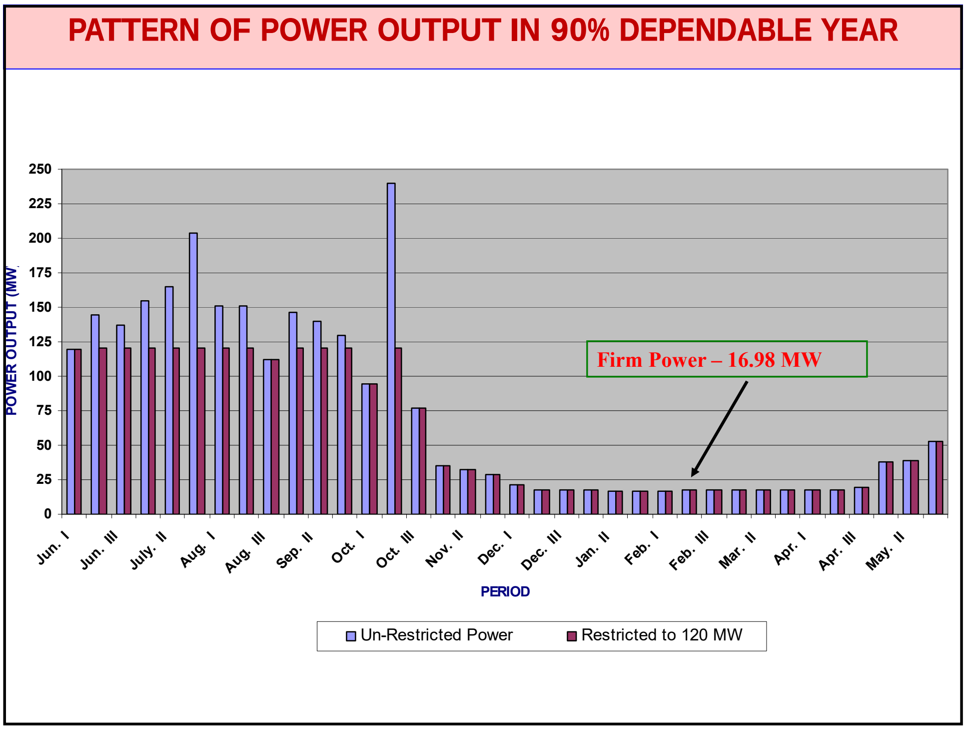
PATTERN OF POWER OUTPUT IN 90% DEPENDABLE YEAR
POWER OUTPUT (MW
250
225
200
175
150
125
100
75
50
25
0
Jun. I
Jun. III
July. II
Aug. I
Aug. III
Sep. II
Oct. I
Oct. III
Nov. II
Dec. I
Dec. III
Jan. II
Feb. I
Feb. III
Mar. II
Apr. I
Apr. III
May. II
PERIOD
Firm Power – 16.98 MW
Un-Restricted Power
Restricted to 120 MW

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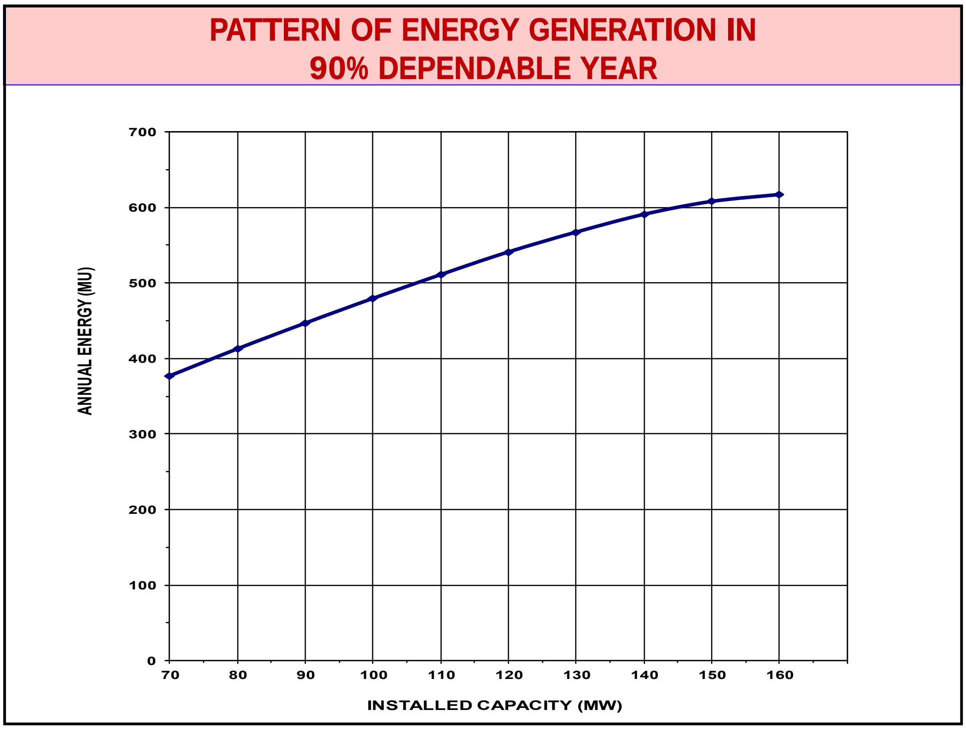
PATTERN OF ENERGY GENERATION IN
90% DEPENDABLE YEAR
ANNUAL ENERGY (MU)
700
600
500
400
300
200
100
0
70
80
90
100
110
120
130
140
150
160
INSTALLED CAPACITY (MW)

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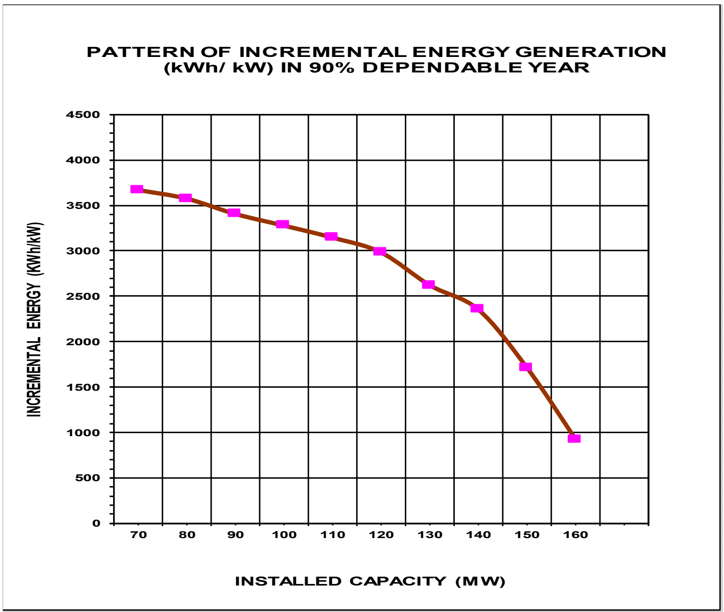
PATTERN OF INCREMENTAL ENERGY GENERATION (kWh/ kW) IN 90% DEPENDABLE YEAR



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POWER POTENTIAL STUDIES - CONCLUSIONS

- It is seen that incremental energy generation shows a decreasing pattern for installed capacities beyond 120 MW.
- It is, further, seen that incremental Energy (KWh/KW) tends to fall sharply for installed capacities beyond 120 MW.

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SELECTION OF INSTALLED CAPACITY

Selected as 120 MW based on :

- Incremental Analysis
- Load Factor (about 14.15%-Minimal and 51.44% Annual) / Firm Power Considerations (about 16.98 MW)
- Potential Exploited (More than 83%)

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DESIGN ENERGY GENERATION

Month	Inflows	Unrestricted Potential		Energy Generation with IC=120 MW	Design Energy with 95% m/c Availability	Peaking Hrs.
	(Cumecs)	MW	MU	MU	MU	
Jun. I	128.14	119.89	28.77	28.77	27.33	23.43
Jun. II	154.10	144.18	34.60	28.80	27.36	24.00
Jun. III	146.66	137.22	32.93	28.80	27.36	24.00
July. I	165.56	154.90	37.18	28.80	27.36	24.00
July. II	175.77	164.45	39.47	28.80	27.36	24.00
July. III	217.73	203.71	53.78	31.68	30.10	24.00
Aug. I	161.78	151.37	36.33	28.80	27.36	24.00
Aug. II	161.78	151.37	36.33	28.80	27.36	24.00
Aug. III	119.95	112.23	29.63	29.63	29.63	22.45
Sep. I	156.37	146.30	35.11	28.80	27.36	24.00
Sep. II	149.56	139.93	33.58	28.80	27.36	24.00
Sep. III	138.47	129.56	31.09	28.80	27.36	24.00
Oct. I	101.18	94.66	22.72	22.72	22.72	18.93
Oct. II	256.66	240.14	57.63	28.80	27.36	24.00
Oct. III	81.65	76.39	20.17	20.17	20.17	15.28
Nov. I	37.55	35.13	8.43	8.43	8.43	7.03
Nov. II	34.40	32.18	7.72	7.72	7.72	6.44
Nov. III	30.24	28.29	6.79	6.79	6.79	5.66

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DESIGN ENERGY GENERATION

Month	Inflows	Unrestricted Potential		Energy Generation with IC-120 MW	Design Energy with 95% m/c Availability	Peaking Hrs.
	(Cumecs)	MW	MU	MU	MU	
Dec. I	23.18	21.69	5.21	5.21	5.21	4.34
Dec. II	18.52	17.33	4.16	4.16	4.16	3.47
Dec. III	18.65	17.45	4.61	4.61	4.61	3.49
Jan. I	18.65	17.45	4.19	4.19	4.19	3.49
Jan. II	18.27	17.09	4.10	4.10	4.10	3.42
Jan. III	18.27	17.09	4.51	4.51	4.51	3.42
Feb. I	18.14	16.98	4.07	4.07	4.07	3.40
Feb. II	19.15	17.92	4.30	4.30	4.30	3.58
Feb. III	18.77	17.57	3.37	3.37	3.37	3.51
Mar. I	18.77	17.57	4.22	4.22	4.22	3.51
Mar. II	18.77	17.57	4.22	4.22	4.22	3.51
Mar. III	18.77	17.57	4.64	4.64	4.64	3.51
Apr. I	18.65	17.45	4.19	4.19	4.19	3.49
Apr. II	18.52	17.33	4.16	4.16	4.16	3.47
Apr. III	21.17	19.81	4.75	4.75	4.75	3.96
May. I	40.19	37.61	9.03	9.03	9.03	7.52
May. II	41.83	39.14	9.39	9.39	9.39	7.83
May. III	55.94	52.34	13.82	13.82	13.82	10.47
Annual Energy (MU)			**649.20**	**540.84**	**523.44**	

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DIURNAL PONDAGE REQUIREMENTS

Design Net Head	**103.67m**
Installed Capacity	**120 MW**
Peaking Discharge	**128.25 Cumec**
Firm Power	**16.98 MW**
LF (Minimal-Daily)	**14.15%**
Peaking Hrs.	**3.40 Hrs.**
Minimum Discharge Q	**18.15 Cumec**
Pondage Requirements	**1.35 MCM**

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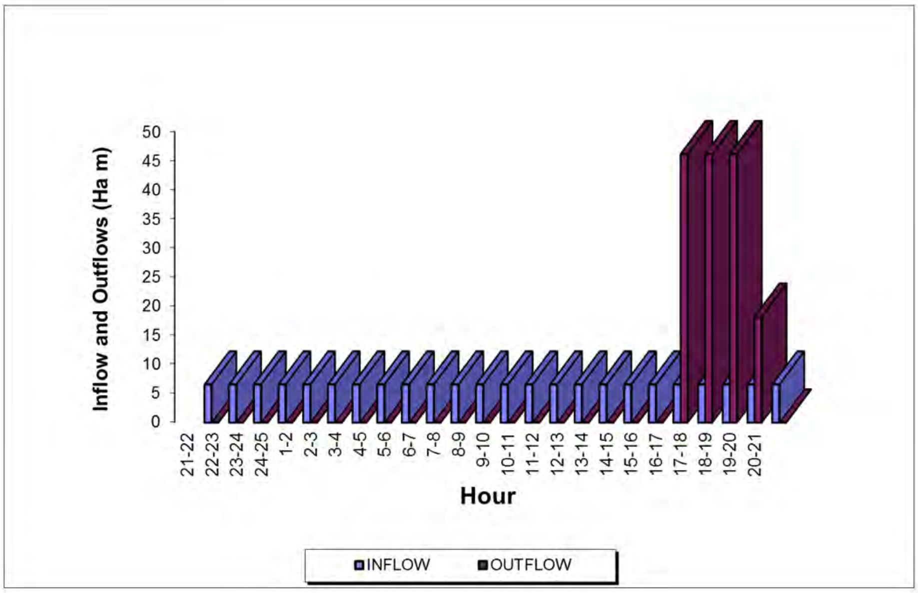
PATTERN OF HOURLY OPERATION OF PONDAGE



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COMPARISON OF RANGIT STAGE-IV WITH AND WITHOUT PONDAGE

	Rangit with Pondage	Rangit without Pondage
Firm Power	18 MW	18 MW
I.C.	120 MW	60-70 MW
Unit Size	40 MW	20-25 MW
Cost	Civil Cost almost similar in both	
Peaking	120 MW	Not possible
Annual Energy	540 MU	340-360 MU

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SELECTION OF UNIT SIZE

Main Consideration :**Transport Constraints**
Unit Size Considered: **30 MW, 40 MW & 60 MW**
Unit Size selected : **40 MW**

Advantages:

- **Ease of Transport**
- **Flexibility of Operation**
- **Greater availability in case of planned & Forced Outages**

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RESERVOIR SIMULATION STUDIES

OBJECTIVES

These studies involve operation of a reservoir or a group of reservoirs with past historical inflow data for:

New Schemes

- **Selection of optimum level of dam height and Reservoir Parameters like FRL and MDDL**
- **Level of firm power and energy generation etc. considering different operational objectives & constraints**

Existing Schemes

- **Evolve Suitable Operational Policy**

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RESERVOIR SIMULATION STUDIES

Reservoir simulation studies are categorized as under:

Carry Over Operations:

When good flows available are stored in the reservoir for use in lean flow years.

Annual Operations:

Also called Year to Year operation in which reservoir is brought down to MDDL at the end of each hydrological year.

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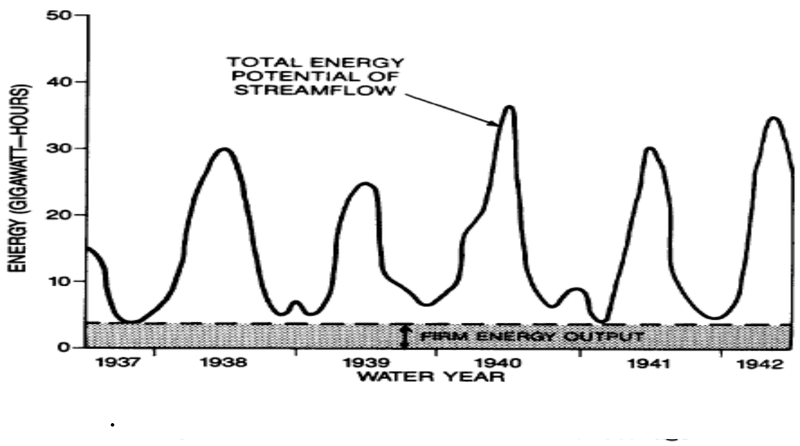
ENERGY POTENTIAL AND FIRM POWER WITHOUT STORAGE



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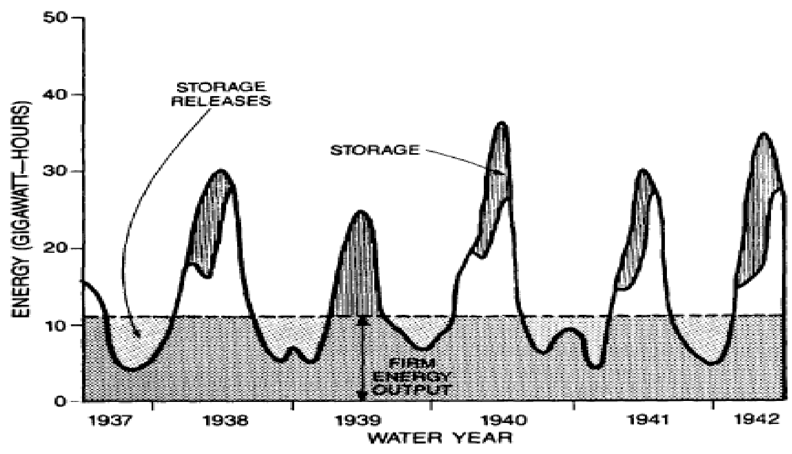
ENERGY POTENTIAL AND FIRM POWER WITH STORAGE



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RESERVOIR SIMULATION STUDIES

INPUTS

- **Hydrological Data (Inflow/Evaporation data)**
- **Physical Characteristics (Reservoir Area-capacity characteristics, Tailwater Rating Curve, losses in WCS, Electrical and Hydraulic efficiencies etc.)**
- **Operational Constraints (minimum discharge to meet irrigation, operation rule curve etc.)**
- **Operational Objectives (no. of permissible failures)**

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RESERVOIR SIMULATION STUDIES
INPUT DATA

- Hydrological Flow Data i.e. Monthly/ Ten Daily arranged in Hydrological Year-wise (June to May etc.)
- Evaporation Loss Data i.e. Month-wise in mm

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RESERVOIR SIMULATION STUDIES
INPUTS DATA

PHYSICAL CHARACTERISTICS

- Reservoir Area-Elevation and Capacity-Elevation Characteristics
- Specify Full Reservoir Level & Minimum Drawdown level
- Tail Water Level (Fixed Value or Rating Curve)
- Losses in Water Conductor System
- Electrical and Hydraulic efficiencies etc.

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RESERVOIR SIMULATION STUDIES
INPUT DATA

OPERATIONAL CONSTRAINTS

- **Minimum Discharges to meet environmental or other non-power water requirements like Irrigation and Drinking Water Supply etc.**
- **Operation Rule Curve for Moderation of Flood Peaks.**
- **Releases for Navigation, if any**

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RESERVOIR SIMULATION STUDIES
INPUT DATA

OPERATIONAL OBJECTIVES

- **Number of Permissible Failures in simulation considering desired level of Dependability.**

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Dependability Concept

- In any storage scheme, storage capacity is a function of both targeted draft and the reliability.
- For fixed draft, storage would increase with increase in level of reliability which means higher dam height and thus greater cost of project.
- For fixed storage, greater reliability means lower draft and consequently lesser firm power output.

Dependability Criteria

Power	-	90%
Irrigation	-	75%
Drinking Water Supply & Navigation	-	100%

RESERVOIR SIMULATION STUDIES

OUTPUT

In the form of different variables like

- Reservoir Levels
- Power house & Spillway discharges
- Evaporation
- Head
- Energy/power generation.

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STANDARD RESERVOIR OPERATION POLICY

- Select Tentative Firm Power corresponding to dependable flows and average head conditions.
- Considering the committed releases for Irrigation, Navigation & Drinking Water Supply & rule curve for operation of reservoir, simulate the reservoir, hydrological year-wise.
- In case no. of failures are less, increase firm power and simulate the reservoir again.
- In case no. of failures are more, decrease firm power and simulate the reservoir again.
- Continue the process till objectives are accomplished.

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RESERVOIR SIMULATION POLICY
(CARRY OVER BASIS)

- Start Simulation either (i) from beginning of monsoon with Reservoir at MDDL or (ii) from end of monsoon with Reservoir at FRL (preferably (i)).
- Releases are initially made for firm power or other objectives and the extra water is stored in the reservoir for use in lean years .
- Reservoir is gradually built up to FRL.
- When reservoir is at FRL, any extra water available is used for generating secondary generation up to the installed capacity.
- Any extra water still available is spilled.

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RESERVOIR SIMULATION POLICY
(ANNUAL OPERATION)

- Start Simulation either (i) from beginning of monsoon with Reservoir at MDDL or (ii) from end of monsoon with Reservoir at FRL (preferably (i)).
- Releases are made for firm power or other objectives & extra water is stored in the reservoir.
- Reservoir is gradually build up to FRL.
- When reservoir is at FRL, extra water available is used for generating secondary generation up to the installed capacity.
- Any extra water still available is spilled.
- During lean flow periods, reservoir is gradually drawn down so as to be at MDDL at the onset of next monsoon.

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SELECTION OF FRL

FRL is selected mainly based on techno-economic & and other considerations like

- Submergence in the Reservoir Area
- Tail Water Level of Upstream Developments
- Geological Constraints in raising Dam Height
- Availability of adequate flows

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SELECTION OF MDDL

In case of Storage Schemes, techno-economic and other aspects like silt (New Zero Elevation) & turbine operational limits are the main consideration.

(Sometimes, aspects like diameter of Intake, water cover required to avoid vortex formation etc. are also considered.)

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SELECTION OF MDDL IN STORAGE SCHEMES

TABLE 5-1
Discharge and Head Ranges for Different Types of Turbines

Turbine Type	Ratio of Minimum Discharge to Rated Discharge	Ratio of Minimum Head to Maximum Head
Francis	0.40	0.50
Vertical shaft Kaplan	0.40	0.40
Horizontal shaft Kaplan	0.35	0.33
Fixed blade propeller	0.65	0.40
Fixed gate adjustable blade propeller	0.50	0.40
Fixed geometry units (pumps as turbines)	-	0.80
Pelton (adjustable nozzles)	0.20	0.80

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SELECTION OF 90% Dependable Year

- Annual Energy is worked out for all the years (N) using simulation model considering operational constraints and desired operational objectives corresponding to set of input data.
- Annual Energy Generation is arranged in Descending Order.
- If the data are Available for 'n' no. of years, then Nth year as below would constitute 90% Dependable Year :

  Nth Year = (n+1) * 0.9 and

  Energy Benefits in the Nth year (arranged in descending order) would constitute energy benefits in 90% dependable year.

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To find 90% Dependable Energy

S. No.	Year	Annual Energy (MU)
1	1978/79	20987.5
2	1979/80	19319.4
3	1980/81	20547.6
4	1981/82	21573.0
5	1982/83	18611.3
6	1983/84	18971.7
7	1984/85	19965.2
8	1985/86	20261.8
9	1986/87	18570.8
10	1987/88	18156.2
11	1988/89	18862.4
12	1989/90	21279.0

S. No.	Year	Annual Energy (MU)
13	1990/91	21479.3
14	1991/92	19735.4
15	1992/93	16295.7
16	1993/94	18386.5
17	1994/95	17136.7
18	1995/96	20739.6
19	1996/97	21634.3
20	1997/98	18492.8
21	1998/99	20404.9
22	1999/00	19859.9
23	2000/01	19411.7
24	2001/02	18822.6
25	2002/03	16862.8

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TO FIND 90% DEPENDABLE ENERGY ARRANGE IN DESCENDING ORDER

S. No.	Year	Annual Energy (MU)
1	1996/97	21634.3
2	1981/82	21573.0
3	1990/91	21479.3
4	1989/90	21279.0
5	1978/79	20987.5
6	1995/96	20739.6
7	1980/81	20547.6
8	1998/99	20404.9
9	1985/86	20261.8
10	1984/85	19965.2
11	1999/00	19859.9
12	1991/92	19735.4

S. No.	Year	Annual Energy (MU)
13	2000/01	19411.7
14	1979/80	19319.4
15	1983/84	18971.7
16	1988/89	18862.4
17	2001/02	18822.6
18	1982/83	18611.3
19	1986/87	18570.8
20	1997/98	18492.8
21	1993/94	18386.5
22	1987/88	18156.2
23	1994/95	17136.7
24	2002/03	16862.8
25	1992/93	16295.7

90% Dep. Year Energy = 17136.7

90% Dep. Year = (25+1)*0.9=23.4
i.e., 23rd year

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SELECTION OF INSTALLED CAPACITY

Main Consideration :

- Techno-economic considerations like incremental cost-benefit analysis
- System Considerations like lean period load factor determined by power absorption studies

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SELECTION OF UNIT SIZE

Main Consideration :

Transport Constraints

Other Considerations:

- Techno-economic Consideration.
- System Considerations.
- Size of Cavity in case of UG Power House especially in Himalayan Region.
- Space Considerations.
- Greater availability in case of Planned & Forced Outages

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Case Study

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ASSUMPTIONS/ CONSIDERATIONS IN THE STUDIES FOR KOLODYNE-I

- Hydrological Year - June to May.
- Tail Water Level - 176.3.m (Constant)
- Overall TG Efficiency - 90% (Constant)
- Head Loss in WCS - 2.5m (Constant)
- Dependability Criteria - 90%
- New Zero Elevation - 279 m
- Constraint on Dam Ht. - 420 m (Submergence of plain land)

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SALIENT FEATURES
KOLODYNE HE PROJECT ST.-I

Location	Lunglei distt, Mizoram
River	Mat River
I.C.	120 MW
Gross Storage-	514 MCM (at FRL-390m)
Live Storage	395 MCM
Annual Runoff-Max	2008 MCM
Min.	726 MCM
Average	1116 MCM
Max. Net Head (m)	about 210 m

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NATURAL INFLOWS(CUMECS)

Year	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May
1-2	63.7	69.6	83.8	73.3	40.1	11.0	5.3	1.0	0.5	0.9	1.4	4.0
2-3	18.8	72.7	98.0	66.7	36.7	6.4	3.9	1.7	1.0	0.8	1.8	8.3
3-4	38.0	83.3	101.3	120.8	98.0	21.0	6.9	2.8	4.1	3.4	9.5	21.6
4-5	26.0	58.5	135.4	81.3	68.8	9.6	3.0	1.0	0.8	1.5	3.5	32.5
5-6	53.5	73.1	52.8	60.0	19.2	0.7	0.8	0.3	0.1	0.5	1.4	12.3
6-7	43.3	93.5	86.4	70.1	35.3	10.1	2.2	0.5	0.2	0.7	0.7	2.2
7-8	8.6	90.8	127.3	49.0	25.0	7.8	4.4	1.8	0.3	3.4	5.1	25.5
8-9	34.6	53.6	93.1	52.5	9.5	5.6	2.5	1.2	1.3	1.9	4.9	45.4
9-10	73.7	47.3	67.1	67.8	22.8	1.8	1.6	1.1	0.3	1.2	2.1	32.5
10-11	37.6	54.4	88.6	85.3	47.30	13.9	4.4	5.4	6.0	14.5	85.1	6.7
11-12	16.8	60.4	46.0	45.7	27.40	4.6	2.7	1.6	0.8	1.1	5.2	77.2
12-13	197.0	49.5	86.6	98.3	64.50	7.1	5.9	2.4	1.2	2.6	2.9	24.1
13-14	49.8	63.6	72.2	135.1	219.0	16.9	6.8	5.2	15.0	26.2	109.5	41.9
14-15	102.2	216.8	96.1	55.2	21.70	6.5	3.0	1.2	0.9	1.2	3.4	7.6
15-16	32.5	110.6	64.9	72.5	31.8	8.1	2.4	0.7	0.8	2.0	1.4	38.8
16-17	85.4	66.0	76.4	52.6	45.8	22.0	9.8	2.9	3.6	3.5	33.4	54.5
17-18	107.8	45.2	92.6	41.6	16.1	7.7	3.1	2.2	2.4	3.1	3.1	68.9
18-19	39.0	115.3	50.1	73.6	40.4	54.7	13.9	2.3	2.7	14.0	51.5	79.8
19-20	106.7	160.4	93.1	56.6	18.7	7.5	3.0	1.3	2.0	2.9	11.0	44.3

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AREA-CAPACITY CHARACTERISTICS

Elevation (m)	Area (Sq.Km.)	Capacity (Mcum.)
252	0	0
260	0.20	0.80
280	0.85	10.15
300	1.89	37.85
320	2.95	86.35
340	4.19	157.25
360	6.22	258.90
380	8.94	409.40
400	19.89	673.57

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EVAPORATION LOSSES

Month	Evaporation (mm)
Jan	45.0
Feb	65.0
Mar	122.0
Apr	130.0
May	139.0
Jun	115.0
Jul	111.0
Aug	108.0
Sep	91.0
Oct	80.0
Nov	47.0
Dec	40.0

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FRL, MDDL AND FIRM POWER

FRL (m)	MDDL (m)	Firm Power (MW)
350	285	13.80
	290	13.95
	295	13.97
	300	14.41
	305	14.50
360	300	17.44
	304	17.65
	308	18.21
	312	18.25
	316	18.30

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FRL, MDDL AND FIRM POWER

FRL (m)	MDDL (m)	Firm Power (MW)
370	310	22.00
	312	22.10
	315	23.36
	317	23.42
	320	23.45
380	318	28.70
	320	28.80
	322	29.91
	325	29.98
	327	30.00

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FRL, MDDL AND FIRM POWER

FRL (m)	MDDL (m)	Firm Power (MW)
390	327	37.49
	330	38.10
	332	38.20
	335	38.30
	340	38.35
400	330	45.12
	332	46.24
	335	46.30
	337	48.14
	340	48.50

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FRL, MDDL AND FIRM POWER

FRL (m)	MDDL (m)	Firm Power (MW)
410	340	48.55
	342	48.60
	344	49.95
	346	50.00
	348	50.21
420	344	49.94
	346	49.98
	348	50.00
	350	50.70
	352	50.75

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INCREMENTAL FIRM POWER AND ENERGY

FRL (M)	MDDL (m)	Firm Power (MW)	Incremental Firm Power (MW)	I.C. (MW)	Annual Energy (GWh)	Incremental Energy (GWh)
350	300	14.41	-	45	204.78	-
360	308	18.21	3.80	55	240.67	35.89
370	315	23.36	5.15	70	280.76	40.09
380	322	29.91	6.56	90	348.77	68.01
390	330	38.10	8.19	120	392.86	44.09
400	337	48.14	10.04	150	421.75	28.89
410	344	49.95	1.81	150	437.60	15.85
420	350	50.70	0.75	150	444.30	6.70

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COST OF ENERGY GENERATION

FRL (m)	Installed Capacity (MW)	Total Cost (Rs.Crores)	Present Worth of Energy (GWh)	Cost of Generation (Rs./kWh)	Incremental Cost of Generation (Rs./kWh)
350	45	315.50	1760.95	2.52	-
360	55	373.00	2069.58	2.53	2.61
370	70	443.00	2414.32	2.58	2.85
380	90	526.50	2999.15	2.47	2.01
390	120	633.50	3378.29	2.63	3.96
400	150	741.50	3626.72	2.87	6.11
410	150	808.44	3763.06	3.02	6.90
420	150	938.93	3820.64	3.45	31.83

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INCREMENTAL COST PER MW

FRL (M)	I.C. (MW)	Cost of Civil Works (Rs.Crores)	Cost of Elect. Works (Rs.Crores)	Total Cost (Rs.Crores)	Cost per Mw (Rs.Crs)	Incremental Cost of I.C. per MW
350	45	257.00	58.50	315.50	7.01	-
360	55	301.50	71.50	373.00	6.78	5.75
370	70	352.00	91.00	443.00	6.33	4.67
380	90	409.50	117.00	526.50	5.85	4.18
390	120	477.50	156.00	633.50	5.28	3.57
400	150	546.50	195.00	741.50	4.94	3.60
410	150	613.44	195.00	808.44	5.39	INFINITY
420	150	743.93	195.00	938.93	6.26	INFINIY

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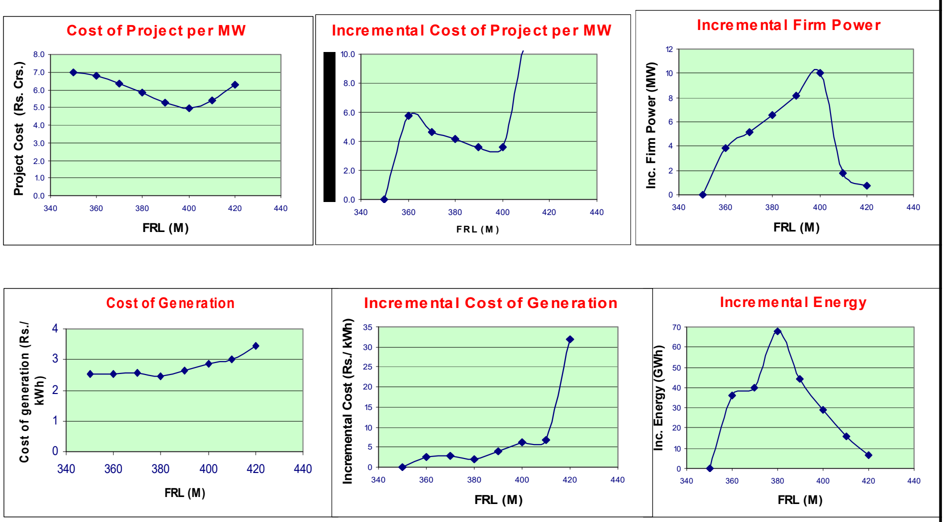
Results of the Study



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CONCLUSIONS

- The incremental firm power shows an increasing trend up to FRL-400M and decreases sharply beyond 400m.
- The incremental energy generation shows an increasing pattern upto FRL of 380 M and declines further.
- The cost per MW of proposed installed capacity shows A decreasing trend upto FRL of 400m and rises sharply beyond 400m.
- The per MW cost of incremental energy is minimum at FRL of 390m.

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CONCLUSIONS

- Cost of energy generation is lowest for the FRL of 380 m viz. Rs. 2.47/kWh. It increases marginally for FRL of 390 m and rises sharply beyond 390 m.
- Incremental cost of energy generation viz. Rs. 2.01/kWh is also minimum at FRL of 380 m and shows an increasing trend for FRL beyond 380 m.
- Therefore, it can be concluded that the FRL of 380m is the optimum FRL for this project and corresponding installed capacity of 120 MW should be provided at the project

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SELECTION OF UNIT SIZE

Unit Size Considered: 30, 40 & 60 MW

Unit Size selected : 60 MW

Advantages:

- **No Transport Constraints envisaged since the power house is located close to National Highway with bridges of 70 R (This loading consists of a tracked vehicle of 700 kN or a wheeled vehicle of total load of 1000 kN as per Indian Road Congress (IRC) Code) load bearing capacity**
- **Economical due to lower cost per MW and would involve lesser civil works in power house and WCS**

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POWER POTENTIAL STUDIES (PUMPED STORAGE SCHEMES)

- Based on available live storage, constraints on operating levels of both the reservoirs and average net head, work out Energy Generation Potential of the reservoir system.
- Based on energy generation potential of the reservoir system and future peak demand requirements of the state/region, suitable Installed Capacity can be selected which should provide peaking capacity in the range of 3-6 hours.

  Energy Generation Potential (MWh) = Installed Capacity (MW) x No. of operating hours
- Select unit size by considering
  - Geological constraints for design of water conductor system, power house etc., if any.
  - Transportation constraints, if any.

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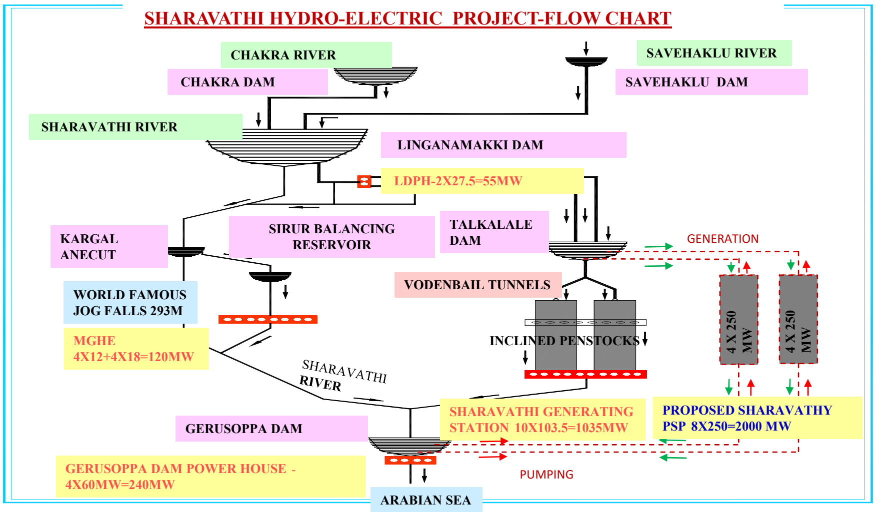
SCHEMATIC DIAGRAM



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POWER POTENTIAL STUDIES (SHARAVATHY PSP IN KARNATAKA)

- Sharavathy PSP is being planned on river Sharavathy utilizing waters of Talaklale reservoir (Upper Reservoir) and Gerusoppa reservoir (Lower Reservoir)
- Live storage of upper reservoir is 13.9 MCM , out of which 3.2 MCM would continue to be used for existing Sharavathy HEP (1035 MW)
- Around 10.4 MCM spare live storage is available which could be used for peaking purpose

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DATA/ PARAMETERS OF STUDY

Operating Levels and Storage available at Existing Reservoirs

Talakalale - Upper reservoir			Gerusoppa reservoir		
	Level (m)	Storage (MCum)		Level (m)	Storage (MCum)
FRL	522.12	129.65	FRL	55.00	130.74
MDDL	520.59	115.78	MDDL	43.50	72.04
Live Storage		13.87	Live Storage		58.70

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DATA/ PARAMETERS OF STUDY

- Head losses in WCS - 6 m
- Average Net Head - 466 m
- Generating Mode Efficiency - 91.1 %
- Pumping Mode Efficiency - 90 %
- Overall Cycle Efficiency - 80 %

Power in MW (for 6 Hrs of peaking operation)	Storage Required (In MCM)
1000	5.2
1250	6.5
1500	7.8
1750	9.1
2000	10.4

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DATA/ PARAMETERS OF STUDY (GENERATING MODE)

Scenario 1: Operating Levels considered for Operation Simulation

Talakalale - Upper reservoir			Gerusoppa - Lower reservoir		
	Level (m)	Storage (MCu)		Level (m)	Storage (MCum)
FRL-PS Operation	522.12	129.65	FRL-PS Operation	45.79	82.30
MDDL-PS Operation	521.00	119.39	MDDL-PS Operation	43.50	72.04
Pondage for PS Operation		10.26	Pondage for PS Operation		10.26

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Reservoir Operation Simulation (Generating Mode)

				Upper Reservoir					
Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Initial Pond Level (m)	Initial Storage (MCum)	Outflow from Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)
1	10.0	2000.0	473.50	522.12	129.65	0.28	129.37	522.09	522.11
2	10.0	2000.0	473.60	522.09	129.37	0.28	129.08	522.06	522.08
3	10.0	2000.0	473.69	522.06	129.08	0.28	128.80	522.03	522.05
4	10.0	2000.0	473.79	522.03	128.80	0.28	128.51	522.00	522.02
5	10.0	2000.0	473.89	522.00	128.51	0.28	128.23	521.97	521.99
6	10.0	2000.0	473.99	521.97	128.23	0.28	127.95	521.94	521.96
7	10.0	2000.0	474.08	521.94	127.95	0.28	127.66	521.91	521.93
8	10.0	2000.0	474.18	521.91	127.66	0.28	127.38	521.88	521.90
9	10.0	2000.0	474.28	521.88	127.38	0.28	127.09	521.85	521.87
10	10.0	2000.0	474.37	521.85	127.09	0.28	126.81	521.81	521.83
11	10.0	2000.0	474.47	521.81	126.81	0.28	126.52	521.78	521.80
12	10.0	2000.0	474.56	521.78	126.52	0.28	126.24	521.75	521.77
13	10.0	2000.0	474.66	521.75	126.24	0.28	125.95	521.72	521.74
14	10.0	2000.0	474.76	521.72	125.95	0.28	125.67	521.69	521.71
15	10.0	2000.0	474.85	521.69	125.67	0.28	125.38	521.66	521.68
16	10.0	2000.0	474.95	521.66	125.38	0.28	125.10	521.63	521.65
17	10.0	2000.0	475.05	521.63	125.10	0.29	124.81	521.60	521.62
18	10.0	2000.0	475.14	521.60	124.81	0.29	124.53	521.57	521.59

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Reservoir Operation Simulation (Generating Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Upper Reservoir Initial Pond Level (m)	Initial Storage (MCum)	Outflow from Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)
19	10.0	2000.0	475.24	521.57	124.53	0.29	124.24	521.54	521.56
20	10.0	2000.0	475.33	521.54	124.24	0.29	123.96	521.50	521.53
21	10.0	2000.0	475.43	521.50	123.96	0.29	123.67	521.47	521.49
22	10.0	2000.0	475.53	521.47	123.67	0.29	123.39	521.44	521.46
23	10.0	2000.0	475.62	521.44	123.39	0.29	123.10	521.41	521.43
24	10.0	2000.0	475.72	521.41	123.10	0.29	122.82	521.38	521.40
25	10.0	2000.0	475.81	521.38	122.82	0.29	122.53	521.35	521.37
26	10.0	2000.0	475.91	521.35	122.53	0.29	122.25	521.32	521.34
27	10.0	2000.0	476.00	521.32	122.25	0.29	121.96	521.28	521.30
28	10.0	2000.0	476.10	521.28	121.96	0.29	121.67	521.25	521.27
29	10.0	2000.0	476.20	521.25	121.67	0.29	121.39	521.22	521.24
30	10.0	2000.0	476.29	521.22	121.39	0.29	121.10	521.19	521.21
31	10.0	2000.0	476.39	521.19	121.10	0.29	120.82	521.16	521.18
32	10.0	2000.0	476.48	521.16	120.82	0.29	120.53	521.13	521.15
33	10.0	2000.0	476.58	521.13	120.53	0.29	120.25	521.09	521.11
34	10.0	2000.0	476.67	521.09	120.25	0.29	119.96	521.06	521.08
35	10.0	2000.0	476.77	521.06	119.96	0.29	119.67	521.03	521.05
36	10.0	2000.0	476.85	521.03	119.67	0.29	119.39	521.00	521.02
	360.0					10.26			

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Reservoir Operation Simulation (Generating Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Lower Reservoir Initial Pond Level (m)	Initial Storage (MCum)	Inflow into Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)	Average Net Head (m)
1	10.0	2000.0	473.50	43.50	72.04	0.28	72.32	43.57	43.54	472.57
2	10.0	2000.0	473.60	43.57	72.32	0.28	72.61	43.63	43.61	472.47
3	10.0	2000.0	473.69	43.63	72.61	0.28	72.89	43.70	43.68	472.37
4	10.0	2000.0	473.79	43.70	72.89	0.28	73.18	43.76	43.74	472.28
5	10.0	2000.0	473.89	43.76	73.18	0.28	73.46	43.83	43.81	472.18
6	10.0	2000.0	473.99	43.83	73.46	0.28	73.75	43.90	43.87	472.08
7	10.0	2000.0	474.08	43.90	73.75	0.28	74.03	43.96	43.94	471.99
8	10.0	2000.0	474.18	43.96	74.03	0.28	74.31	44.03	44.01	471.89
9	10.0	2000.0	474.28	44.03	74.31	0.28	74.60	44.09	44.07	471.79
10	10.0	2000.0	474.37	44.09	74.60	0.28	74.88	44.16	44.14	471.70
11	10.0	2000.0	474.47	44.16	74.88	0.28	75.17	44.22	44.20	471.60
12	10.0	2000.0	474.56	44.22	75.17	0.28	75.45	44.29	44.27	471.51
13	10.0	2000.0	474.66	44.29	75.45	0.28	75.74	44.35	44.33	471.41
14	10.0	2000.0	474.76	44.35	75.74	0.28	76.02	44.42	44.40	471.32
15	10.0	2000.0	474.85	44.42	76.02	0.28	76.31	44.48	44.46	471.22
16	10.0	2000.0	474.95	44.48	76.31	0.28	76.59	44.55	44.53	471.12
17	10.0	2000.0	475.05	44.55	76.59	0.29	76.88	44.61	44.59	471.03
18	10.0	2000.0	475.14	44.61	76.88	0.29	77.16	44.67	44.65	470.93

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Reservoir Operation Simulation (Generating Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Lower Reservoir							Average Net Head (m)
				Initial Pond Level (m)	Initial Storage (MCum)	Inflow into Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)		
19	10.0	2000.0	475.24	44.67	77.16	0.29	77.45	44.74	44.72		470.84
20	10.0	2000.0	475.33	44.74	77.45	0.29	77.73	44.80	44.78		470.74
21	10.0	2000.0	475.43	44.80	77.73	0.29	78.02	44.87	44.84		470.65
22	10.0	2000.0	475.53	44.87	78.02	0.29	78.30	44.93	44.91		470.55
23	10.0	2000.0	475.62	44.93	78.30	0.29	78.59	44.99	44.97		470.46
24	10.0	2000.0	475.72	44.99	78.59	0.29	78.87	45.06	45.04		470.36
25	10.0	2000.0	475.81	45.06	78.87	0.29	79.16	45.12	45.10		470.27
26	10.0	2000.0	475.91	45.12	79.16	0.29	79.45	45.18	45.16		470.18
27	10.0	2000.0	476.00	45.18	79.45	0.29	79.73	45.25	45.22		470.08
28	10.0	2000.0	476.10	45.25	79.73	0.29	80.02	45.31	45.29		469.99
29	10.0	2000.0	476.20	45.31	80.02	0.29	80.30	45.37	45.35		469.89
30	10.0	2000.0	476.29	45.37	80.30	0.29	80.59	45.43	45.41		469.80
31	10.0	2000.0	476.39	45.43	80.59	0.29	80.87	45.50	45.48		469.70
32	10.0	2000.0	476.48	45.50	80.87	0.29	81.16	45.56	45.54		469.61
33	10.0	2000.0	476.58	45.56	81.16	0.29	81.45	45.62	45.60		469.51
34	10.0	2000.0	476.67	45.62	81.45	0.29	81.73	45.68	45.66		469.42
35	10.0	2000.0	476.77	45.68	81.73	0.29	82.02	45.74	45.72		469.33
36	10.0	2000.0	476.85	45.74	82.02	0.29	82.30	45.79	45.77		469.25
	360.0					10.26					

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Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Upper Reservoir					
				Initial Pond Level (m)	Initial Storage (MCum)	Inflow into the Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)
1	10.0	2000.0	381.27	521.00	119.39	0.23	119.62	521.02	521.01
2	10.0	2000.0	381.21	521.02	119.62	0.23	119.84	521.05	521.04
3	10.0	2000.0	381.15	521.05	119.84	0.23	120.07	521.07	521.07
4	10.0	2000.0	381.10	521.07	120.07	0.23	120.30	521.10	521.09
5	10.0	2000.0	381.04	521.10	120.30	0.23	120.53	521.13	521.12
6	10.0	2000.0	380.98	521.13	120.53	0.23	120.76	521.15	521.14
7	10.0	2000.0	380.92	521.15	120.76	0.23	120.99	521.18	521.17
8	10.0	2000.0	380.88	521.18	120.99	0.23	121.22	521.20	521.19
9	10.0	2000.0	380.82	521.20	121.22	0.23	121.44	521.23	521.22
10	10.0	2000.0	380.76	521.23	121.44	0.23	121.67	521.25	521.24
11	10.0	2000.0	380.70	521.25	121.67	0.23	121.90	521.28	521.27
12	10.0	2000.0	380.64	521.28	121.90	0.23	122.13	521.30	521.29
13	10.0	2000.0	380.58	521.30	122.13	0.23	122.36	521.33	521.32
14	10.0	2000.0	380.53	521.33	122.36	0.23	122.59	521.35	521.34
15	10.0	2000.0	380.47	521.35	122.59	0.23	122.82	521.38	521.37

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Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Upper Reservoir Initial Pond Level (m)	Initial Storage (MCum)	Inflow into the Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)
16	10.0	2000.0	380.41	521.38	122.82	0.23	123.04	521.40	521.39
17	10.0	2000.0	380.35	521.40	123.04	0.23	123.27	521.43	521.42
18	10.0	2000.0	380.29	521.43	123.27	0.23	123.50	521.45	521.45
19	10.0	2000.0	380.23	521.45	123.50	0.23	123.73	521.48	521.47
20	10.0	2000.0	380.17	521.48	123.73	0.23	123.96	521.50	521.50
21	10.0	2000.0	380.11	521.50	123.96	0.23	124.18	521.53	521.52
22	10.0	2000.0	380.05	521.53	124.18	0.23	124.41	521.55	521.55
23	10.0	2000.0	379.99	521.55	124.41	0.23	124.64	521.58	521.57
24	10.0	2000.0	379.93	521.58	124.64	0.23	124.87	521.60	521.60
25	10.0	2000.0	379.87	521.60	124.87	0.23	125.10	521.63	521.62
26	10.0	2000.0	379.81	521.63	125.10	0.23	125.32	521.65	521.64
27	10.0	2000.0	379.75	521.65	125.32	0.23	125.55	521.68	521.67
28	10.0	2000.0	379.69	521.68	125.55	0.23	125.78	521.70	521.69
29	10.0	2000.0	379.63	521.70	125.78	0.23	126.01	521.73	521.72
30	10.0	2000.0	379.57	521.73	126.01	0.23	126.23	521.75	521.74

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Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Upper Reservoir Initial Pond Level (m)	Initial Storage (MCum)	Inflow into the Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)
31	10.0	2000.0	379.51	521.75	126.23	0.23	126.46	521.78	521.77
32	10.0	2000.0	379.45	521.78	126.46	0.23	126.69	521.80	521.79
33	10.0	2000.0	379.39	521.80	126.69	0.23	126.92	521.83	521.82
34	10.0	2000.0	379.33	521.83	126.92	0.23	127.15	521.85	521.84
35	10.0	2000.0	379.27	521.85	127.15	0.23	127.37	521.88	521.87
36	10.0	2000.0	379.21	521.88	127.37	0.23	127.60	521.90	521.89
37	10.0	2000.0	379.15	521.90	127.60	0.23	127.83	521.92	521.92
38	10.0	2000.0	379.09	521.92	127.83	0.23	128.06	521.95	521.94
39	10.0	2000.0	379.02	521.95	128.06	0.23	128.28	521.97	521.96
40	10.0	2000.0	378.96	521.97	128.28	0.23	128.51	522.00	521.99
41	10.0	2000.0	378.90	522.00	128.51	0.23	128.74	522.02	522.01
42	10.0	2000.0	378.84	522.02	128.74	0.23	128.97	522.05	522.04
43	10.0	2000.0	378.78	522.05	128.97	0.23	129.19	522.07	522.06
44	10.0	2000.0	378.72	522.07	129.19	0.23	129.42	522.10	522.09
45	10.0	2000.0	378.66	522.10	129.42	0.23	129.65	522.12	522.11
	450.0					10.26			

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Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Lower Reservoir						Average Pond Level (m)
				Initial Pond Level (m)	Initial Storage (MCum)	Outflow from Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)	
1	10.0	2000.0	381.27	45.79	82.20	0.23	81.98	45.74	45.77	481.25
2	10.0	2000.0	381.21	45.74	81.98	0.23	81.75	45.69	45.72	481.32
3	10.0	2000.0	381.15	45.69	81.75	0.23	81.52	45.64	45.67	481.40
4	10.0	2000.0	381.10	45.64	81.52	0.23	81.29	45.59	45.62	481.47
5	10.0	2000.0	381.04	45.59	81.29	0.23	81.06	45.54	45.57	481.55
6	10.0	2000.0	380.98	45.54	81.06	0.23	80.83	45.49	45.52	481.62
7	10.0	2000.0	380.92	45.49	80.83	0.23	80.60	45.44	45.47	481.70
8	10.0	2000.0	380.88	45.44	80.60	0.23	80.38	45.39	45.42	481.74
9	10.0	2000.0	380.82	45.39	80.38	0.23	80.15	45.34	45.37	481.81
10	10.0	2000.0	380.76	45.34	80.15	0.23	79.92	45.29	45.32	481.89
11	10.0	2000.0	380.70	45.29	79.92	0.23	79.69	45.24	45.27	481.97
12	10.0	2000.0	380.64	45.24	79.69	0.23	79.46	45.19	45.22	482.04
13	10.0	2000.0	380.58	45.19	79.46	0.23	79.23	45.14	45.17	482.12
14	10.0	2000.0	380.53	45.14	79.23	0.23	79.00	45.09	45.12	482.19
15	10.0	2000.0	380.47	45.09	79.00	0.23	78.78	45.04	45.07	482.27

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Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Lower Reservoir						Average Pond Level (m)
				Initial Pond Level (m)	Initial Storage (MCum)	Outflow from Pond (MCum)	Final Storage (MCum)	Final Pond Level (m)	Average Pond Level (m)	
16	10.0	2000.0	381.27	45.04	78.78	0.23	78.55	44.98	45.02	482.34
17	10.0	2000.0	381.21	44.98	78.55	0.23	78.32	44.93	44.97	482.42
18	10.0	2000.0	381.15	44.93	78.32	0.23	78.09	44.88	44.92	482.50
19	10.0	2000.0	381.10	44.88	78.09	0.23	77.86	44.83	44.87	482.57
20	10.0	2000.0	381.04	44.83	77.86	0.23	77.64	44.78	44.81	482.65
21	10.0	2000.0	380.98	44.78	77.64	0.23	77.41	44.73	44.76	482.72
22	10.0	2000.0	380.92	44.73	77.41	0.23	77.18	44.68	44.71	482.80
23	10.0	2000.0	380.88	44.68	77.18	0.23	76.95	44.63	44.66	482.88
24	10.0	2000.0	380.82	44.63	76.95	0.23	76.72	44.58	44.61	482.95
25	10.0	2000.0	380.76	44.58	76.72	0.23	76.50	44.52	44.56	483.03
26	10.0	2000.0	380.70	44.52	76.50	0.23	76.27	44.47	44.51	483.10
27	10.0	2000.0	380.64	44.47	76.27	0.23	76.04	44.42	44.46	483.18
28	10.0	2000.0	380.58	44.42	76.04	0.23	75.81	44.37	44.40	483.26
29	10.0	2000.0	380.53	44.37	75.81	0.23	75.58	44.32	44.35	483.33
30	10.0	2000.0	380.47	44.32	75.58	0.23	75.36	44.27	44.30	483.41

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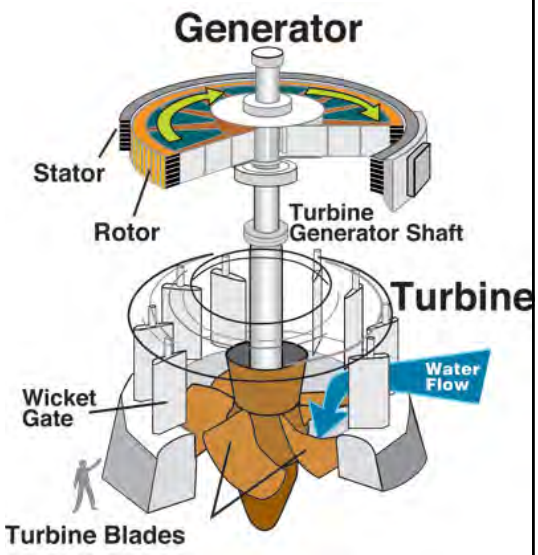
Reservoir Operation Simulation (Pumping Mode)

Interval No	Time Interval (Minutes)	Station Output (MW)	Discharge (cumecs)	Lower Reservoir: Initial Pond Level (m)	Lower Reservoir: Initial Storage (MCum)	Lower Reservoir: Outflow from Pond (MCum)	Lower Reservoir: Final Storage (MCum)	Lower Reservoir: Final Pond Level (m)	Lower Reservoir: Average Pond Level (m)	Average Pond Level (m)
31	10.0	2000.0	379.51	44.27	75.36	0.23	75.13	44.21	44.25	483.49
32	10.0	2000.0	379.45	44.21	75.13	0.23	74.90	44.16	44.20	483.56
33	10.0	2000.0	379.39	44.16	74.90	0.23	74.67	44.11	44.14	483.64
34	10.0	2000.0	379.33	44.11	74.67	0.23	74.45	44.06	44.09	483.72
35	10.0	2000.0	379.27	44.06	74.45	0.23	74.22	44.01	44.04	483.79
36	10.0	2000.0	379.21	44.01	74.22	0.23	73.99	43.95	43.99	483.87
37	10.0	2000.0	379.15	43.95	73.99	0.23	73.76	43.90	43.94	483.95
38	10.0	2000.0	379.09	43.90	73.76	0.23	73.54	43.85	43.88	484.02
39	10.0	2000.0	379.02	43.85	73.54	0.23	73.31	43.79	43.83	484.10
40	10.0	2000.0	378.96	43.79	73.31	0.23	73.08	43.74	43.78	484.18
41	10.0	2000.0	378.90	43.74	73.08	0.23	72.85	43.69	43.72	484.26
42	10.0	2000.0	378.84	43.69	72.85	0.23	72.63	43.64	43.67	484.33
43	10.0	2000.0	378.78	43.64	72.63	0.23	72.40	43.58	43.62	484.41
44	10.0	2000.0	378.72	43.58	72.40	0.23	72.17	43.53	43.57	484.49
45	10.0	2000.0	378.66	43.53	72.17	0.23	71.94	43.48	43.51	484.56
	450.0					10.26				

123

Hydraulic Turbines overview

- Hydraulic turbines : water turns blades to produce rotation
- Potential/Kinetic power ⟶ Mechanical Power ⟶ Electric power



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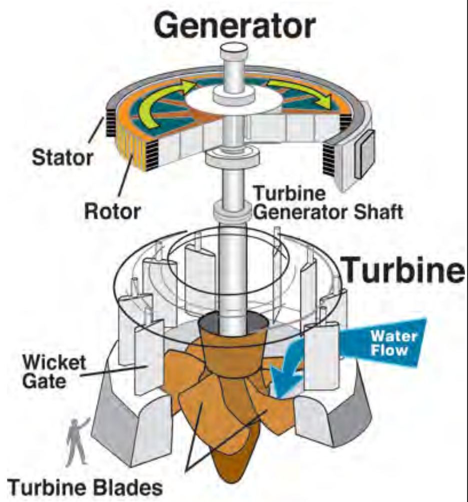
Hydraulic Turbines overview

Components of a Turbine :

- Runner : rotational part (Shaft + blades)
- Diffuser (at output) : channel-shaped increasing in size
- Distributor : optional component at the input that controls the angle of attack



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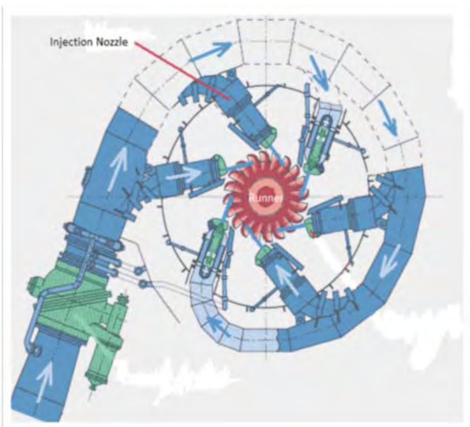
THE PELTON TURBINE



- Action turbine
- No diffuser
- High operating head
- The wheel is not submerged
- The number of opened nozzle changes the power

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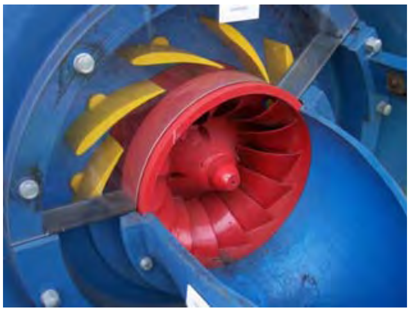
THE FRANCIS TURBINE

- Reaction turbine
- Change of the flow pressure
- Operating head between 20 and 500 meters
- Two main components: upstream guide vanne and runner



127

THE KAPLAN TURBINE

- Evolution of the Francis turbine
- The guidvanes and runner adapted to the waterflow
- High efficiency for a wide range of water flow
- Can be used as a pump for Storage pumping

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TURBINES RUNNERS

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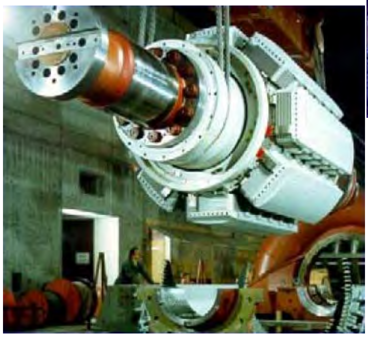
CHOICE METHOD OF HYDRAULIC TURBINES

- Ns = (120*f)/P, where f: frequency and P: no. of Poles
- For a same electric frequency (50 Hz) :
  - Low speed ➔ many poles
  - High speed ➔ less poles
- Many poles



4 pairs 750 rpm

40 pairs – 75 rpm

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CHOICE METHOD OF HYDRAULIC TURBINES

- Characteristics of water flow and turbine

Power of the turbine: $P = \rho g H \dot{v}$

Where ρ is efficiency, g is specific gravity, H is Head and $\dot{v}$ is specific speed

Angular velocity: $\omega_{turbine} \propto \sqrt{H}$ ➡ **N**

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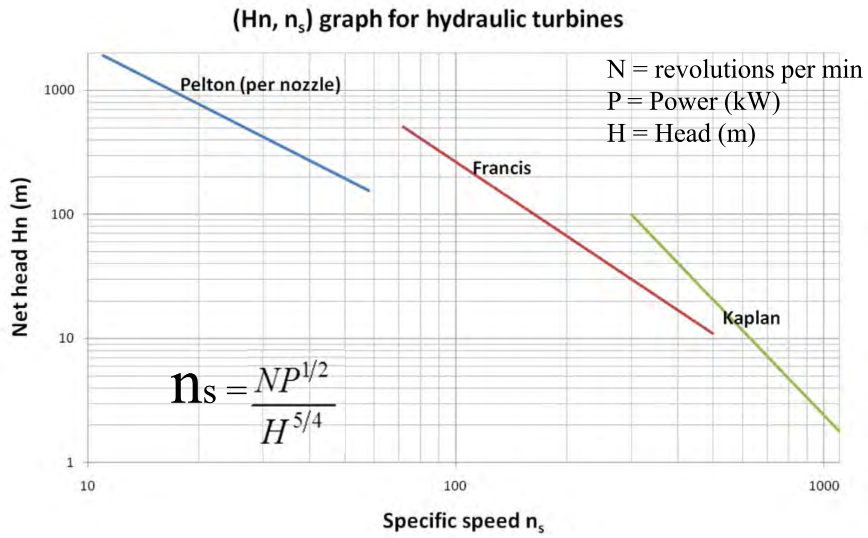
CHOICE METHOD OF HYDRAULIC TURBINES

Comparison of the specific speed with widely built turbines



$$n_s = \frac{NP^{1/2}}{H^{5/4}}$$

132

Environment and Forest issues

Issue:

Due to the considerable time taken in the process of Environment and Forest Clearances due to various issues relating to **e-flows, free flow stretch requirement, Longitudinal Connectivity** etc., commencement of construction works of Hydro projects often gets delayed.

Suggested solution:

- It is desirable that all the clearances relating to Environment & Forest, Wildlife etc. should be given in time bound manner.
- The e-flows may be prescribed for hydro projects considering case to case basis and in a judicious manner.
- **Free flow stretch requirements** should be based on river gradient and velocity.
- The e-flows, once prescribed should not be revisited for a project.

133

Land Acquisition & R&R Issues

Issue:

In some of the States like Arunachal Pradesh, Land records are not available, as such, land acquisition becomes an issue.

Dislocation of the people from their houses/ workplaces etc. and their resettlement is a sensitive issue and involves a lot of time and money. Many times this issue leads to court cases resulting in delay in project execution/completion.

Suggested solution:

State should play a pro-active role in Land acquisition and R&R to mitigate the issues.

134

Inadequate Infrastructural facilities

Issue:

Hydro projects are normally located in difficult terrain having poor accessibility. As such, substantial time is lost due to lack of adequate Infrastructural facilities at the project site allotted to a developer by the State Govt.

Suggested solution:

Adequate infrastructure facilities need to be developed in the States by concerned agencies, matching with the schedule of development of hydro projects to reduce their gestation period.

135

Law & Order / Local issues

Issue:

Protests by the local people against the construction activities like blasting, muck disposal etc. and demands for employment, extra compensation etc. often create law and order problems which delays the commencement and affects progress of the works.

Suggested solution:

State should play a pro-active role to provide a conducive environment for construction of hydro projects.

Implementation of various Corporate Social Responsibility Plans and proper co-ordination with local bodies & State Authorities can also minimize the issues.

136

Geological Surprises

Issue:

A large number of HE projects has been delayed due to geological surprises.

Suggested solution:

With the use of latest technologies in Survey & Investigation and preparation of Bankable DPR, the geological surprises could be reduced to some extent.

137

Natural Calamities

Issue:

Natural calamities like unprecedented rain / flash floods, cloud burst, earthquake etc delay the completion of project.

Suggested solution:

Efficient preparedness and Disaster Management Plan should be in place to tackle Natural Calamaties.

138

Inter-State Issues

Issue:

Delay in implementation due to inter-state disputes between the states.

Suggested solution:

Concerned State Governments have to play active role to resolve the inter-state matters for hydro development in their states.

Pending resolution of inter-state aspects, some of the projects could be taken up in Central Sector to avoid time and cost overruns.

139

High Tariff of Hydro Projects

Issue:

Tariff from hydro projects has tended to be higher in initial years as compared to other sources of power (conventional as well as renewable sources) mainly due to construction of complex structures which have long gestation period, unavailability of loans of lower interest rate & longer tenures, high R&R cost, infrastructure (roads & bridges) cost etc. As such, many hydro projects even after commissioning are facing financial distress due to dishonoring of PPAs / non-signing of PPAs.

- **Suggested solution:**By providing Longer tenure and lower interest bearing loan instruments.
- Excluding cost of enabling infrastructure from project cost for tariff calculations and reimbursement of the same from appropriate funds of the concerned department/ entities of the GOI/ State Govts.
- Extending fiscal/financial incentives to all hydro projects.

140

Levying of Water Cess

Issue:

Levying of water cess by some of the States also affects the viability of the project and increased the tariff of the order of 50p-Rs 1/unit.

Suggested solution:

Hydro projects are already contributing significantly towards the revenue of the state and the decision of the State Govts. to impose water cess on hydro-electric projects would only result in increase in tariff making them unviable in some cases. As such, withdrawal of free power to State Govts. which charge Water Cess could be considered. Withdrawal of free power to State Govts. which charge Water Cess could be considered.

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Levying of Water Cess

Sl. No.	Power Station	Installed Capacity (MW)	Annual Design Energy (MU)	Composite Tariff for the year 2018-19 without water usage charges	Composite Tariff with J&K Water usage charges		Water Usage (MCM)
1	SALAL	690	3082	1.23	1.12	2.35	360.91
2	URI	480	2587.38	1.64	0.46	2.11	135.37
3	DULHASTI	390	1906.8	5.50	0.49	5.99	109.00
4	URI-II	240	1123.77	4.75	0.87	5.61	133.38
5	Nimoo Bazgo	45	239.33	9.24	0.64	9.88	14.62
6	Chutak	44	212.93	8.26	0.24	8.50	4.62
7	Sewa-II	120	533.53	4.33	0.19	4.52	9.16
8	Kishanganga	330	1712.96	3.38	0.15	3.52	8.10
	Total	2339	11399	3.14	0.63	3.77	**775.16**

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Other Emerging Issues

- Non-implementation of Differential Tariff for Hydro
- Need to value unique attributes of Hydro Power - balancing power, black start capability, reactive power management capability etc.
- Lack of long-term low interest bearing financing
- Issue of Longitudinal Connectivity

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Electro-mechanical Equipment/Systems in Hydro Electric Plant

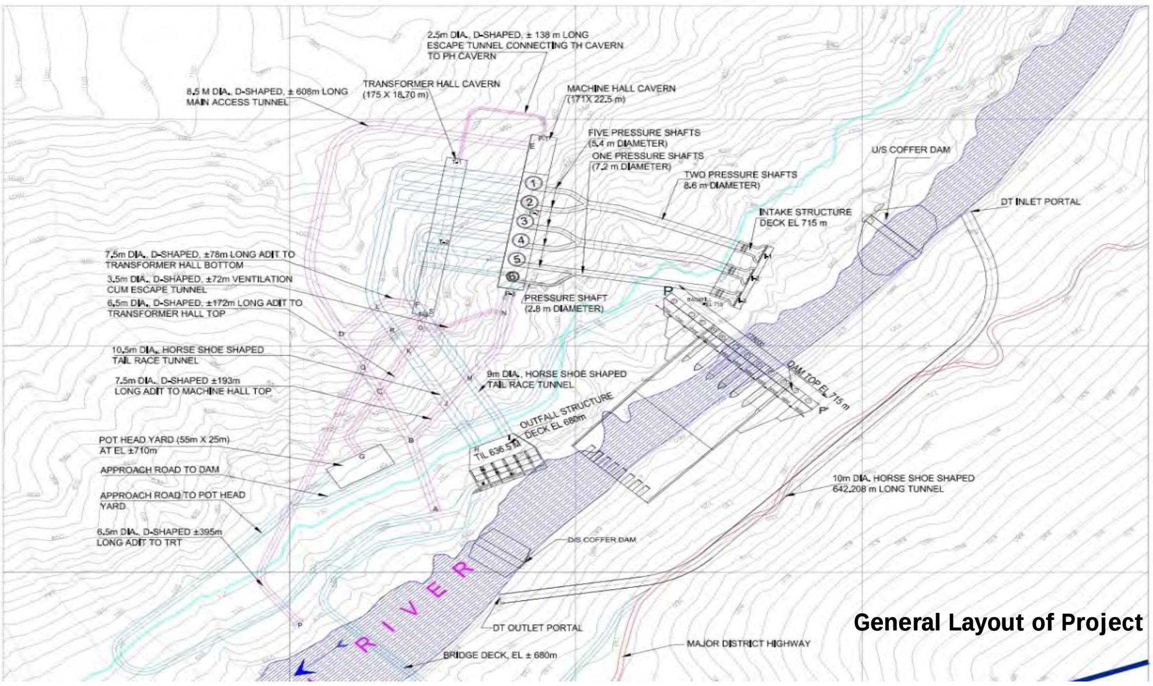
By:

Deepak Sharma

Deputy Director

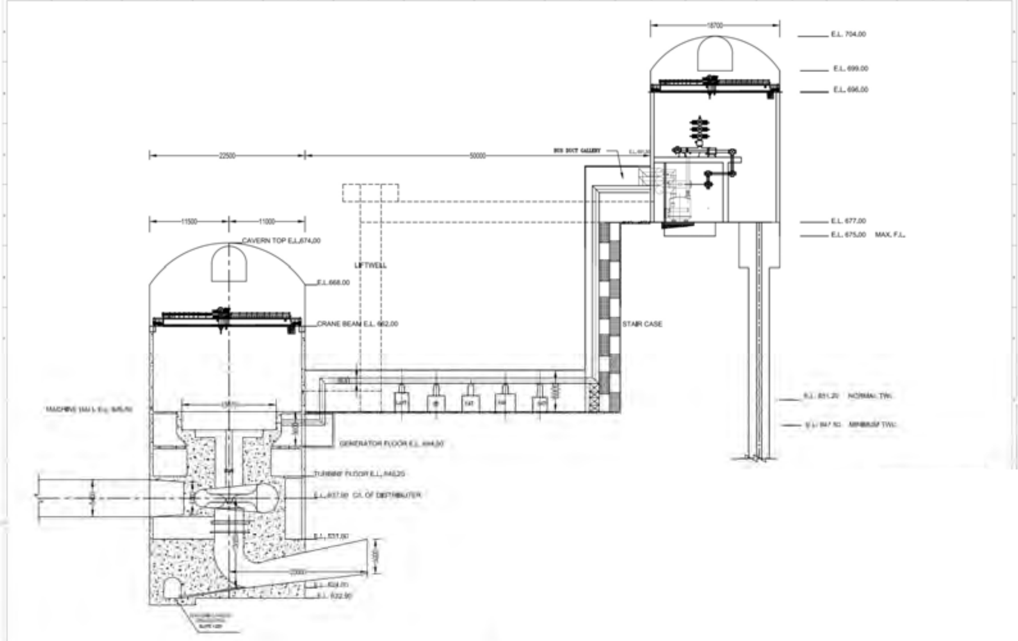
Central Electricity Authority

1



General Layout of Project

2



Cross-section of Power House

3

Main Electro-Mechanical Equipment

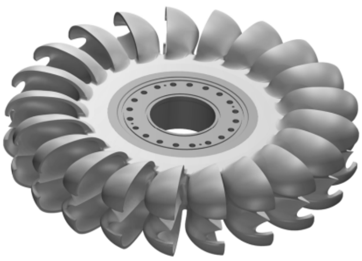
4

Hydraulic Turbines

- Turbine
  - Impulse Turbine
    - Tangential flow
      - Pelton
  - Reaction turbine
    - Axial flow
      - Kaplan
    - Mixed flow
      - Francis

5

Pelton



Francis

Kaplan

6

Turbine Governor

- To regulate the Turbine-Generator from hunting and instability at all levels and ensures stable operation under all possible operating conditions
- To control turbine automatically and manually

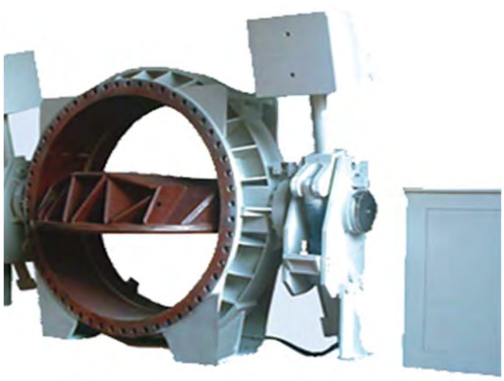
7

Main Inlet Valve (MIV)

- Types:

Spherical Valve

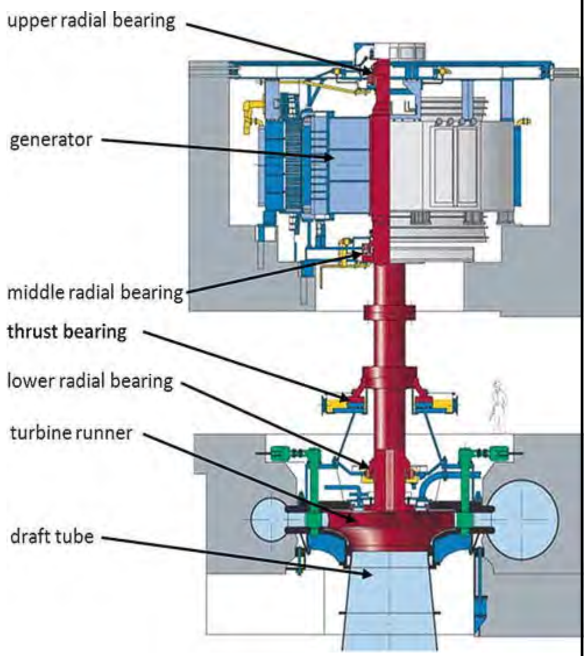
Butterfly Valve



8

Generator & its Excitation System

- It is of rated output to match the turbine output.
- Rated in MVA, kV, 3 phase, 50 Hz, Power Factor and Efficiency
- To be capable of continuously delivering 10% overload.
- Excitation system will be capable of continuously supplying the maximum excitation power required to produce 110% of rated output, at rated generator voltage, rated power factor etc.



9

MAIN STEP UP TRANSFORMER (GT)

- The rating of the transformers shall be matched with the output of Generator.
- LV side is connected to Generator through Bus Duct
- HV side is connected to Switchyard

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Switchyard

- Connected to Main GTs either through XLPE cables or Bus Duct.
- Purpose to evacuate generated power to the grid
- Main components comprise of:

1. Wave Trap
2. Lightning Arrester
3. Current & Potential Transformers
4. Circuit Breaker
5. Isolator
6. Gantry Towers

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Mechanical Auxiliary Systems

12

1. EOT Crane

- Rating is decided by the heaviest item to be lifted in power house
- The heaviest item is usually Assembled Rotor
- EOT crane normally consists of Main Hook along with auxiliary hook and Monorail

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2. COOLING WATER SYSTEM

- Required for cooling of Oil-Bearings of Turbine & Generator, GT oil, Ventilation System.
- Types:

1. Open Loop System – if silt content in water is very less
2. Closed Loop System – if silt content in water is very high
3. Mixed Loop System - if silt content in water is high

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3. DRAINAGE, DEWATERING & STATION FLOOD DEWATERING SYSTEM

- Drainage System is used for regular draining of seepages from power house walls/equipment
- Dewatering System is used for dewatering of Draft Tube, Penstock below Tail Water Level.
- Flood Dewatering System is used in case of flooding of Power House due to unforeseen circumstances
- Submersible type pumps are used for these systems

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4. COMPRESSED AIR SYSTEM

- Provided for generator brakes, turbine shaft seal and station service requirements

5. Oil Purification System

- Provided for filtration/purification of lubricating oil used in bearings and transformer oil

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6. Ventilation and Air-Conditioning System

- Provided for maintaining proper indoor air quality.
- Use of blowers, exhaust fans, air-washers etc. such that requisite air changes can be achieved.
- Some areas of the Power House like Control Room & Conference room etc. are provided with air-conditioning systems.

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7. Fire Protection and Detection System

- Provided to timely detection of the occurrence & quick extinguishing of fire
- Main Types:

1. **AUTOMATIC WATER SPRAY SYSTEM**
2. **FIRE - EXTINGUISHING SYSTEM FROM HYDRANTS**
3. **PORTABLE FIRE EXTINGUISHERS**

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8. LIFT

9. MECHANICAL WORKSHOP EQUIPMENT

10. TEST LABORATORY

11. SECURITY AND SURVEILLANCE

19

Electrical Auxiliary Systems

20

1. STATION AUXILIARY SUPLY SYSTEM

- Station Service Transformers – Used for meeting the Station requirement such as illumination, fire protection system etc.
- Unit Auxiliary Transformers – Used for supplying power to Unit Auxiliaries such as cooling water system, lubrication system etc.
- DG Set- Used in case of emergency and black-start of the unit, rating of DG set based on minimum power requirement to start a single unit and essential station auxiliaries.
- Supply from Local Grid

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2. Illumination System

- To illuminate Power house, Dam site and other areas of the project using energy efficient lighting system such as LED

3. Grounding System

The Power House, Transformer area and Switch Yard area provided with interlinked ground mat, by using mild steel flat of suitable cross section with grounding electrodes to provide protection from Over voltage and leakage current

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4. D.C. Auxiliary Services (220V & 48 V)

- Required for Field Flashing and Electrical Braking.
- Supplies power to measuring instruments.
- Converters are provided for 48V supply from 220V DC Supply. 48V DC is used for communication purposes.
- Provides Emergency Illumination during unforeseen circumstances

5. Public Address System

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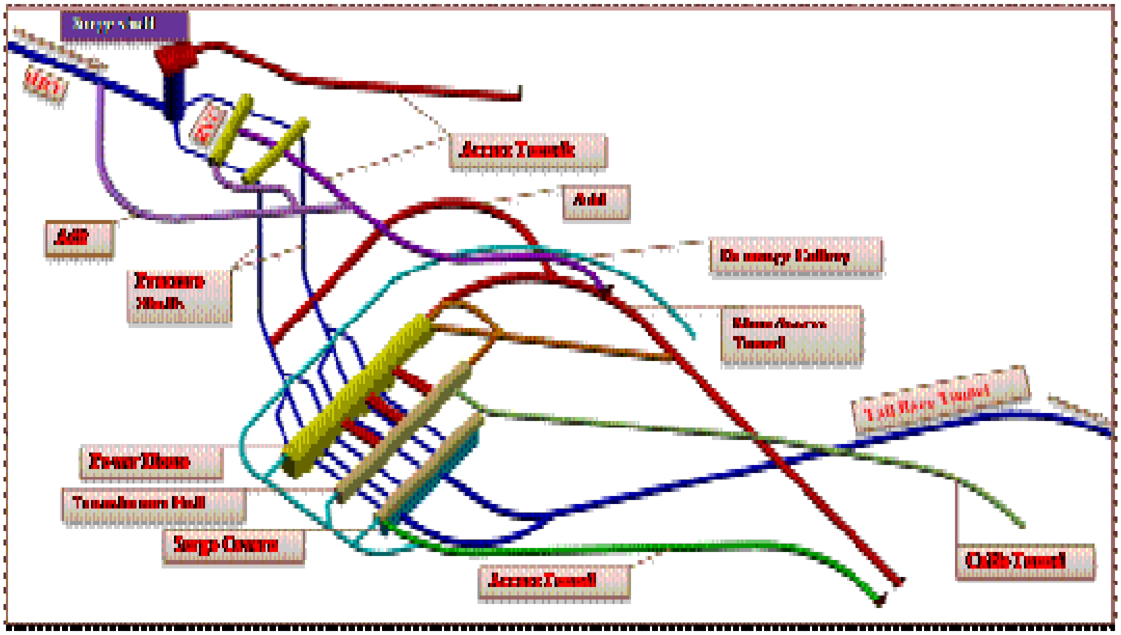
PLANNING, ANALYSIS AND DESIGN
OF
UNDERGROUND STRUCTURES

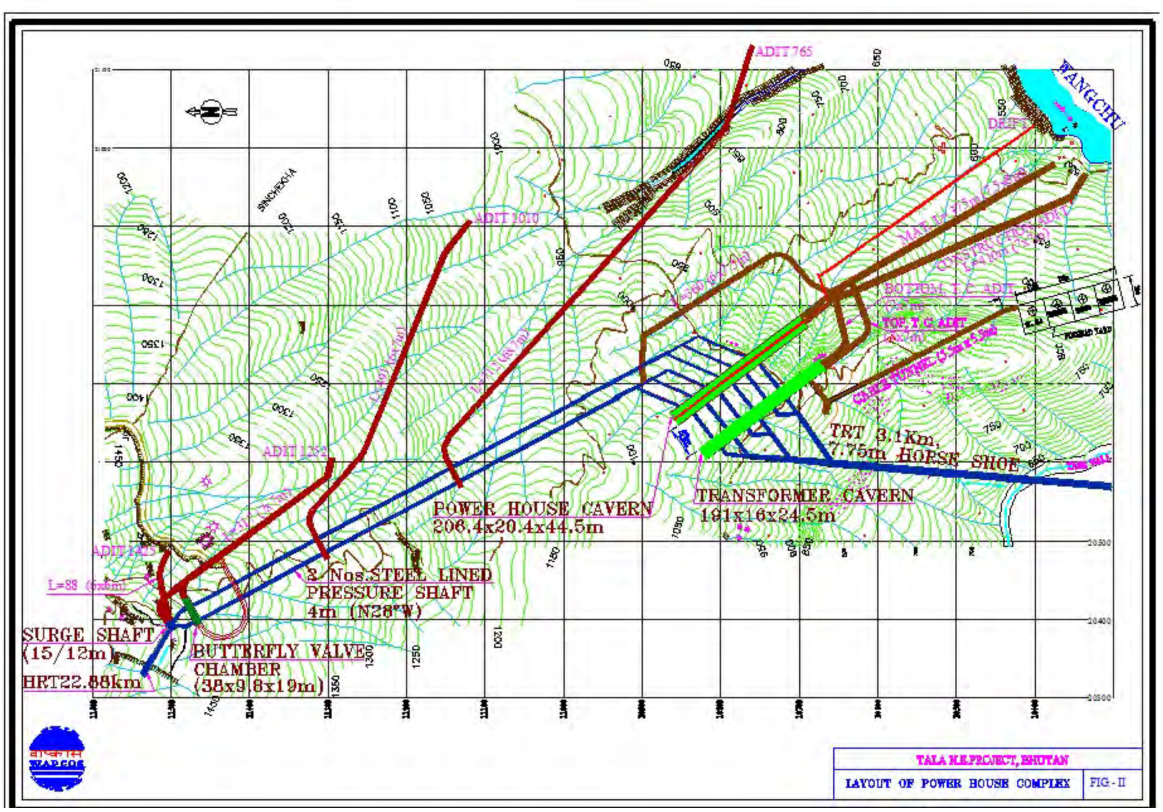
(FOR HYDROPOWER PROJECTS)

1.0 INTRODUCTION

Hydropower is a renewable, economic, non-polluting and environment friendly source of energy. Hydropower Projects require various underground structures such as tunnels, shafts, caverns etc. which are used for different purposes. Depending on the type of scheme, the projects could be fully underground or partially underground. The world's longest power tunnel is the Head race Tunnel for Nathpa Jhakri H.E,Project in Himachal Pradesh, which is 10.15m dia and 27.5km long. In Tala H.E.Project, in Bhutan the Head race Tunnel is 6.8m dia and 23km long. In India, the widest underground cavern has been made for Srisailam Left Bank Power House which is 25.7m wide x 236m long x 52m high. The rock mass in which the underground structures to be constructed is highly anisotropic and discontinuum. Various geological and geotechnical parameters influence the behaviour of the underground openings and each design should be carried out considering the site specific situations. The complex problem of underground structure design is tackled by utilizing the expertise of various disciplines such as, engineering geology, geotechnical engineering, structural engineering etc. In this lecture, the analysis and design of underground structures such as tunnels, shafts, caverns etc. which are commonly required in hydroelectric power projects are explained.

2.0 TYPICAL LAYOUT OF SOME HYDROELECTRIC POWER PROJECTS





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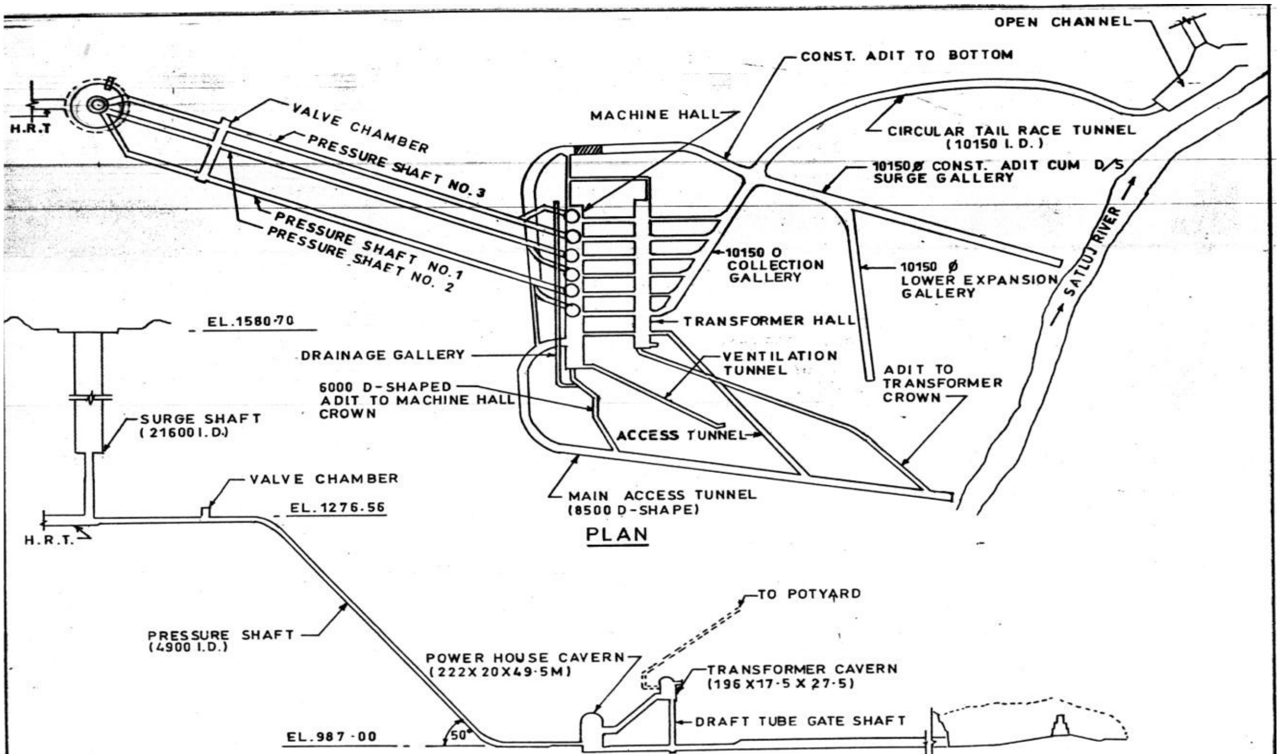
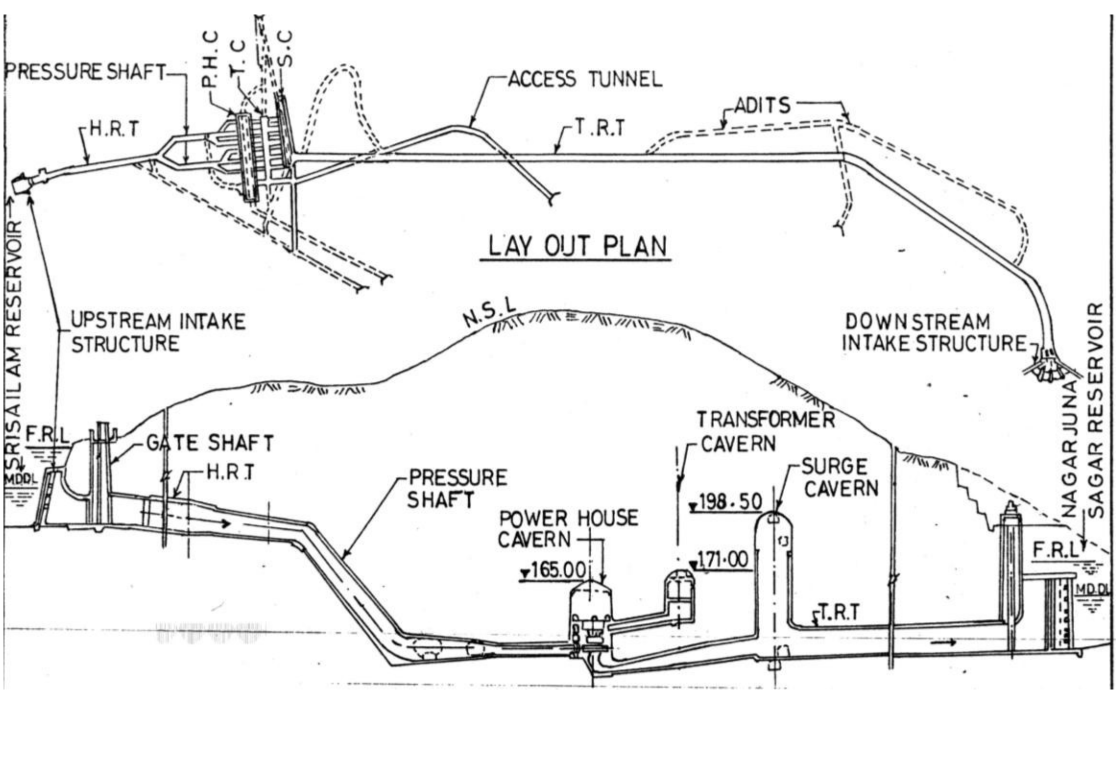


TYPICAL LONGITUDINAL PROFILE ALONG WATER CONDUCTOR SYSTEM

NATHPA JHAKRI HYDRO-ELECTRIC PROJECT (1500MW)



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3.1 COMPONENTS IN A TYPICAL UNDERGROUND HYDROPOWER PROJECT

- Diversion dam
- **Diversion tunnel**
- Intake structure
- **Underground sedimentation basin**
- **Headrace tunnel**
- **Headrace surge shaft**
- **Pressure shaft**
- **Caverns for powerhouse, transformers etc.**
- **Tailrace tunnel**
- **Tailrace surge shaft**
- Outfall structure
- Switchyard
- Network of interconnecting tunnels and adits

3.2 TUNNELS

- These are underground passages made without removing the overlying rock.
- Constitutes one of the most important and challenging component, particularly in the run-of-river schemes in the Himalayas
- Tunnels can either have pressure flow or free flow or both

Shapes of the tunnels

- **Circular section**
- **D-Shaped section**
- **Horse shoe section**
- **Modified horse shoe section**

The particular cross section to be adopted depends on the following factors:

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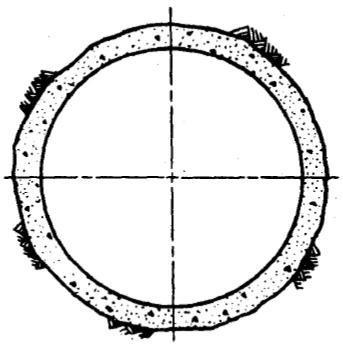
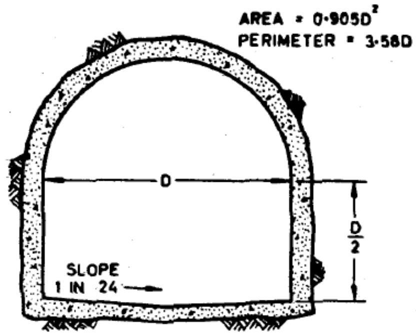
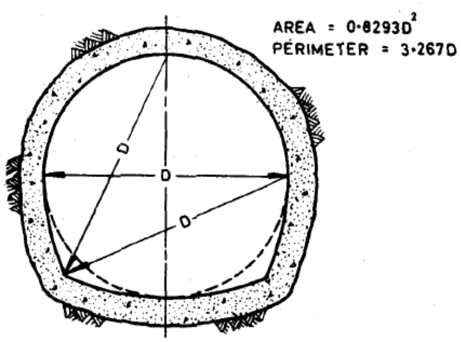
- Geologic conditions along the alignment
- Hydraulic requirements
- Structural considerations and
- Functional requirements



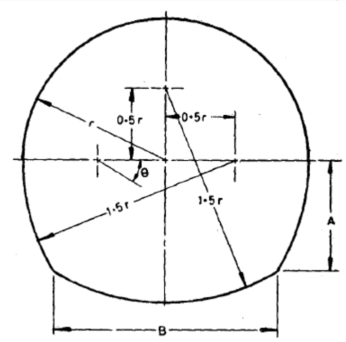
CIRCULAR



D-SHAPED



HORSE -SHOE



$r = 0{\cdot}987\ 580\ R$

where

R = Radius of Hydraulically Equivalent Circle

Area of Section	$= 3{\cdot}253\ 572\ r^2$
Perimeter of Section	$= 6{\cdot}426\ 334\ r$
Hydraulic Radius	$= 0{\cdot}506\ 287\ r$
A	$= 0{\cdot}780\ 776\ r$
B	$= 1{\cdot}561\ 553\ r$
θ	$= 31° 22' 01''$

MODIFIED HORSE -SHOE

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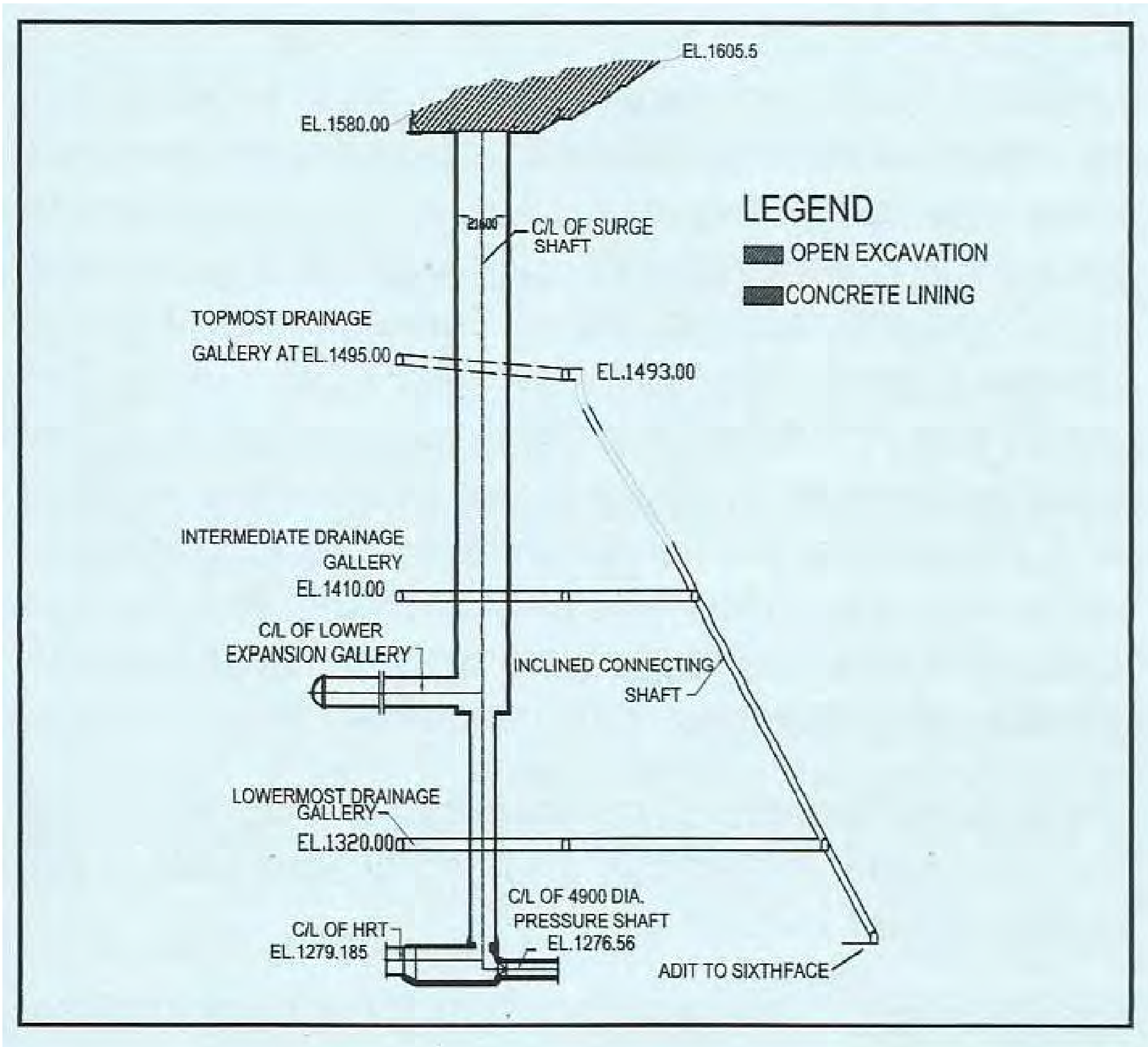
3.2 SHAFTS


EL.1605.5
EL.1580.00
21600
C/L OF SURGE SHAFT
LEGEND
OPEN EXCAVATION
CONCRETE LINING
TOPMOST DRAINAGE GALLERY AT EL.1495.00
EL.1493.00
INTERMEDIATE DRAINAGE GALLERY
EL.1410.00
C/L OF LOWER EXPANSION GALLERY
INCLINED CONNECTING SHAFT
LOWERMOST DRAINAGE GALLERY
EL.1320.00
C/L OF 4900 DIA. PRESSURE SHAFT
EL.1276.56
C/L OF HRT
EL.1279.185
ADIT TO SIXTHFACE

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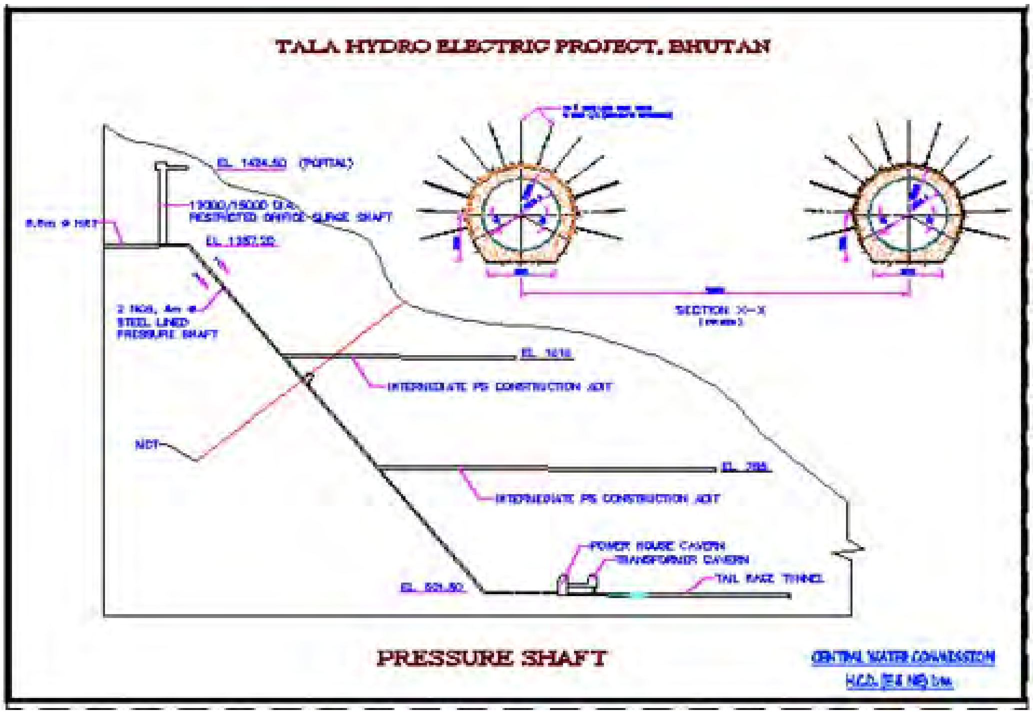
TALA HYDRO ELECTRIC PROJECT, BHUTAN
NCT
INTERMEDIATE PS CONSTRUCTION ADIT
INTERMEDIATE PS CONSTRUCTION ADIT
POWER HOUSE CAVERN
TRANSFORMER CAVERN
TAIL RACE TUNNEL
SECTION X-X
PRESSURE SHAFT
CENTRAL WATER COMMISSION

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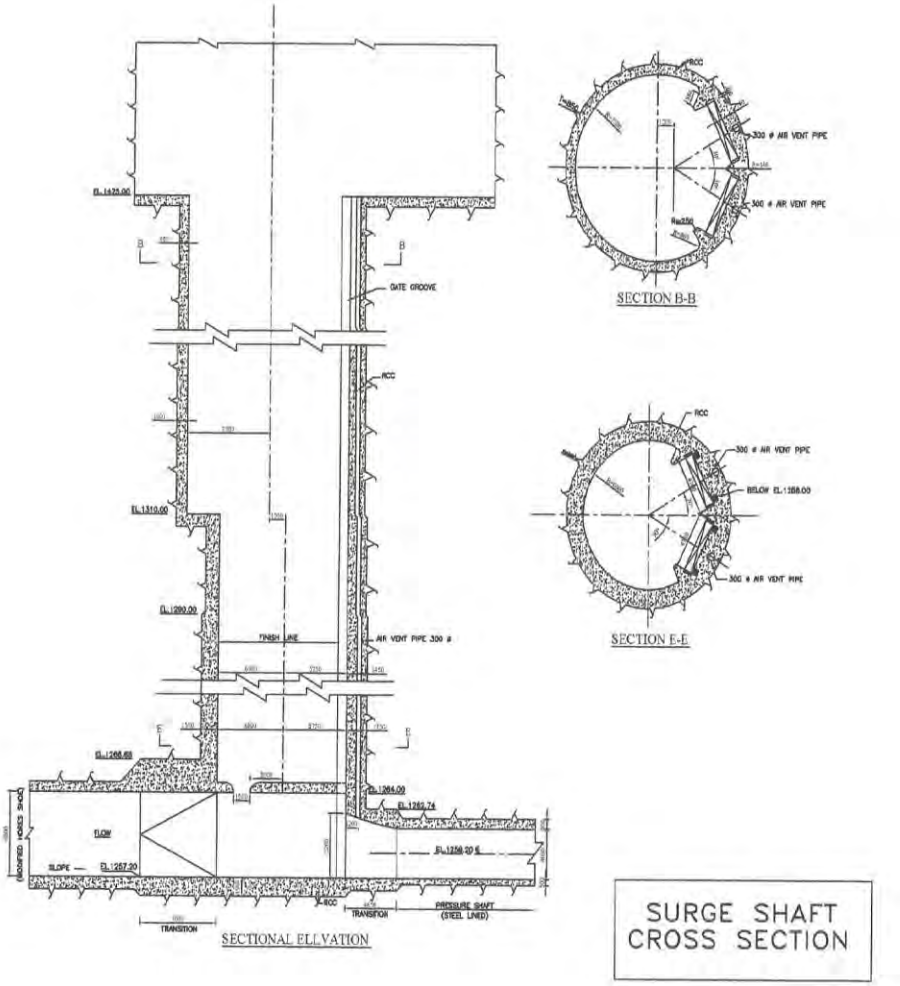

SURGE SHAFT
CROSS SECTION

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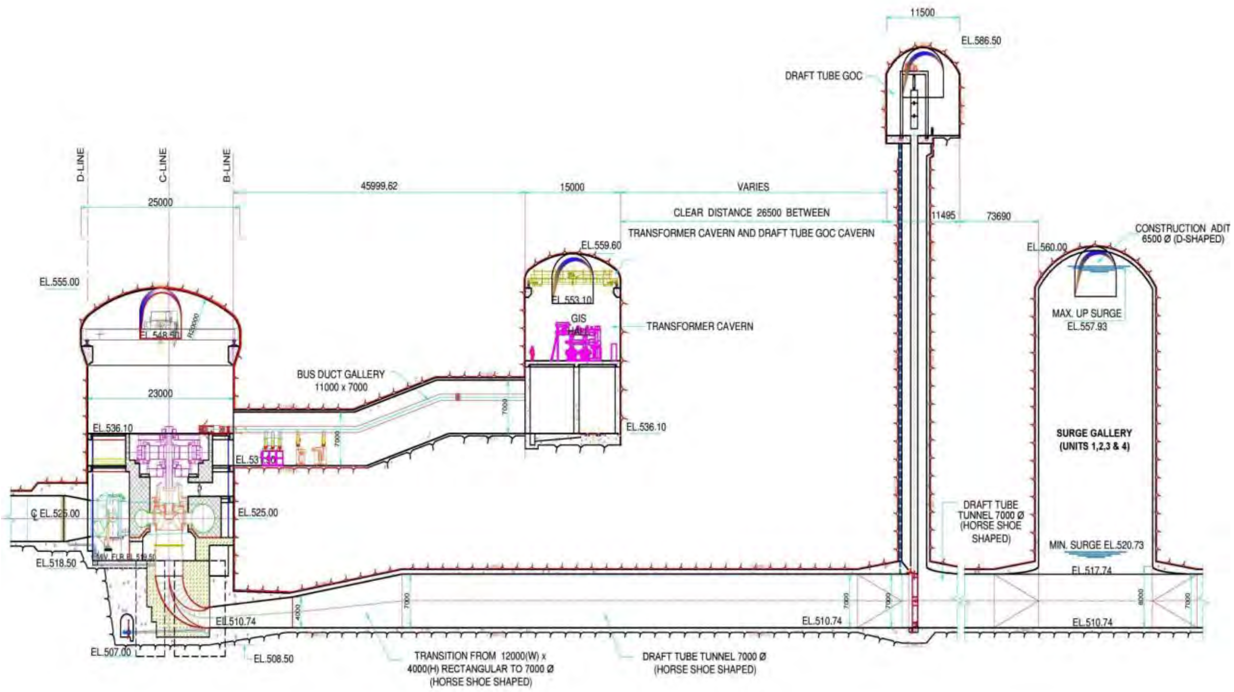
3.3 CAVERNS (POWER HOUSE , TRANSFORMER HALL ETC.)



4.1 UNDERGROUND STRUCTURE – FAILURE MECHANISMS

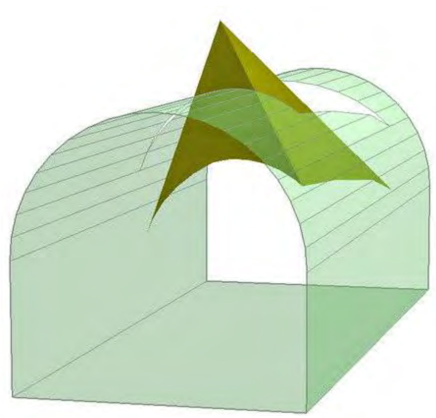
Due to creation of an underground opening, readjustment of the pre-existing state of stress and displacements takes place and unless adequate rock support is not provided, the structure could fail.

For an underground opening, the failure can occur mainly in two ways. These are :

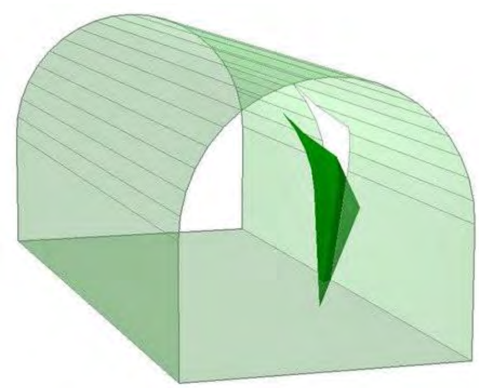
I. Structurally- controlled and / or
II. Stress- controlled

4.2 Structurally- controlled Failure.

- *jointed rock masses at relatively shallow depth.*
- *wedges falling from the roof or sliding out of the sidewalls of the openings.*
- *wedges formed by intersecting structural features, such as bedding planes and joints.*
- *Unless steps are taken to support these loose wedges, the stability of the opening may deteriorate rapidly and the structure could fail.*
- *unidirectional body force ie., gravity.*



ROOF WEDGE



SIDE WALL WEDGE

The design approach consists of the following:

- Determination of average dip and dip direction of significant discontinuity sets.
- Identification of potential wedges which can slide or fall from the back or walls.
- Calculation of the factor of safety of these wedges, depending upon the mode of failure.
- Calculation of the amount of reinforcement required to bring the factor of safety of individual wedges up to an acceptable level.

`

4.3 Stress-controlled failure.

- *Excavation at more depth.*
- *High stresses.*
- *Rock mass is relatively homogeneous and isotropic.*
- *The failure is essentially stress related.*

5.1 STRESS CONTROLLED INSTABILITY

By assuming the rock mass to be Continuous, Homogeneous, Isotropic and Linear Elastic material, stress and displacements can be computed for simple excavation shapes such as circle , elliptical etc.

In-situ stress

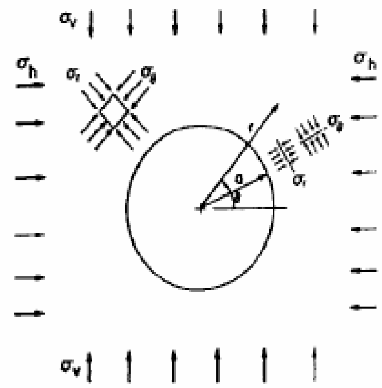
The pre existing state of stress in the rock mass before any excavation is carried out is called the in-situ stress.

The magnitude and directions of in situ stress can be determined by carrying out in - situ stress measurements. The common methods are;

- Flat jack technique
- Overcoring technique
- Hydro-fracturing method

Closed form solutions for induced stress (*which are produced due to the disturbance caused by the excavation)* are available for simple shapes such as circle, elliptical etc. For complex 3-dimensional excavation geometry, the induced stresses can be computed by Numerical Methods.

Box 8-1. Stresses Around a Circular Opening In a Biaxial Stress Field



Notation used to describe stresses around a circular opening in a biaxial stress field

a	radius of tunnel shaft
r	radial distance to any point
Θ	angular distance to any point
σ_h, σ_v	original (pre-tunneling) stress field at the tunnel level
σ_θ, σ_r	final (post tunneling) radial and tangential stresses around the tunnel
E	is Young's Modulus of the rock
υ	is the Poisson's Ratio
u_a	is the radial displacement at radius a
v_a	is the tangential displacement at radius a

The stresses are:

radial stress $\sigma_r = 0.5(\sigma_v + \sigma_h)(1 - a^2/r^2) + 0.5\,(\sigma_v - \sigma_h)(1 + 3a^4/r^4 - 4a^2/r^2)\cos 2\Theta$

circumferential stress $\sigma_\theta = 0.5(\sigma_v + \sigma_h)(1 + a^2/r^2) - 0.5\,(\sigma_v - \sigma_h)\,(1 + 3a^4/r^4)\cos 2\Theta$

shear stress $\tau_{r\theta} = 0.5(\sigma_h - \sigma_v)(1 - 3a^4/r^4 + 2a^2/r^2)\sin 2\Theta$

At the boundary of excavation, **r = a,**

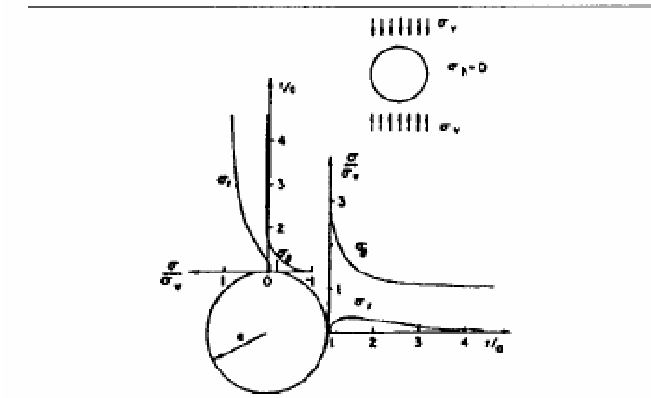
The radial stress, $\sigma_r = 0$,

The circumferential stress, $\sigma_\theta = \sigma_v\,[(1+k) - 2(1-k)\cos 2\theta]$

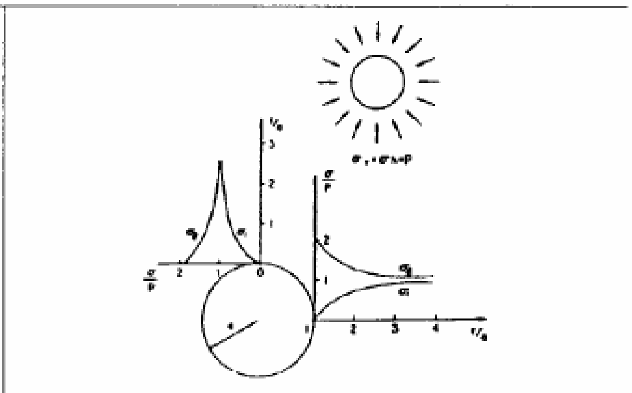
The shear stress , $\tau_{r\theta} = 0$

where , $\sigma_h = k \cdot \sigma_v$

For the case when the horizontal stress is zero ie., $\sigma_h = 0$ and hydrostatic stress field ie., $\sigma_h = \sigma_v$, the radial and tangential stress distribution are shown below.

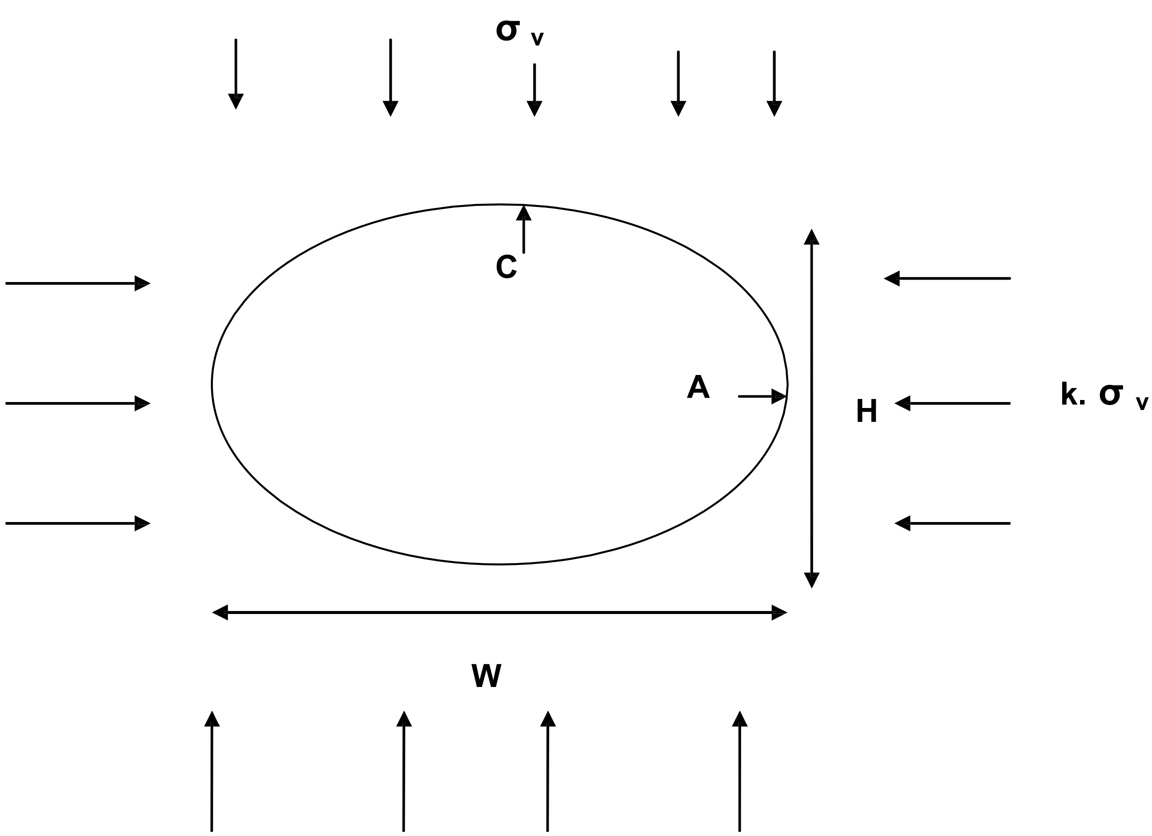


Radial stress (σ_r) and tangential stress (σ_θ) along the vertical and horizontal axes of a circular tunnel (shaft) in a uniaxial stress field (σ_V).



Radial stress (σ_r) and tangential stress (σ_θ) around a circular tunnel (shaft) in a hydrostatic stress field (P).

5.2 Elliptical Excavations- stress and displacements



The tangential boundary stresses are given by the following equations:

$$\sigma_A = \sigma_v \left[1 + \sqrt{\frac{2.W}{\rho A}} - k\right]$$

$$\sigma_C = \sigma_v \left[k\left(1 + \sqrt{\frac{2.H}{\rho C}}\right) - 1\right]$$

where σ_A and σ_C are the tangential boundary stresses at location A & C.

ρA and ρC are radii of curvature at A & C.

5.3 Design principles from the above closed form analysis:

- Critical stress concentrations increase as radius of curvature of the boundary decreases. Hence openings with sharp corners should be avoided.
- The optimum shape of opening in a hydrostatic stress field (k=1) is circle.
- Boundary stresses in an elliptical opening can be reduced to a maximum if the axis ratio of the opening can be matched to the ratio between the in-situ stresses.
- When the value of k is very low, tensile stresses occur on the boundaries of all excavation shapes. These tensile stresses will become compressive as the value of k increases above (*approx*) k=1/3.

5.4 Stresses around multiple openings.

The creation of an underground opening forces the native stresses and displacements to readjust. Based on simple elastic theory, it can be seen that this zone of readjustment can include points that are located upto 5 times the diametrical distance from the centre of the opening. Thus if another tunnel is located such that the pillar thickness equals 9 (R1 + R2), where R1 & R2 are the radii of the two adjacent tunnels, then the readjustment of the stresses and displacements will be independent of the second tunnel. In actuality, rock is not that homogeneous and as such elastic theory may not be fully applicable. In practice, if the pillar thickness equals the largest adjacent tunnel, the multiple openings behaves as a single opening and the stability of the intervening pillar should be analysed in detail.

6.1 NUMRICAL METHODS FOR STRESS ANLYSIS.

- Used for complicated non uniform or non geometric shapes, tunnel intersections, bifurcation, stacked tunnels, power house caverns etc.
- Accommodates different material properties including joints, shear zones

& faults.

- Linear or non- linear behaviour
- Time dependent behaviour

Input Parameters for numerical analysis.

- IN-SITU STRESS – vertical / horizontal
- ROCK MASS
  - modulus of deformation
  - Poisson's ratio
  - uniaxial compressive strength
  - density
- mi,mb,s,a - for Hoek & Brown failure criteria
- c , Ø - for Mohr- Coloumb failure criteria
- JOINT PARAMETERS - c , Ø , normal & shear stiffness
- SUPPORT PARAMETERS – for rock bolt , shotcrete etc.

Commonly used software for analysis / design of underground structures:

- **EXAMINE 2D / 3D** , **PHASE** ^{2} - B.E METHOD
- **FLAC 3D** **–** FINITE DIFFERENCE METHOD
- **UDEC / 3-DEC** - DISTINCT ELEMENT METHOD

7.1 FAILURE CRITERIA

7.2 Hoek – Brown failure criteria:

The Hoek – Brown failure criteria is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a$$

where σ_1' and σ_3' are the maximum and minimum effective stresses at failure,
m_b is the value of the Hoek-Brown constant m for the rock mass,
s and a are constants which depend upon the rock mass characteristics, and
σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

7.3 Mohr-Coloumb failure criteria:

$$\sigma 1f = \sigma 3 \tan^{2} (45+Ø/2) + 2 c \tan$$

$$(45+Ø/2) \text{ FACTOR OF SAFETY} = \sigma 1f / \sigma 1$$

7.4 Importance of selection of failure criteria:

The Hoek- Brown failure criterion, which assumes isotropic rock and rock mass behaviour, should only be applied to those rock masses in which there are sufficient number of closely spaced discontinuities. When the structure being analysed is large and the block size is small in comparison, the rock mass can be treated as a Hoek-Brown material.

Where the block size is of the same order as that of the structure being analysed or when one of the discontinuity sets is significantly weaker than the others, Hoek - Brown criterion should not be used. In these cases the stability of the structure should be analysed by considering failure mechanisms involving the sliding or rotation of blocks and wedges defined by intersecting structural features.

8.1 DESIGN OF UNDERGROUND STRUCTURES:

The various steps involved in the design of an underground opening are:

- **Planning of Layout**
- **Shape / Size of Opening:**

  Circular, Modified Horse Shoe, Horse Shoe, D-Shape Etc.
- **Design of Rock Support System:**

  Rock Bolts / Rock Anchors / Cable Anchors

  Sohtcrete (Plain / Wiremesh / Steel Fibre Reinforced)

  Steel Ribs Etc.
- **Design of Lining, if Required:**

  Steel

  Concrete (Plain / Reinforced)

`

8.2 PLANNING OF LAYOUTS OF TUNNELS

For planning the layout of tunnels various factors like geology, minimum rock cover above the roof of the tunnel, easy accessibility by provision of construction adits etc. will have to be considered. After dam and the power house locations are finalized, the layout of tunnels are planned such that the length is the least. Depending upon the type and strength of rockmass, the rock cover above the tunnel should preferably be restricted to such a depth that squeezing rock condition should not occur at any stretch of tunnel. For this, we may have to keep optimum rock cover. Minimum slope for proper natural drainage of water is also to be considered.

The location and length of construction adits will have to be carefully selected and laid out. The approximate length of main tunnel from each face of excavation should be kept to a maximum of 3000 m. As this length exceeds 3000m, the cycle time for excavation increases very much. Ensuring proper ventilation of farther end of tunnel also becomes quite difficult. For the same consideration the adit length also should be kept to a minimum. The adit should preferably have down slope from the tunnel so that the drainage of water from the tunnel and adit can take place by gravity..Location of portal for adits and tunnels will have to be done in consultation with the geologist. Volume of open excavation and overburden at site of portal should be minimum. Rock cover of minimum 2 x diameter of tunnel is necessary at the portal face of tunnel / adit. The rock mass on the sides and above the portal should be stable or it should be possible to stabilize the slopes by suitable treatment by using rock bolts and shotcreting etc.

The minimum rock cover above the roof of the tunnel along its length should be 3 times the diameter of the tunnel. Overburden is not considered as rock cover. The rock cover actually required can be designed according to the internal pressure of the tunnel at different sections. The lining of water tunnels is usually done with concrete either plain or RCC and sometimes steel depeding on the design requirement. Depending on the rockmass conditions a portion of internal pressure of water can be transferred to rock and the lining is designed for the balance internal pressure. The portion of internal pressure taken by rock is called 'rock participation'.

8.3 PLANNING OF CAVERNS (POWER HOUSE , TRANSFORMER HALL ETC.)

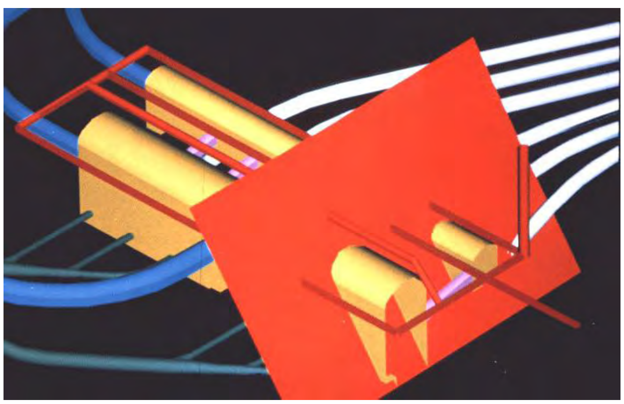
The planning of underground caverns is made considering the following geological & geotechnical data:

- Faults
- Shear zones
- Discontinuities
- Joint pattern, spacing, roughness etc.
- In-situ Data
    - In-situ stress
    - Pore water pressure
- Deformation modulus

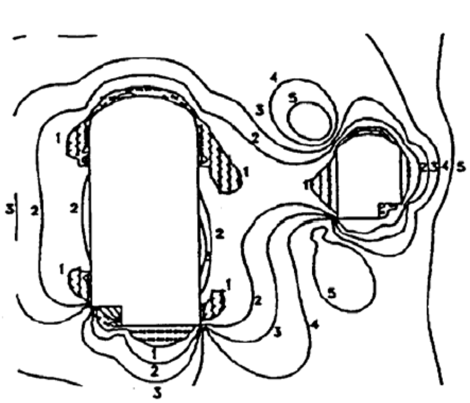
Based on the above, the following are decided:

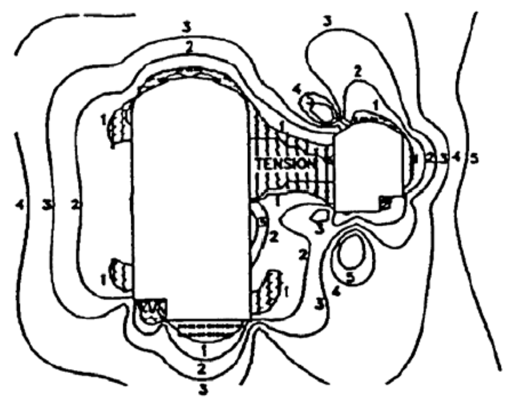
- Deciding layout of power house cavern
- Deciding orientation
- Support system

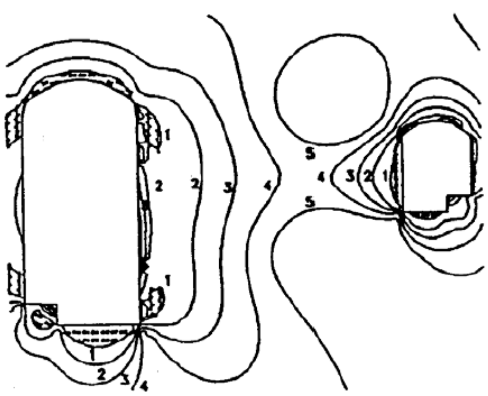
The orientation of caverns is done from the consideration of (1) structural discontinuities like joint sets and (2) in-situ stress. The axis of the cavern is placed perpendicular to the strike of major joint set if structurally controlled failure is expected. In case stress induced failures are of major concern, as in the case of deep seated caverns, the axis of the cavern is oriented along the direction of major principal in-situ stress.



In underground Hydro projects, usually the transformers are placed in a smaller cavern parallel to the main cavern. This has the advantage of reducing the size of main cavern and isolating the transformers in case of fire. The Electrical designers prefer placement of both caverns as close as possible to reduce the cost of bus bar which connect between the generators and the transformers. However placing the two caverns close together leads to unfavourable stress conditions in the pillar between the two caverns. By carrying out number of studies, it has been suggested that for weak rock masses the pillar **should not be less than the height of the larger cavern** and wherever possible it should be slightly greater. In very poor rock masses, in which the overstressed zones are larger, **the pillar thickness should be 1.5 times the height of the larger cavern.** These multiple caverns should be subjected to numerical analysis for confirming the adequacy of the design.







METHODS OF SUPPORT DESIGN

The various methods of support design for tunnels / underground openings can be grouped under :

- **Empirical Methods**
- **Analytical Methods**
- **Graphical Methods**
- **Observational Methods**

Empirical Methods

- The empirical approach for the design of support system relates the experience gained on rock condition and support requirements at previous projects to conditions anticipated at proposed site.

- The empirical approach makes use of statistical analysis of observations.

Rock mass classification which are based on such co-relation are most commonly used for estimation of rock loads and requirement of support system

Empirical Approach

The Rock Mass Classification Systems could be of the following two types:

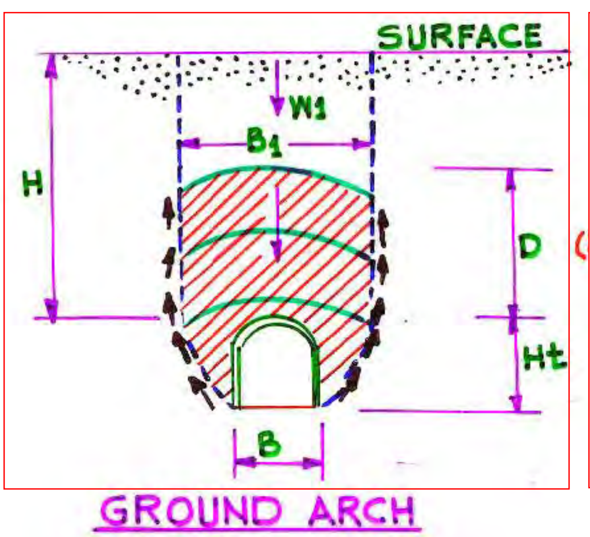
a) QUALITATIVE:

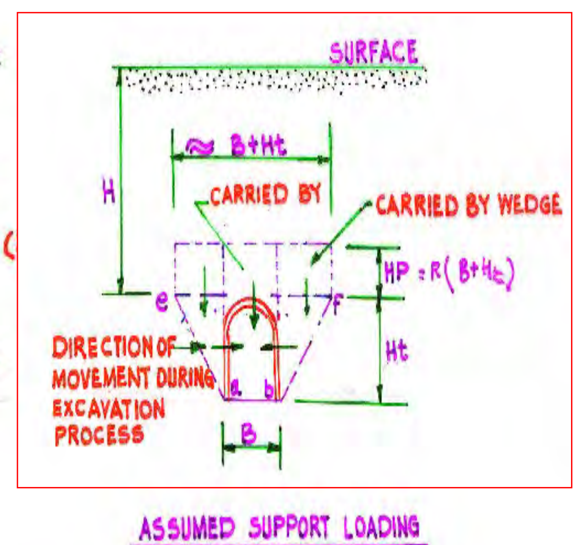
Terzaghi's Rock Load

b) QUANTITATIVE:

- DEERE'S R.Q.D.
- C.S.I.R. (BIENIAWSKI)**RMR**
- N.G.I (BARTON) **"Q"**

Terzaghi's Rock Load Concept:





GROUND ARCH

ASSUMED SUPPORT LOADING

Terzaghi's Rock Load Classification Table:

S.No.	Rock Condition	Rock Load factor Hp	Remarks
1	HARD AND INTACT	ZERO	light lining reqd. only if spalling or poping occur.
2	HARD STRATIFIED OR SCHISTOSEC	0 TO 0.5 B	light support load may change erratically from point to point
3	MASSIVE MODERATELY JOINTED	0 TO 0.25 B	
4	MODERATELY BLOCKY AND SEAMY	0.25B TO 0.35 (B+Ht)	no side pressure
5	VERY BLOCKY AND SEAMY	0.35 TO 1.10 (B+Ht)	little or no side pressure
6	COMPLETELY CRUSHED BUT CHEMICALLY INTACT	1.10 (B+Ht)	considerable side pressure softening effect of seepage towards bottom of tunnel reqs. either continuous support for lower ends of ribs or circular ribs
7	SQUEEZING ROCK	1.10 TO 2.10 (B+Ht)	heavy side pressure invert struts reqd. circular ribs are recommended
8	SQUEEZING ROCK GREAT DEPTH	2.10 TO 4.50 (B+Ht)	
9	SWELLING ROCK	UPTO 250 FT. IRRESPECTIVE OF VALUE OF (B+Ht)	circular ribs reqd. in extreme cases use yielding support

- Terzaghi's method though most commonly used , tends to be subjective
- The geotechnical engineer has to use a considerable amount of judgement while interpreting rock outcrops or borings
- It is left to the judgement of the user as to how he interprets a particular rock type and interpretation may differ from person to person

BIENIAWSKI (C.S.I.R.) RMR METHOD

The following six parameters are used to classify a rock mass using the *RMR* system:

- **Uniaxial compressive strength of rock material.**
- **Rock Quality Designation (*RQD*).**
- **Spacing of discontinuities.**
- **Condition of discontinuities.**
- **Groundwater conditions.**
- **Orientation of discontinuities.**

GUIDELINES FOR EXCAVATION AND SUPPORT OF TUNNELS

(Size 10M and Construction by drill & Blast)

Rock mass Class	**Excavation**	**Rock bolts (20mm diameter, fully**	**Shotcrete**	**Steel Sets**
I – Very Good Rock	Full face, 3 m advance.	**Generally no support required except spot bolting**		
II – Good rock RMR: 61-80	Full face,1-1.5m advance, Complete support 20m from	Locally, bolts in crown 3 m long, spaced 2.5	50 mm in crown where	**None.**
III – Fair rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each	Systematic bolts 4m Long, spaced 1.5-2 m In crown and walls	50-100 mm in crown and 30 mm in	**None.**
IV – Poor rock RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support	Systematic bolts 4-5 m Long, spaced 1-1.5 m In crown and walls	100-150 mm in crown and 100 mm in	Light to medium ribs spaced 1.5 m where required.
V – Very poor Rock RMR : < 20	Multiple drifts 0.5-1.5 m Advance in top heading. Install support concurrently	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls	150-200 mm in crown, 150 mm in	Medium to heavy ribs spaced 0.75m with steel lagging

BARTON'S (N.G.I.) "Q" SYSTEM

Barton prepared an index for tunnelling quality of a rock mass and relate this rock mass quality "Q" to six parameters.

- **RQD**
- Number of joint sets **Jn**
- Joint roughness **Jr**
- Degree of joint alteration **Ja**
- Water inflow **Jw**
- Stress condition **SRF**

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

The **roof** and **wall** support pressures based on "Q' system are as follows:

$$P_{roof} = \frac{2 J_n^{1/2} Q^{-1/3}}{3 J_r} \qquad P_h = \frac{2 J_n^{1/2} Q_w^{-1/3}}{3 J_r}$$

Proof = ultimate roof support pressure in Kg/cm^2
Ph = ultimate wall support pressure in Kg/cm^2, and
Qw = wall factor

Range of Q	Wall Factor Qw
> 10	5.0 Q
0.1 - 10	2.5 Q
< 0.1	1.0 Q

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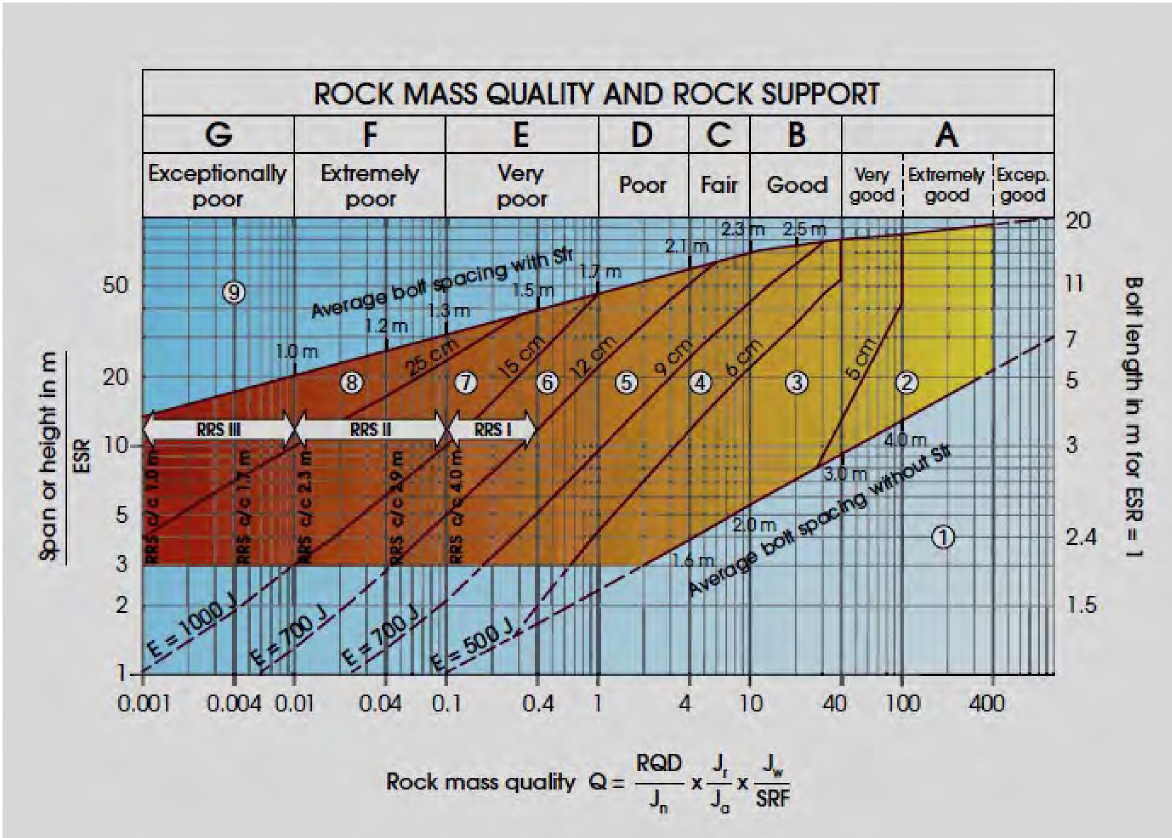


Support categories

① Unsupported or spot bolting
② Spot bolting, **SB**
③ Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B+Sfr**
④ Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500)+B**
⑤ Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700)+B**
⑥ Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E700)+RRS I +B**
⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000)+RRS II+B**
⑧ Cast concrete lining, **CCA** or **Sfr (E1000)+RRS III+B**
⑨ Special evaluation

Bolts spacing is mainly based on Ø20 mm

E = Energy absorbtion in fibre reinforced sprayed concrete

ESR = Excavation Support Ratio

Areas with dashed lines have no empirical data

RRS - spacing related to Q-value

I:
Si30/6 Ø16 - Ø20 (span 10m)
D40/6+2 Ø16-20 (span 20m)

II:
Si35/6 Ø16-20 (span 5m)
D45/6+2 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

III:
D40/6+4 Ø16-20 (span 5 m)
D55/6+4 Ø20 (span 10 m)
D70/6+6 Ø20 (span 20 m)

Si30/6 = Single layer of 6 rebars, 30 cm thickness of sprayed concrete
D = Double layer of rebars
Ø16 = Rebar diameter is 16 mm
c/c = RSS spacing, centre - centre

`

The excavation support ratio **(ESR)** for different underground excavations are as follows:

Sl.No.	Type of excavation	ESR
A.	Temporary mine openings, etc.	3 - 5
B.	Vertical shafts - circular section - rectangular / square section	 2.5 2.0
C.	Permanent mine openings, water tunnels for hydropower (except high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.	1.6
D.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels etc.	1.3
E.	Power stations, major road & railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F.	Underground nuclear power stations, railway stations, sports and public facilities, factories etc.	0.8

Commonly adopted rock support systems

- **ROCK BOLTS / ANCHORS**
- **CABLE SUPPORTS**
- **SHOTCRETE : PLAIN / MESH / FIBRE REINFOCED**
- **STEEL RIBS**

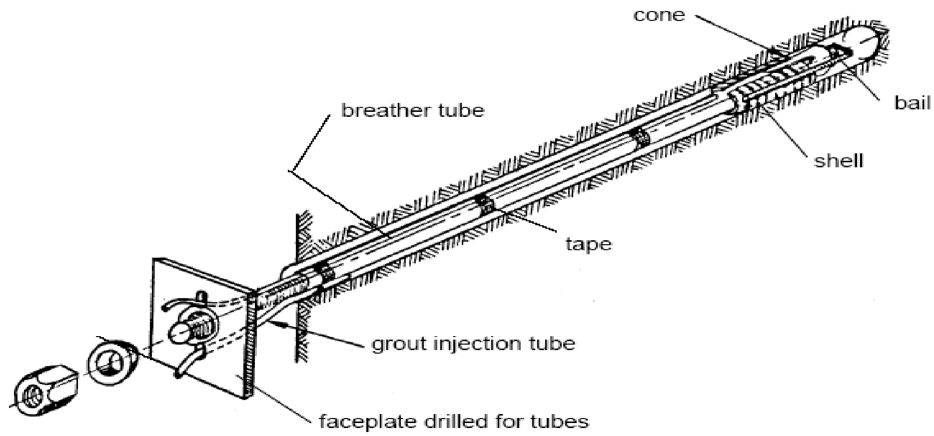
Length of rock bolt

L roof = 2 + (0.15 x SPAN/ESR)

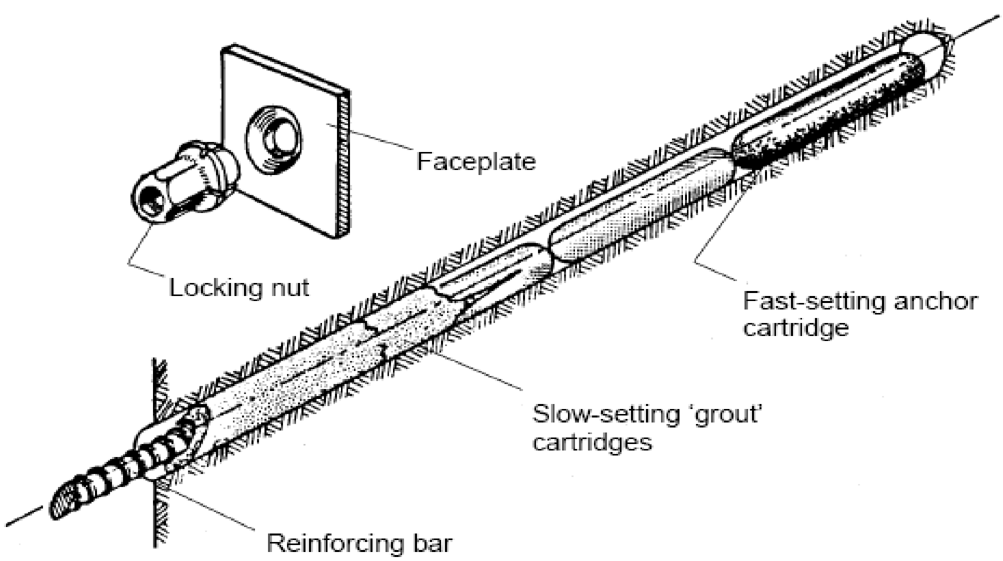
L wall = 2 + (0.15 x HEIGHT/ESR)

Spacing of rock bolts to be computed based on the support pressure.

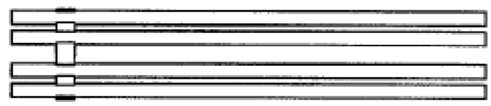
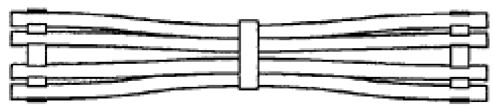
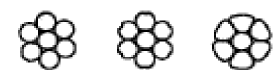
MECHANICALLY ANCHORED ROCK BOLT



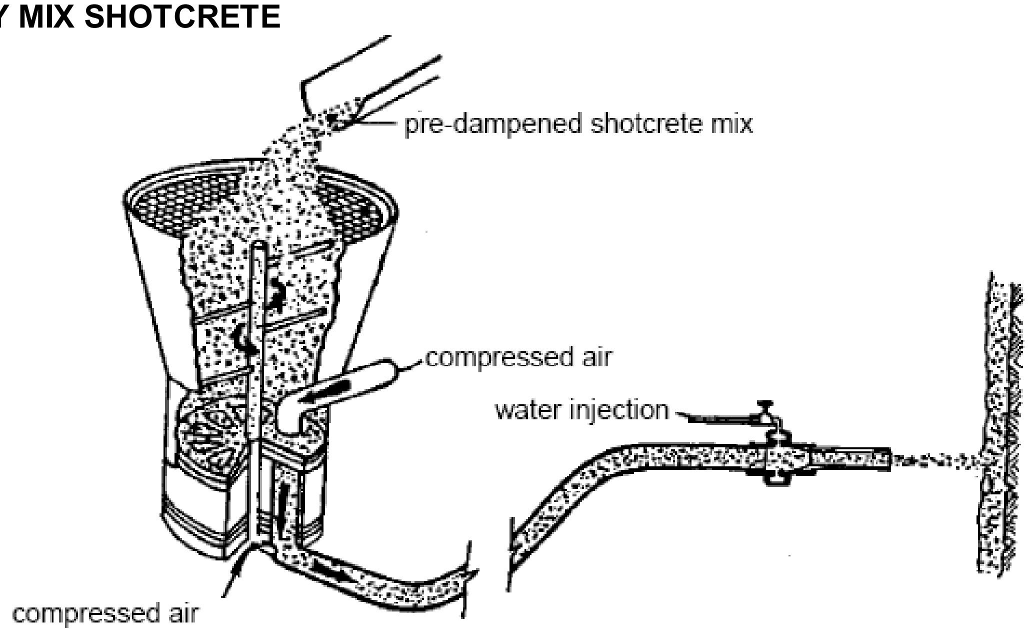
RESIN END ANCHORED ROCK BOLT



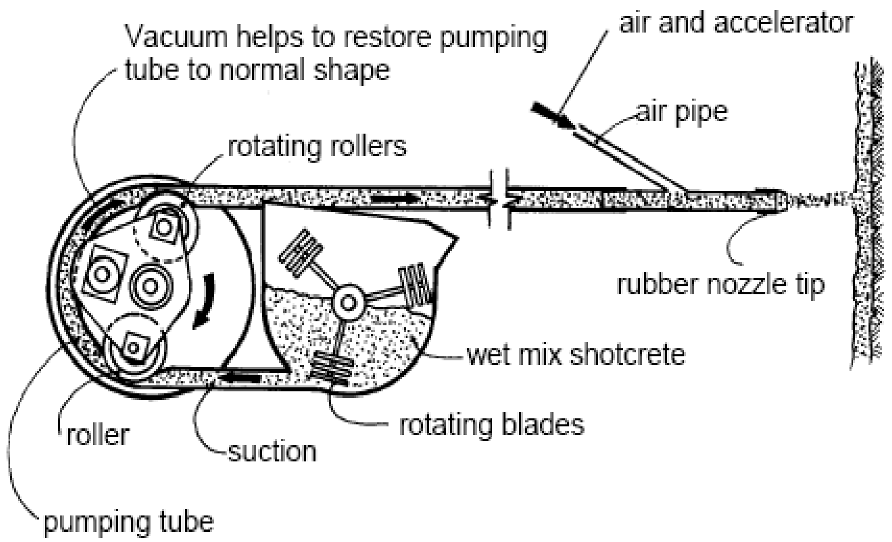
CABLE SUPPORTS

TYPE	LONGITUDINAL SECTION	CROSS SECTION
Multi-wire tendon (Clifford, 1974)		
Birdcaged multi-wire tendon (Jirovec, 1978)		 Antinode Node
Single strand (Hunt & Askew, 1977)		 Normal Indented Drawn
Coated single strand (Hunt & Askew, 1977)		 Sheathed Coated Encapsulated

DRY MIX SHOTCRETE



WET MIX SOTCRETE



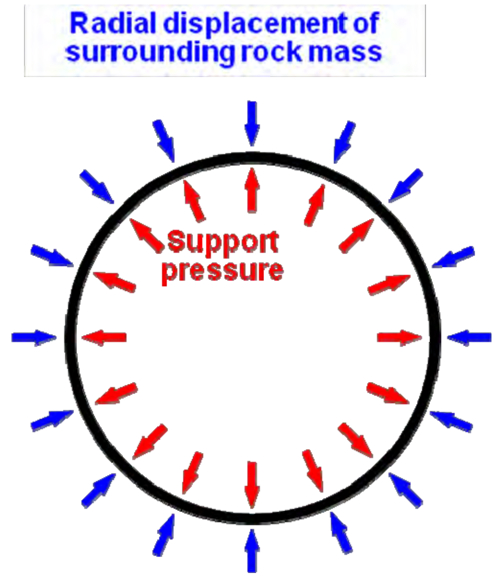
MIN. LENGTH AND MAX. SPACING FOR ROCK REINFORCEMENT

(As per U. S. Army Corps Of Engineers)

PARAMETER	EMPIRICAL RULES
minimum length	greatest of: a. two times the bolt spacing b. three times the width of critical and potentially unstable rock blocks c. for elements above the spring line : 1) spans less than 20 ft – ½ span 2) spans from 60 ft to 100 ft – ¼ span 3) spans 20 ft to 60 ft – interpolate between 10 ft and 15 ft lengths, respectively. d. for elements below the spring line: 1) for openings less than 60 ft. high – use lengths as determined in ‘c’ above 2) for openings greater than 60 ft high – 1/5 the height
maximum spacing	least of : a. 1/2 the bolt length b. 1– 1/2 the width of critical and potentially unstable rock blocks c. 6 ft
minimum spacing	3 to 4 ft

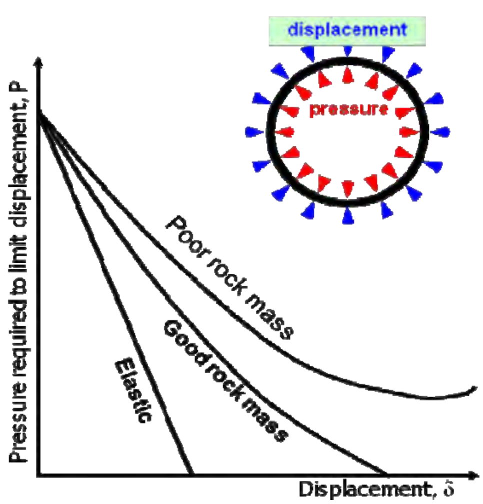
9.1 ROCK-SUPPORT INTERACTION

Rock-support interaction illustrate the interaction between the rock mass surrounding the tunnel and the support material. It is characterised by the load-deformation curve of a tunnel and available support curve of the support material.



Load-Deformation Curve

Tunnel deforms after excavation, at different rate for different rock mass quality. Support pressure required to limit the deformation changes with deformation; initial high, decreasing with further deformation. Load-deformation curve can be produced for a particular tunnel.



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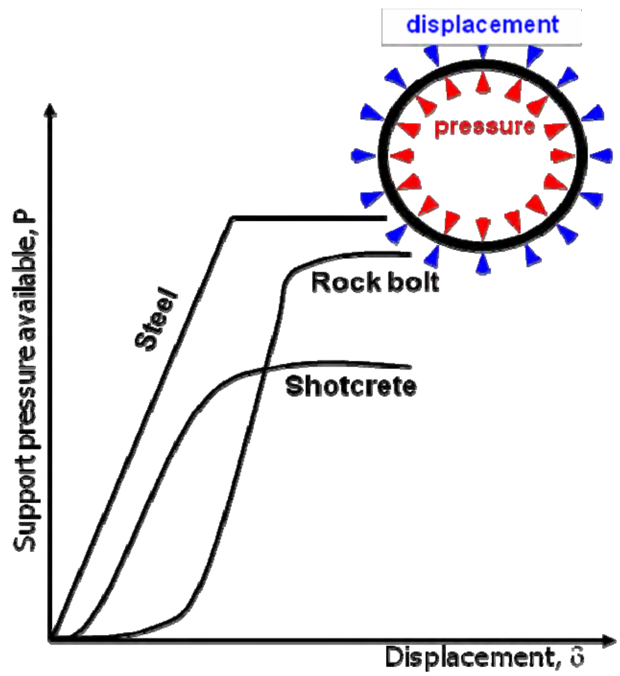
Support Pressure Curve

Available support curve is a load-deformation curve of the support material. It is a property of the reinforcement or support material, e.g., steel and concrete. In general, steel deforms elastically and after yielding, plastically.

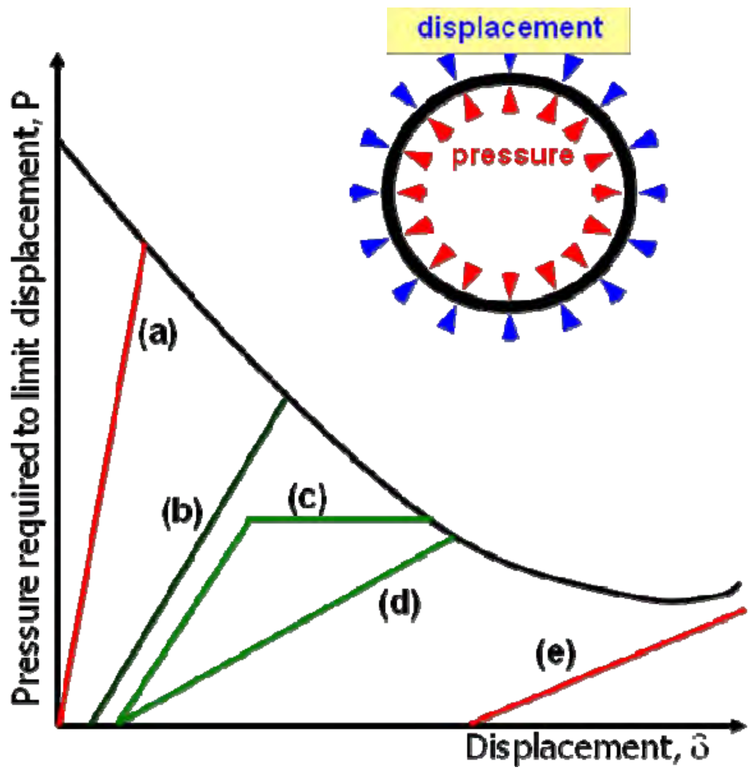


Support Interaction

Load-deformation curve and support pressure curve are analysed together. Support pressure required to limit deformation is to be provided by the available support of the support material, i.e., equilibrium.

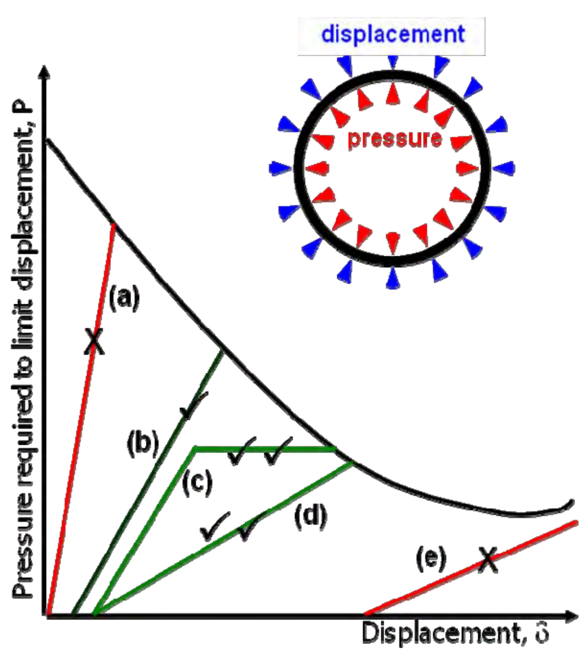
(a) stiff support; (b) medium support

(c) yielding support; (d) soft support; (e) insufficient support



Ideal Support

A good engineering practise is to allow for deformation, but to control further displacement beyond necessary. An ideal support is to match the rock load-deformation curve with the pressure yielded by the support.



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10.0 INSTRUMENTATION OF UNDERGROUND STRUCTURES

Instrumentation of underground opening is carried out for one or more of the following reasons:

- Identification of rock mass or soil properties such as strength, deformability, anisotropy and alterability.
- Identification of state of stress in rock mass.
- Measurement of loads on support, Deformation etc.
- Long term monitoring of the underground structure.

For any underground project, a well planned instrumentation during the construction is essential for monitoring the rock mass response. Also this is useful for long term monitoring after completion of the structure.

11.0 CONCLUSION

For the design of underground structures various methods are available, each having its own merits and demerits. Depending upon the importance and complexity of the problem, suitable method should be selected. Numerical methods are increasingly becoming a more popular engineering tool for underground structure design because of their capability to simulate actual rock mass conditions. However, it must be kept in mind that stability of an excavation depends on number of factors and the results of design based on the empirical methods, numerical methods etc. should be verified by sound engineering judgement and practical experience.

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DESIGN OF POWER INTAKES

POWER INTAKES

Intake structures are constructed at the inlet of Penstocks, tunnels, outlet conduits etc. to draw water from the reservoir, fore bay, canal etc. in a satisfactory manner.

1.1 Functions of Intake Structure:

An intake structure serves the following functions:

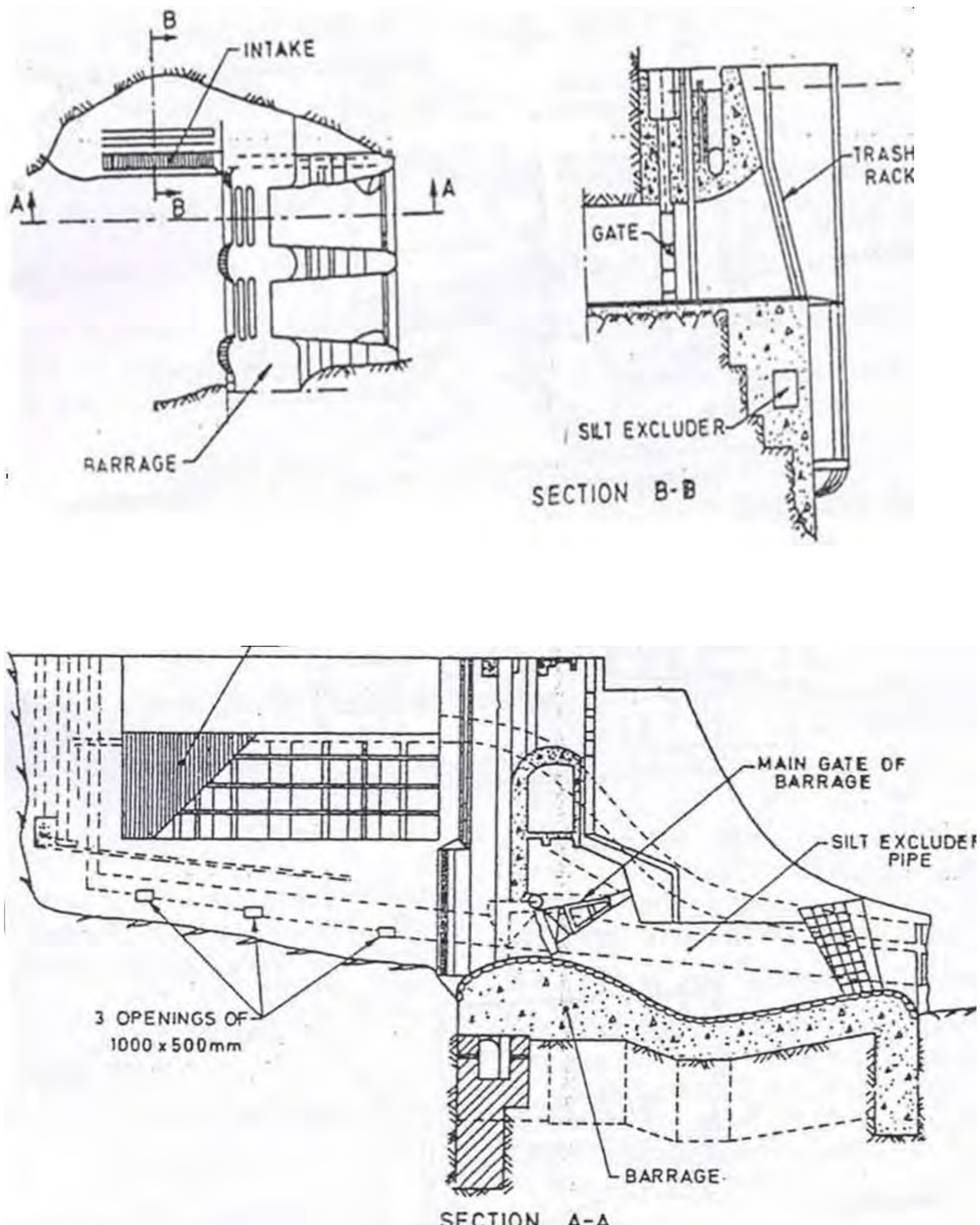
i) Prevents entry of trash, debris, ice, boulders, logs of wood etc. into the conveyance system. This is achieved by providing a trash rack at the entrance.
ii) Controls the flow of water into the conveyance system by providing a gate or a valve.
iii) Enables smooth, easy and turbulence free entry of water into the water conductor system. This is achieved by providing a bell mouth entry at the inlet mouth. This also enables to minimise the head loss at entrance.
iv) Minimises sediment entry from the river into the conveyance system. For this purpose, special devices like silt traps and silt excluders are provided.

2.1 Types of Intake Structures

Depending upon the type of power plant and its layout, intake structures may be broadly classified as,

i) Run-of-river intakes

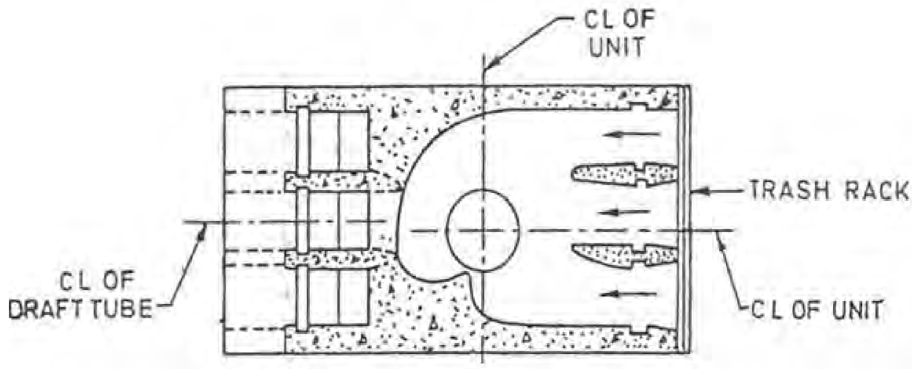
Such intakes are provided for run-of-river plants. A trash rack, made out of steel flats and sections is provided in front of the bell mouth entry to prevent entry of floating debris like grass, leaves, trees etc., and boulders etc. into the water conductor system. The control valve or gate is installed immediately after the bell mouth. A stop log is provided upstream of gate for repairs of the gate. In case of silty rivers, de-silting arrangements are also provided.

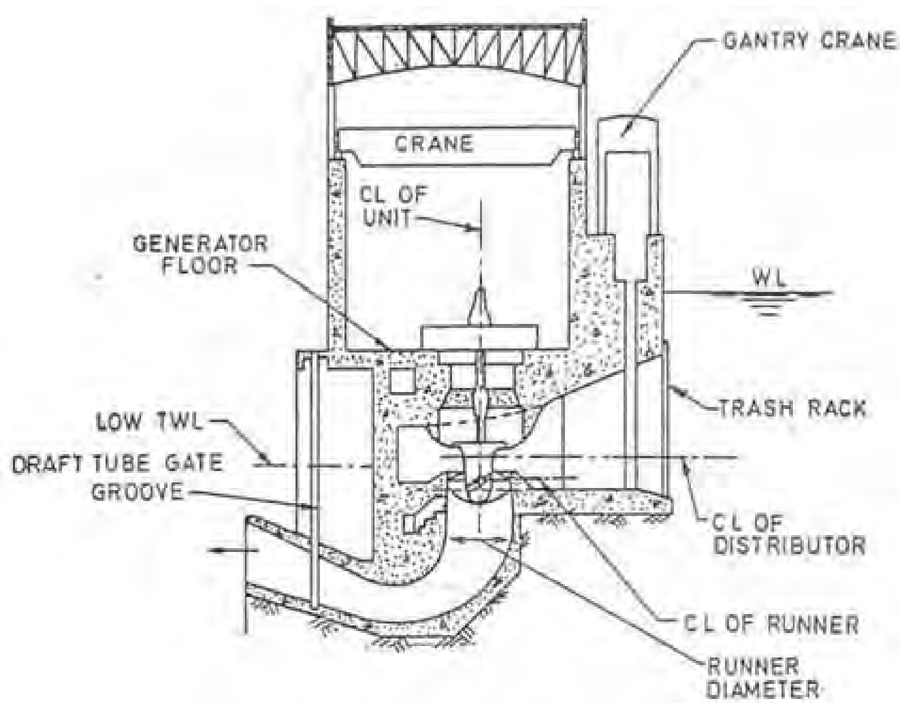


RUN-OF-RIVER INTAKES

ii) Canal Intakes

In a canal power house, generally the power house is located in a bye-pass channel taken from the main canal. Provisions like trash racks, stop logs, gates as in the case of run-of-the-river intakes are made for canal intakes also. Anoopgarh, Suratgarh and Mangrol power houses in the Indira Gandhi Canal are typical Canal Power Houses.





CANAL INTAKES

iii) Reservoir Intakes

This type of intake structure is provided for entry to penstock and tunnels taking off from a reservoir.

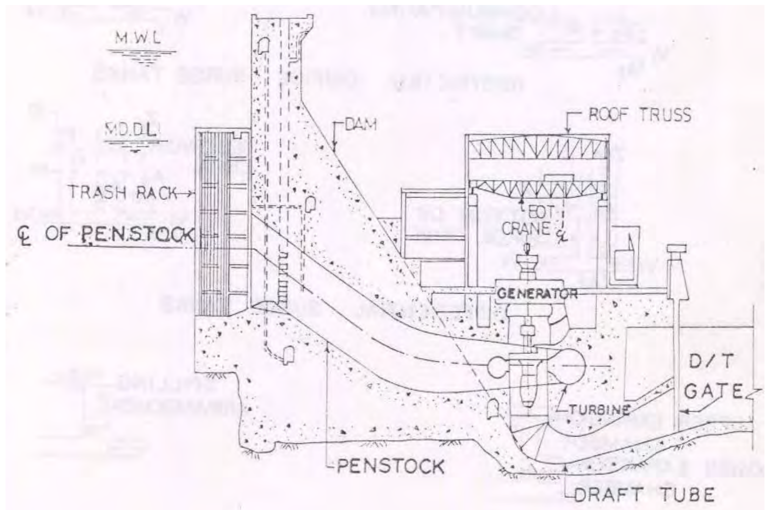
Depending upon the head above the centre line of penstock, the reservoir intakes are categroised as,

a) *Low head,* if head is upto 15 m.
b) *Medium head*, if head is 15 to 30 m.
c) *Height head* , if head is more than 30 m.

The different types of the ***reservoir intakes*** are

1) Dam Intakes

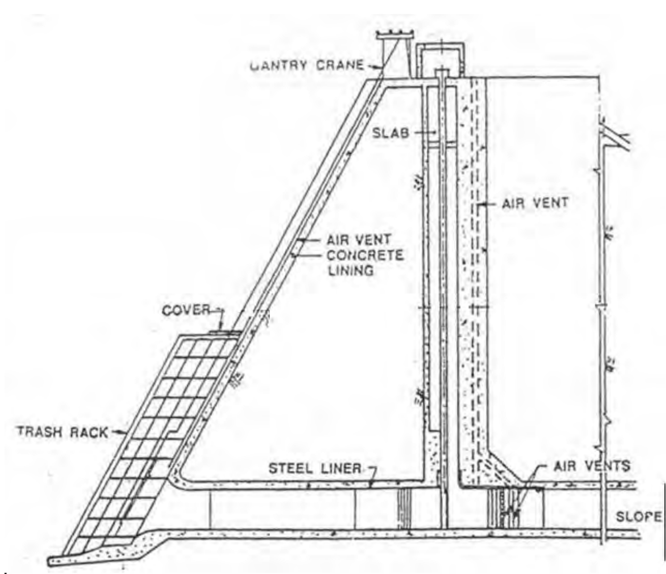
Intake for concrete and masonry dams may have a semi-circular shape in plan and a general cage like structure. The intake structure may rest against the face of the dam and at bottom may be supported on rock or on a slab cantilevering from the dam. Figure shows a typical cage-shaped intake and semi-circular type intake. Penstocks are embedded in the body of the dam. Provisions like trash rack, bell mouth at entry, stop logs, gate etc. are provided. The alignment of the intake may be either horizontal or inclined. The minimum depth of water above the center line of penstock is to be more than 0.8 times the entrance height, h_e.



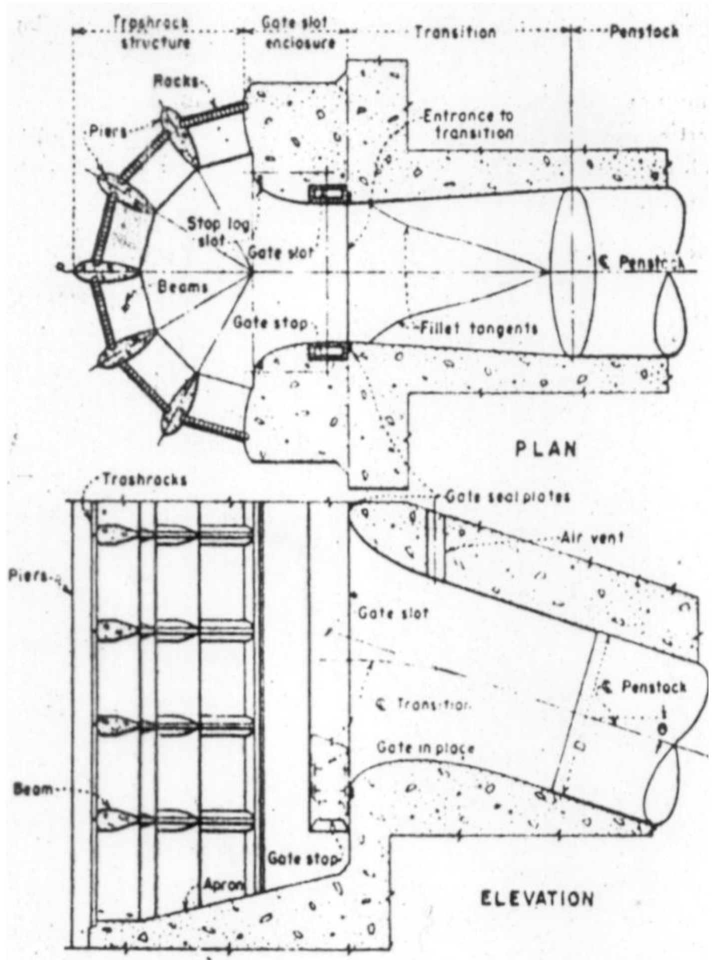
DAM INTAKES

2) Intakes in reservoir independent of dams.

When the intake is provided on a hill slope, this arrangement may be provided. The intake for tunnels taking off from reservoir falls under this type. Good rock exposure is an advantageous situation. A very economical layout for such an intake can be evolved. The provisions as in dam intakes are also provided for this type of intake.



INTAKES INDEPENDENT OF DAM



SEMI CIRCULAR INTAKE

3) Re-entrant type of Intake

This type of intake is adopted :
a) on upstream face of dam;
b) in open channel with flat bottom; and
c) where the width of dam is inadequate to accommodate the intake.

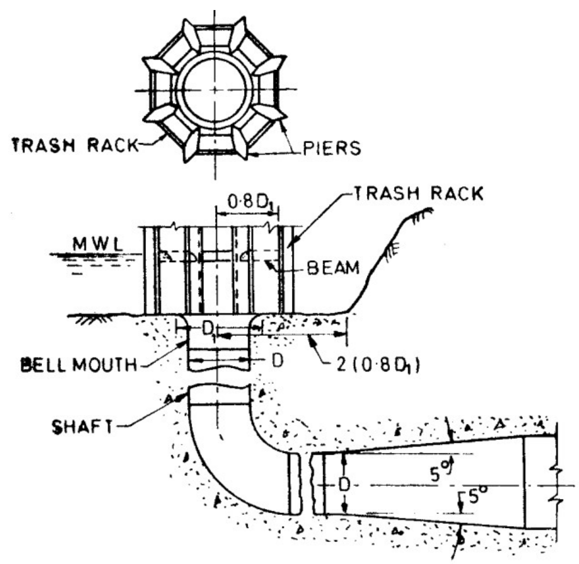
4) Shaft Intakes or glory hole intake

This type of intake is provided for earth dams. In this type, a vertical shaft constructed in the reservoir site which carries water to the penstock tunnel feeding the Power House. The components of a shaft intake are

a) entrance structure with trash rack
b) vertical shaft, followed by an elbow and transitions connecting the shaft with tunnel.
c) intake gate and stop log.
d) access tunnel to the intake structure for entering it from top of dam under submerged conditions.

5) Tower Intakes

Such intakes are provided when it is not convenient to provide the simple intake directly on the upstream of the dam. They are also used when a large discharge is to be handled or when there is a wide fluctuation in the water levels. The tower may be accessed from the dam through a bridge, if the tower is situated near by the dam. Trash rack, stop-log, gates etc. are provided within he tower itself. The tower should be designed for hydrostatic pressure, seismic forces, wind pressure, dead and live loads etc.



TOWER INTAKE

3.1 Components of Intake structure

The main components of an intake structure are :

i) Trash rack
ii) Trash rack supporting structure
iii) Stop logs and control gates
iv) Anti vortex arrangements
v) Bell mouth and transition.

i) *Trash racks*

Trash rack is a metal screen provided at the intake to prevent entry of floating debris like grass, leaves, trees, timber etc. into the water conductor system. In cold countries entry of ice sheets are also prevented, trash racks are sometimes heated up to melt the accumulated ice. 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. Figure shows the general arrangement of trash rack. The trash bars are of mild steel flats with rounded edges at both upstream and downstream for smooth flow. The spacing of trash bars depends upon the type of turbine, its dimension and the peripheral speed of the runner.

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

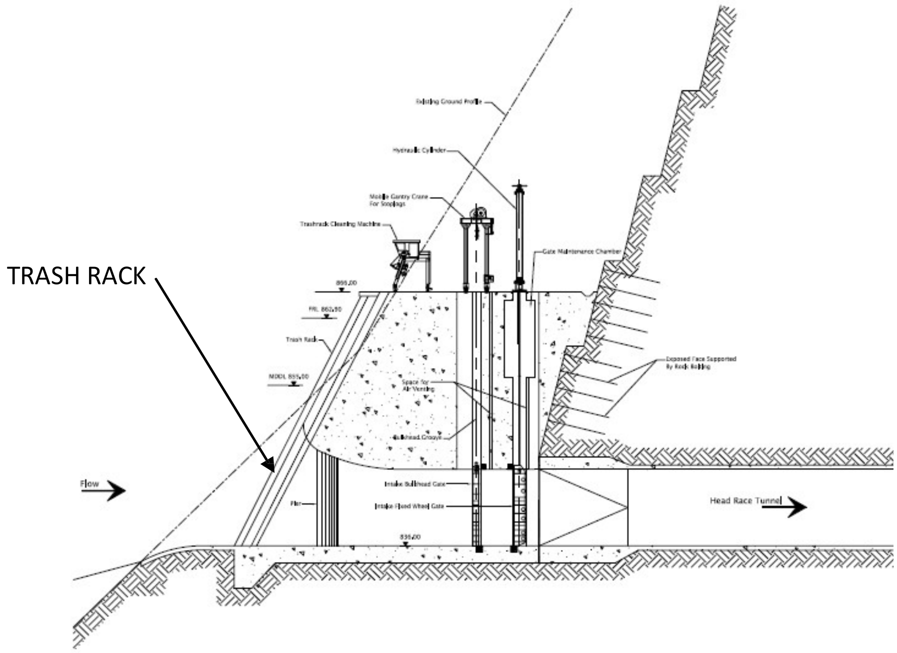
The design loads for trash racks are the dead weight of the assembly, the water pressure and dynamic pressure of the floating materials. An unbalanced pressure is also developed on account of partial or total clogging of the racks. Mosonyi (5), suggests a differential head of 1 to 2 m under normal conditions and 4 to 5 m under exceptional conditions.

U.S.B.R. (8), 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 2/3rd the maximum depth of submergence. Deeply submerged trash racks may be designed for heads up to 6m.

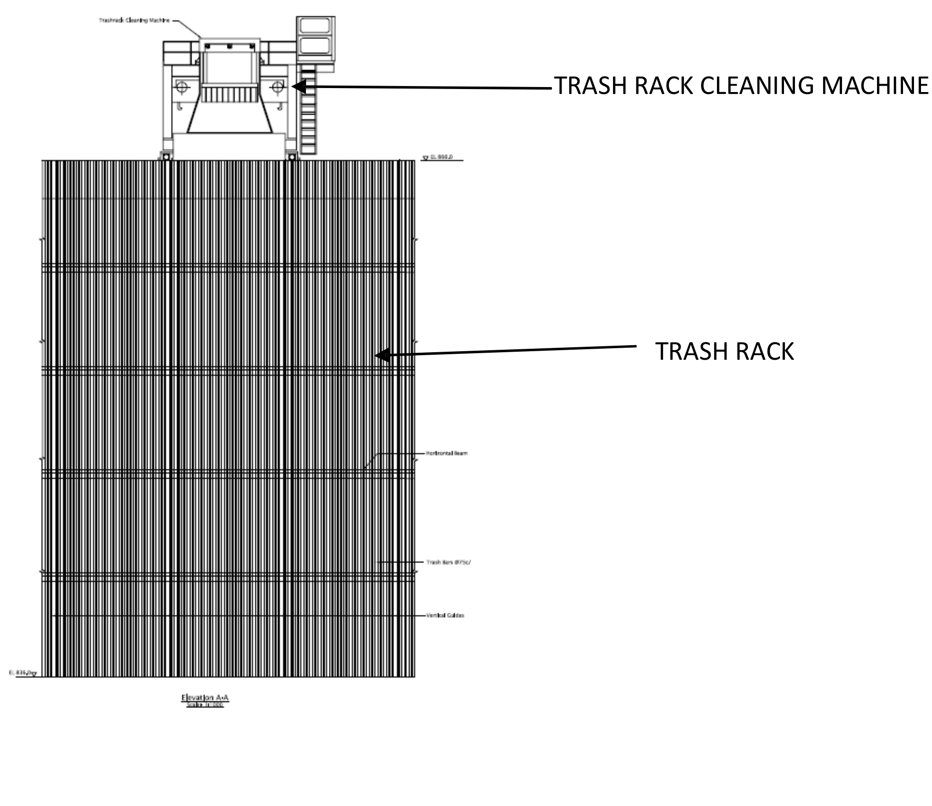
As per IS 9761 for the design of trash rack piers, ribs and screens a differentia head of 3-6 m may be adopted depending upon the efficiency of cleaning of racks being adopted.

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 are deployed for cleaning.

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 structures and may lead to its failure. The velocity of flow through the rack may be about 0.75 m/s. if manual raking is resorted to and 1.5 m/s if the cleaning is by mechanical raking.



TYPICAL INTAKE STRUCTURE



ii) Trash rack supporting structure

This is a reinforced concrete structure of columns (piers) and beams 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 as shown in the figure.

The designs of the supporting structure are done 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 3 to 6 m is considered depending upon the efficiency of cleaning of trash racks being adopted. The columns and beams coming in the flow direction are so shaped as to affect smooth flow.

The shape of trash rack structure may be so adopted to meet the requirements of the 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. However, model studies required for suitability of shape and size of piers and beams of trash racks should also aim at 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 percent of the intake height, h_e, from the centre point of intake.

For an upright semicircular intake structure ,the racks should be located on a semicircle in plan with a minimum radius of 1.142 8 $b_{e,}$ 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 θ), where θ, 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 1.142 8 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 bellmouth, where D_1 is the inlet diameter of the bellmouth.

The piers and beams of the trash rack supporting structure should be sharp nosed and should be streamlined about the required structural section.

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

These 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.

The operating platform 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. 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:

$$F = \frac{Q\sqrt{S}\,(\frac{D}{t})^{(3/2)}}{750\ 000c}$$

Where

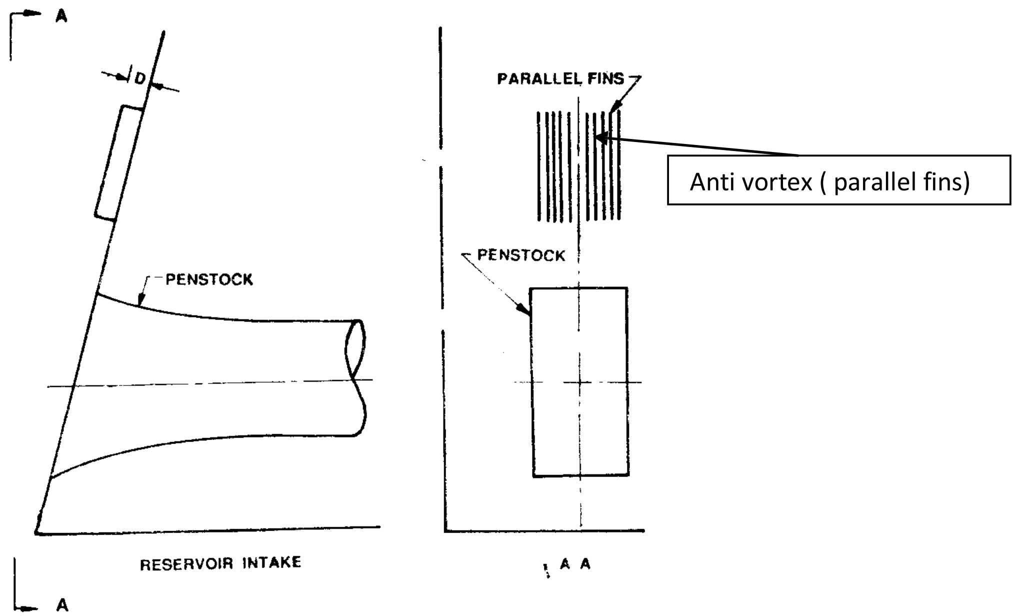
F = Area of air vent pipe in m^2,
Q =Maximum discharge through penstock. Discharge of air through penstock is taken as 21 to 22 percent of penstock discharge,
S =*factor of safety against collapse of pipe (normally assumed between 3 and 4),*
D =diameter of penstock in m,
t =thickness of penstock in m, and
c =co-efficient of discharge through inlet (0.5 for ordinary type of intake valves and 0.7 for short air inlet pipes).

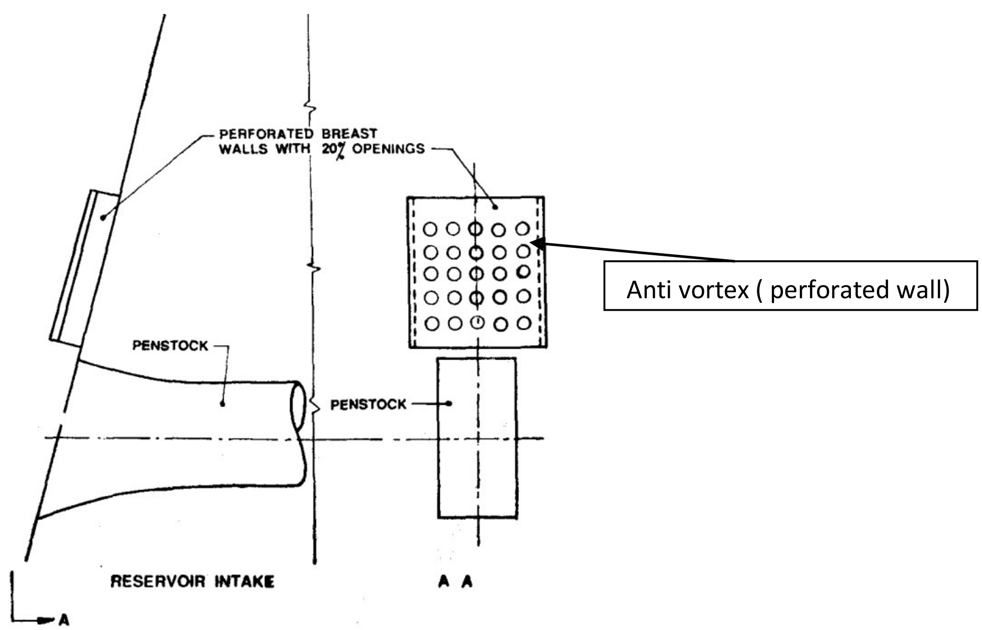
iv) Anti vortex arrangements

These are elements provided 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.

The requirement of water cover may be reduced with the provision of such anti - vortex devices.

For the design of perforated breast wall, anti-vortex louvers and vertical fins, a minimum of 1 m 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 vertical with respect to the dam axis.

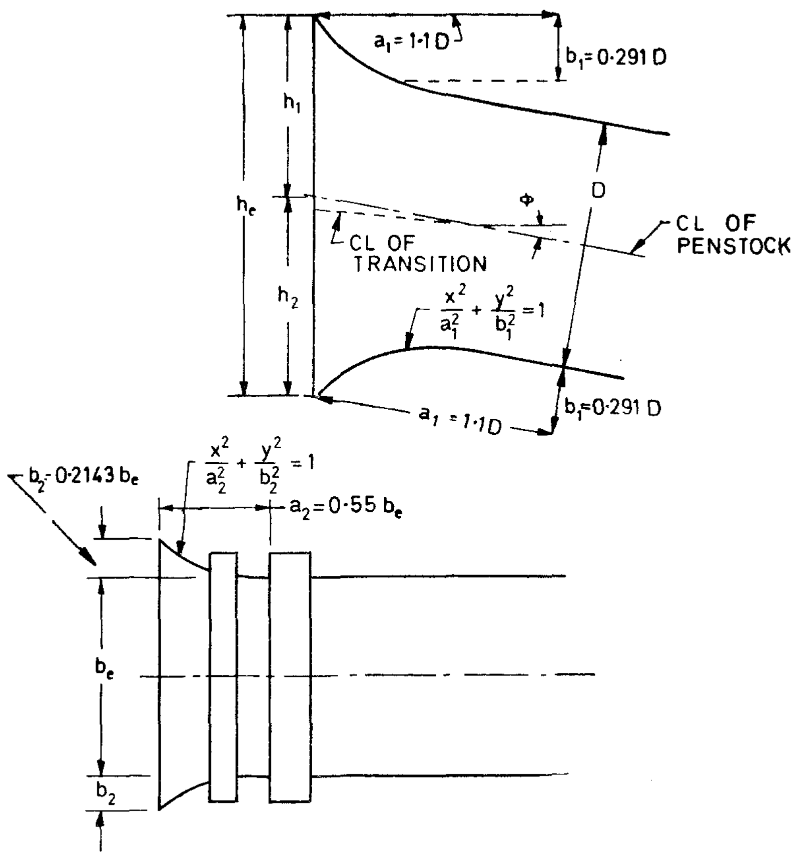
Shape of inlet

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 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 ϕ),
where, ϕ = angle of inclination of penstock centre line to horizontal, as shown in figure



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 (As shown in figure, for lower and upper nappe and side flaring).

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

$$h_2 = D[(\frac{0.791}{\cos\phi} + 0.077\tan\phi)]$$

D is dia of penstock

The opening height, $h_e = h_1 + h_2$

The width of opening, $b_e = (\text{Area}/ h_e)$

Shape of opening

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

$$\frac{x^2}{(1.1D)^2} + \frac{y^2}{(0.291D)^2} = 1$$

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

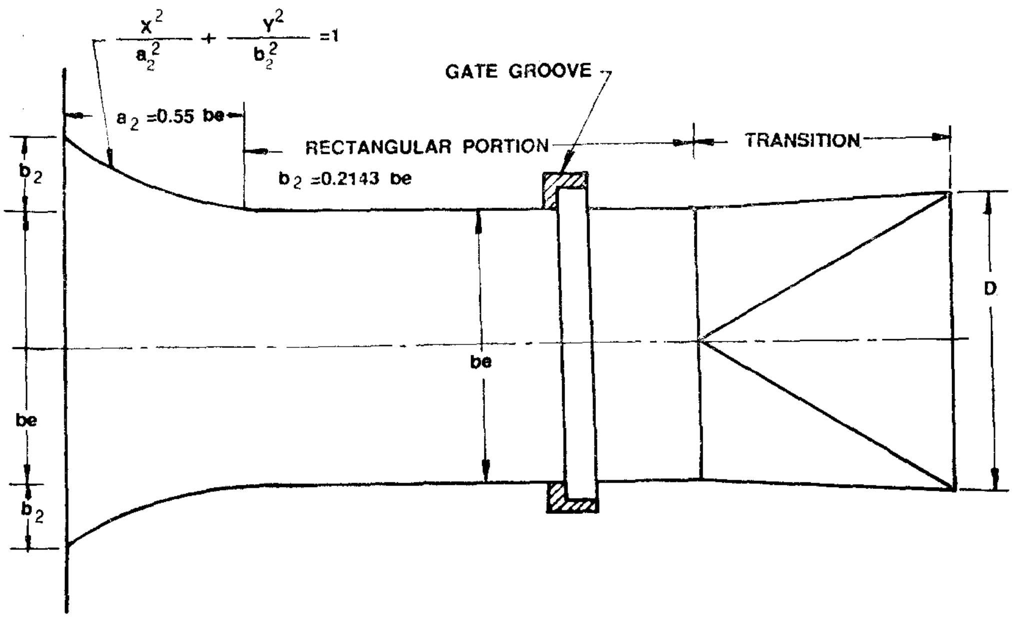
$$\frac{x^2}{(0.55b_e)^2} + \frac{y^2}{(0.2143b_e)^2} = 1$$

While providing side flaring it may be ensured that the size of opening at entry does not create any structural problem 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

In order to obtain hydraulically efficient design of intake transitions from rectangular section to a circular section conduit, the transition should be designed in accordance with the following requirements:

a) Transition or turns should be made about the centre line of mass flow and should be gradual.
b) Side walls should not expand at a rate greater than 5^o from the centre line of mass flow.
c) All slots or other necessary departures from the neat outline should normally be outside the transition zone.



Centre Line of Intake

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, etc.

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.

To prevent vortices, the centre line of intake should be so located as to ensure submergence requirements given in **Fig.18**, which has been developed by an evaluation of minimum design submergence at prototypes operating satisfactorily.

For large size intakes at power plants:

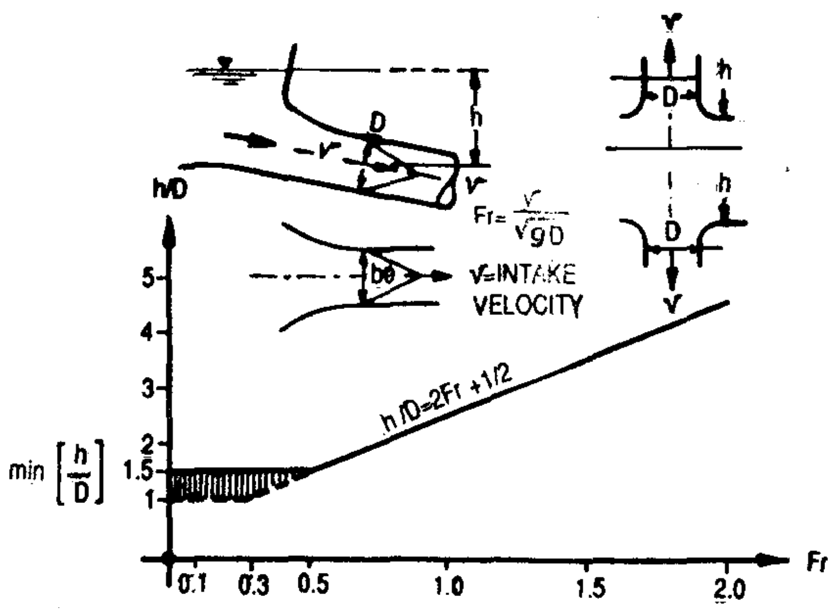
$$\left(F_r = \frac{v}{\sqrt{gD}} < 1/3\right)$$

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:

$$\frac{h}{D} = 0.5 + 2\,F_r$$

The recommendations are 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.



large size intakes ←ϕ→ medium and small size installations, e. g., all kinds of outlet control structures, intakes at navigation locks, diversion tunnels and water supply reservoirs cooling water inlets and especially pump intakes

for power plants, especially pumped storage systems 1m/s - v - 3m/s (mean value : 2m/s)

2m/s - v - 6m/s (mean value : 4m/s)

FIG. 18 RECOMMENDED SUBMERGENCE FOR INTAKES WITH PROPER APPROACH FLOW CONDITION BUT WITHOUT USE OF SPECIAL DEVICES FOR VORTEX SUPPRESSION

4.0 TYPICAL DESIGN FOR AN INTAKE STRUCTURE INCLUDING TRASH RACK

A typical design is given below:

DESIGN DISCHARGE	Q	135	m3/s
DIA OF penstock	D	5.35	m
COEFF OF CONTRACTION	Cc	0.6	
AREA OF penstock	A = pi()*D^2/4	22.48	m2
Angle of Penstock with horizontal	Theta	25	degree
OPENING AREA	A1=A/(Cc*Cos(theta))	41.34	m2
HEIGHT OF THE OPENING	He =H1+H2	**8.224**	**m**
FOR 25 DEGREES	H1	3.3625	**m**
H1=((1.21*(TAN(RADIANS(Theta)))^2+0.0847)^0.5+			
1/(2*COS(RADIANS(Theta)))-1.1*TAN(RADIANS(Theta)))*D			
	H2	4.86143	**m**
H2 =D*(0.791/(COS(RADIANS(Theta)))+0.077*TAN(RADIANS(Theta)))			
HEIGHT OF THE OPENING	He =H1+H2	**8.224**	**m**
WIDTH OF THE OPENING	Be = A1/He	5.027	**m**
ADOPT	**8.224 X 5.027 M OPENING**		
Fixation of CL of Penstock			
FRL	75.5	m	
MDDL	74	m	
Hs =0.3He	2.467	m	
Center Line of Penstock = X	**65.509**	m	
He	**8.224**	m	
Top of the Bell Mouth = X+He/2	68.8715	m	
Bottom of the Bell Mouth = X-He/2	60.648	m	
Top of the Trashrack Strcuture	76.2	m	

SHAPE OF BELLMOUTH			
		adopted	
X = 1.1 D	5.885	**5.885**	m
Y = 0.291 D	1.557	**1.557**	m
Ro = =0.077*D	0.412	**0.412**	m
Horiz Distance of Point A from base line of Dam		**7.1607**	m
Inclined Distance of Point A from base line of Dam		**7.9009**	m
Elevation of Point A		**62.17**	m
EQUATION OF ELLIPSE			
EQUATION FOR PLAN		**adopted**	
X = 0.55 Be	2.765	**2.765**	m
Y = .2143 Be	1.077	**1.077**	m
Min Distance 0.4 Be	2.011	**2.011**	m
Total Width after plan Flare profile	#NAME?	**7.181**	m
ADOPT SEMICIRCULAR TYPE OF INTAKE STRUCTURE			
MIN RADIUS OF TRASH RACK	1.1428Be	5.74	m
MIN RADIUS OF TRASH RACK	0.8 He	6.579	m
VEL IN penstock	V	6.005	m/s
APROACH VEL	Va	0.9	m/s
MAX RADIUS OF TRASH RACK	R=0.354*He(V/Va)^0.5	7.52	m
ADOPT RADIUS		**7.6**	**m**

Trash Rack Strcuture Dimensioning			
Width of Trash Rack Groove		0.32	m
radius upto the centre line of trash rack	R''	7.76	m
angle of trash rack Structure		180	degrees
TRASH RACK Perimeter	=2PI()*R''	24.38	m
no of bays		6	
6 BAYS OF WIDTH=perimeter/no of bays		4.1586	m
Width of the Trashrack frame		3.711	m
Clear Span		3.331	m
TOTAL HEIGHT OF THE TRASH RACK STRUCTURE		19.671	m
NO. of tiers		5	
5 TEIRS OF HEIGHT		3.934	m
	3.671	**3.904**	m
ADOPT3671 x 3904 MM TRASH RACK			
Trash Bar Design			
THICKNESS OF BARS	t	0.012	m
no of divisions		6	
EFFECTIVE LENGTH	L	0.6507	m
RATIO	L/T <70	54.22	SAFE
Runner Dia		4.5	m
SPACING OF BARS C/C	0.15	0.112	m
LOAD ON BAR	W	0.672	T/M
BM	M=WL^2/10	0.0285	T-M
FAILURE STRESS	fy*(1.23-0.0153*L/T)	660.6	kg/cm2
SAFE STRESS	0.6fy*(1.23-0.0153*L/T)	396.4	kg/cm2
Z REQUIRED		**7.18**	**CM3**
width of trash bar		7.5	CM
Z PROVIDED		**11.25**	**CM3**
CHANNEL SECTION DESIGN			
Differential Head for Design		7	m
LOAD ON CHANNEL	W	4.555	t/m
EFFECTIVE SPAN	L	3.671	m
MAX MOMENT	M=WL^2/10	6.137	T-M
Z REQD	Z=M/f	**371.929**	**CM3**
Z - PROVIDED	ISMC 300	**424**	**CM3**
PROVIDE 75x 12 MM ROUNDED STEEL FLAT AT CLEAR SPACING OF 100 MM			
ISMC300 AS FRAME			
Check for Velocity through trash rack			
Beam size		0.6	m
gross area		331.14	m2
Actual Area of trash bar and frame		81.9	m2
net area	70% of gross area or act	231.8	m2
net area (actual)		249.24	m2
clogged area	50% net area	124.62	m2
vel through gross area		0.41	m/s
vel through net area (actual)		0.54	m/s
vel through clogged area		1.08	m/s

DESIGN OF DESILTING CHAMBERS

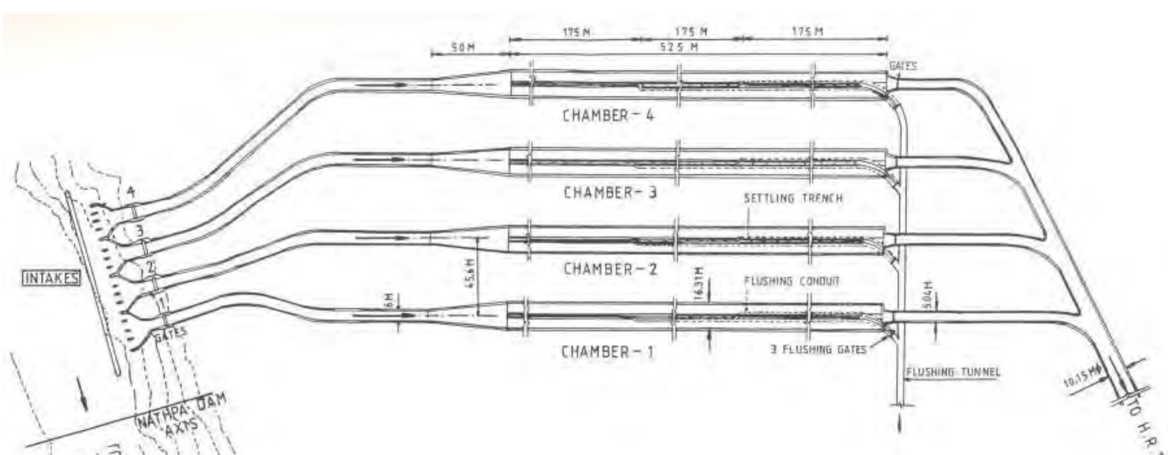
INTRODUCTION

Most of the rivers carry heavy sediment load in suspension and as bed load. The suspended load, especially the sharp edged fine sand (quartz) transported by rivers in mountain reach 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. De-silting basins, also known as silting tanks, settling basins, sediment traps, decantation chambers are used for this purpose.

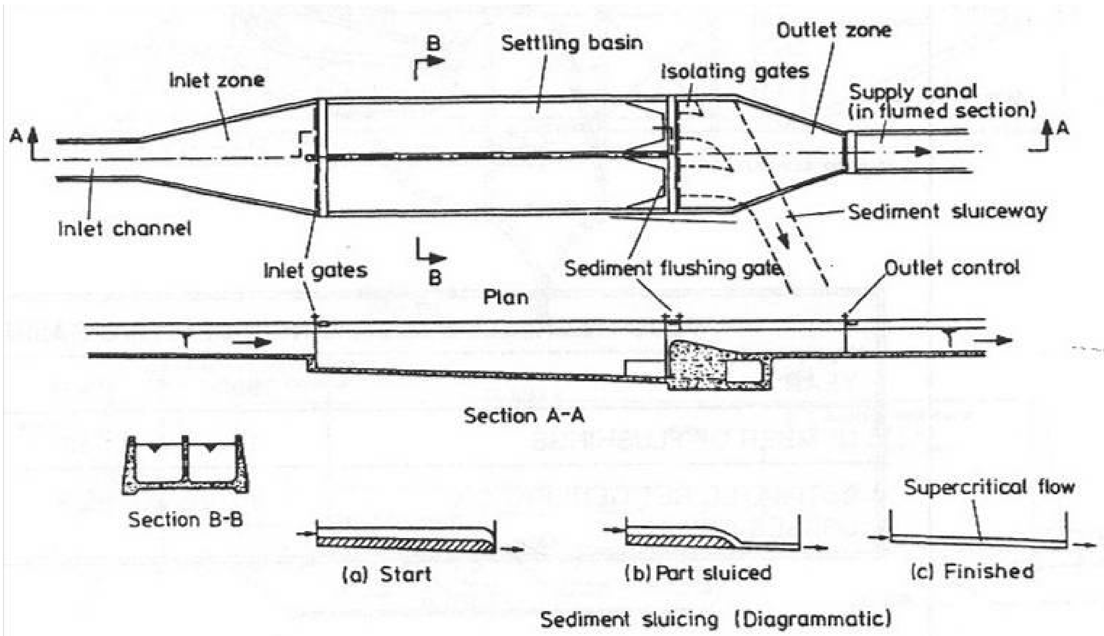
1.1 **Types of desilting basins**

Desilting basins can be classified into various types as,

i) *Natural or artificial* based on the mode of construction
ii) *Manual or mechanical or hydraulic removal of deposition,* on the basis of the method of cleaning.
iii) *Continuous or intermittent,* on the basis of mode of operation.
iv) *Open channel or closed conduit* on the basis of type of flow.
v) *Single or multiple unit*, on the basis of configuration /layout.



UNDERGROUND DESILTING CHAMBERS FOR NATHA JHAKRI H.E. PROJECT (HP)



TYPICAL GRAVITY FLUSHING IN SETTLING BASIN

2.1 Hydraulic design of desilting basin

Apart from the settling efficiency and flushing system, in the hydraulic design the flowing aspects are to be taken into consideration:

i) ***Location and orientation***

Desilting basin is to be located as near the head works / intake as possible to achieve the desired control and minimize sedimentation , but not too near the intake / head works as it would lead to turbulence downstream of the intake / head regulator. Moreover, the required head for flushing may not be available in the immediate vicinity of the head works in the case of hydraulic flushing. The basin 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 basin is required to be increased for reducing the velocity to induce settlement. The increase in area is achieved by suitable horizontal or vertical divergence. For wide basins 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 basins in the tunnels, bed slope has to be steeper than that of the slope provided for wide basin 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 turbulences 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 0.2mm and 0.15m/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 traveled 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 main criteria for efficient settling of sediment in a basin is the fall velocity, but due to the turbulence in the flow the actual fall velocity is reduced and its estimation is very difficult. Subsequently based on diffusion and probability theory several functions are proposed by Lamble, Rouse, and Camp which are justifiable and 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 sill level. The center line of the outlet should coincide with the axis of the desilting basin for uniform with drawl of flow over the entire widths of the basin. Narrow outlets or outlets located on the side, leads to a reduction in the effective length of the basin.

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 basin. 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 basin 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 basin is divided into a number of hoppers. In wide basins, 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 basin. In the case of narrow desilting basins, instead of individual rectangular hopper, a continuous hopper 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 basin to the flushing trench. Normally, 10 to 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 basin 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 3m/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 basin. 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 velocities in the escape channel has to be more or at least equal to the velocities in the flushing system at its outlet at the tail end of the desilting basin. The hydraulic parameters such as width, depth and slope may be calculated on the basin of Maning'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(5)

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 the case 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 the 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 bank of the bend for further efficient transport of the sediment in the river.

3.0 Model Studies

Stilling basins are designed based on broad guidelines, assumptions and experiences. 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 basins. In the case of closed conduit type basins, transperant Perspex material is used for convenience of fabrication and for visualization of the flow in the basin.

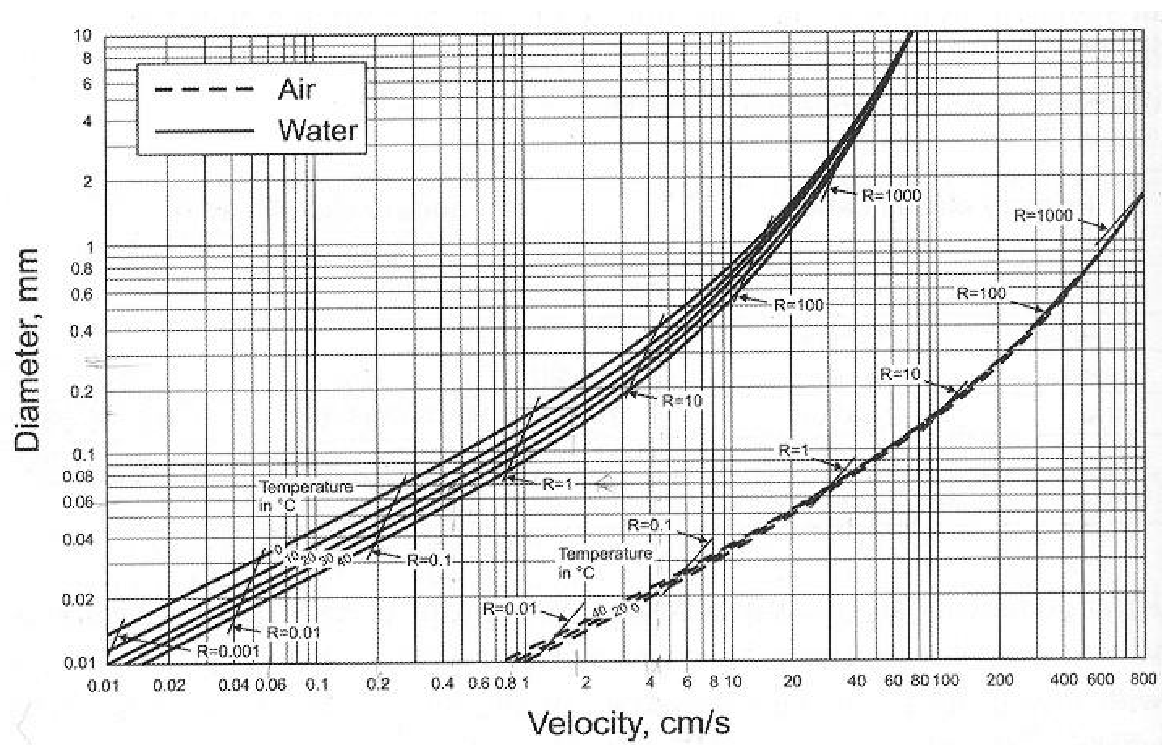
DESILTING BASIN DIMENSIONS- EXAMPLE

Design a settling basin for a powerstation utilising the river of a water carrying sediment. The basin have to be designed to remove particles of size 0.2 mm and above. The design discharge is 15 m^3/s.

Criticle veocity (limiting flow through velocity) can be obtained from Camp equations as :

$V = a\sqrt{d}$

where V = flow through velocity in m/s
d = diameter of particle up to which sediment load is desired to be removed
a = constant which is
0.36 for d >1 mm
0.44 for 1mm > d> 0.1 mm
0.51 for 0.1 m > d
Take height of desilting chamber as 8.75 m
Use Rouse diagram for fall velocity as:



Use suitable assumptions

Solution.

Critical velocity $V = 0.44(0.2)^{1/2} = 0.196$ m/s

In designing the basin, a flow through velocity is taken as 0.2 m/s

From H. Rouse diagram, for a particle size of 0.2 mm fall velocity is = 0.023 m/s.

Denoting the depth of the basin by 'h' and its width by 'b', the discharge passing through the basin is

$Q = b.h.v$ m^3/s.
where 'v' is the flow through velocity.

The second equation expressing the relation between the settling velocity 'w' in the basin and the settling time 'T' is

$T= h/w$ sec

Also retention period should not be less than settling time. The required length of the basin is thus:

$L = v\ T$ m

Combining these equations, $L = h\ v/w$

$L = 8.75 \times 0.2/.023 = 76.1$ m

Provide length L = 77 m

and width b $= Q/h\ v$
$= 15/ (8.75 \times 0.2)$
$= 8.57$ m
Provide width = 8.6 m

Check :

Settling time = 8.75/0.023 = 380 secs

Discharge conveyed during this time= 380 x 15 = 5700 cum
Volume of desilting chamber = h b L
= 8.75 x 8.6 x 77
= 5795 cum
So the size of desilting basin is :

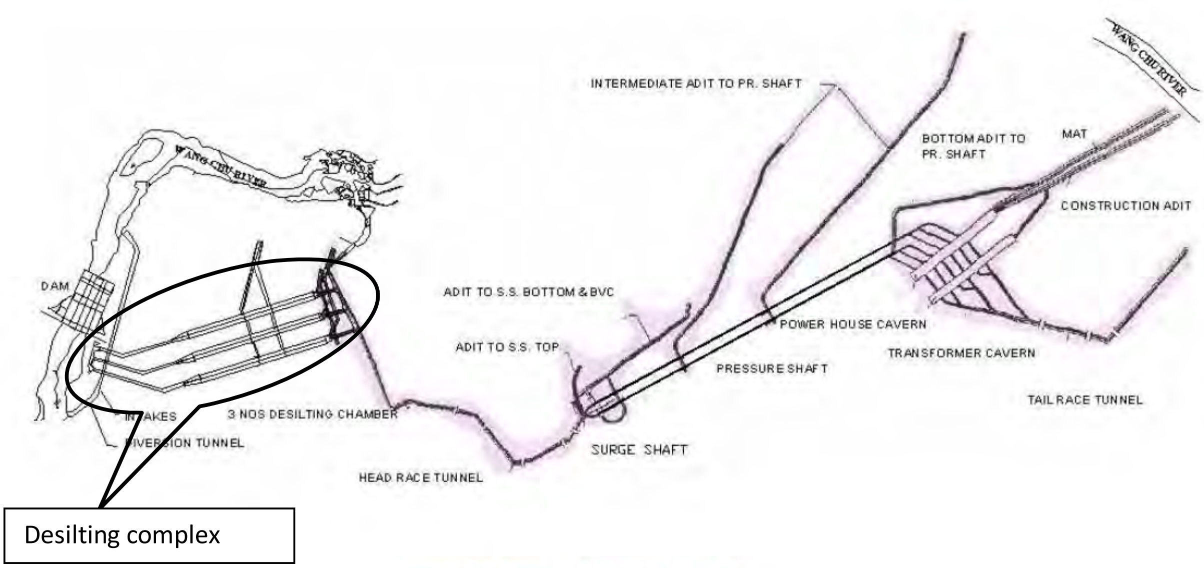
77m (L) x 8.6m(b) x 8.75m(h)

DESILTING CHAMBERS FOR TALA . H.E. PROJECT, BHUTAN

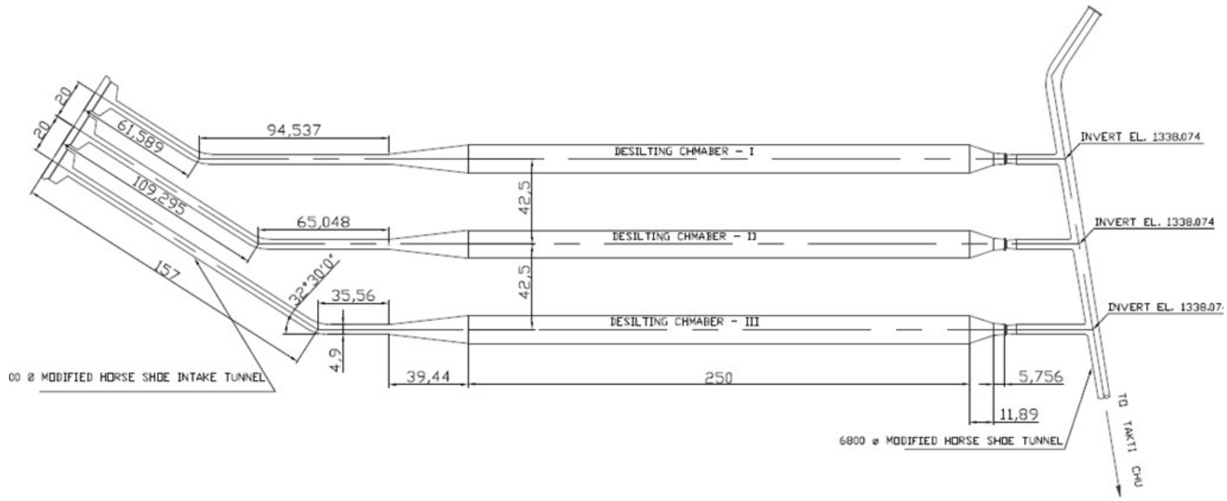
Tala H.E. project comprises of a concrete 92m gravity diversion dam near Honka for diversion of 171m3/s of water for generation of 1020 MW of electricity at 820m net head. There are three side intakes on right river bank, each of which feed three underground desilting chambers. The spillway complex housed, in the central portion of dam comprises of five sluices and one overflow spillway near the left bank would be able to pass the SPF at reservoir water level(RWL) El. 1367m.The 22.25 kilometre concrete lined headrace tunnel is second longest in the Himalayas.

Desilting arrangement

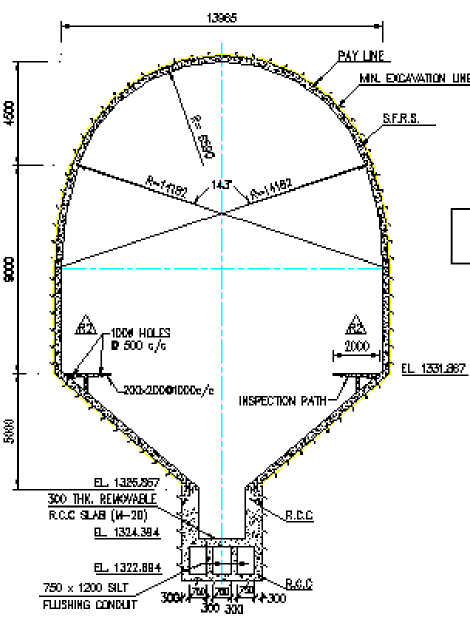
Underground Desilting basin complex is consisting of three chambers. Each fed byindependent inlet tunnels. The maximum width and maximum depth of basin is 18.5 m and 13.92 m respectively. Settling length is 250 m. The design is aimed to exclude particle size above 0.2 mm. The inlet tunnel which is on right bank and have designed for 171 m3/s and includes 20% flushing discharge. The incoming flow to basin diverse in all direction through a 39.44 m. inlet transition and vertically through 1V:2H transition. The outlet of basin converges through 17.646 m transition outlet. The outlet discharge of the desilting basin in head race tunnel is 142.5 m3/s which fed into 22.97 km HRT.



GENERAL LAYOUT PLAN



ALIGNMENT OF INTAKE AND DESILTING CHAMBER COMPLEX



Cross Section of desilting chamber

Model Studies of Tala Desilting Chambers

Physical model studies for desilting chamber has been carried outat Central Water and Power Research Station, Pune. One unit of desilting chamber from inlet to HRT and flushing tunnel, hydraulic model study on 1:30(G.S.) scale model, reproduced partly in fibre glass and partly in transparent Perspex, was carried out at same place. Three numbers of flushing tunnels having size of 0.5m(W) X 1.2m (H) at upstream end and 0.75 m(W) X 1.2 m (H) at the downstream end were also reproduced in the model. Openings of different size at different spacing were provided in the slab separating flushing tunnel and the desilting basin. Initially design discharge of 57 m3/s was run and coarse sediment with high concentration was injected from upstream of inlet and Model was also run for 10% extra discharge at inlet.

It was found that the overall size and shape of the basin is adequate for 90% settlement of material coarser than 0.2 mm diameter. It was also seen that the flushing tunnels below desilting basins were adequate for flushing of the settled sediment. Studies for efficiency of flushing tunnels beyond desilting basin are carried out on a separate model.

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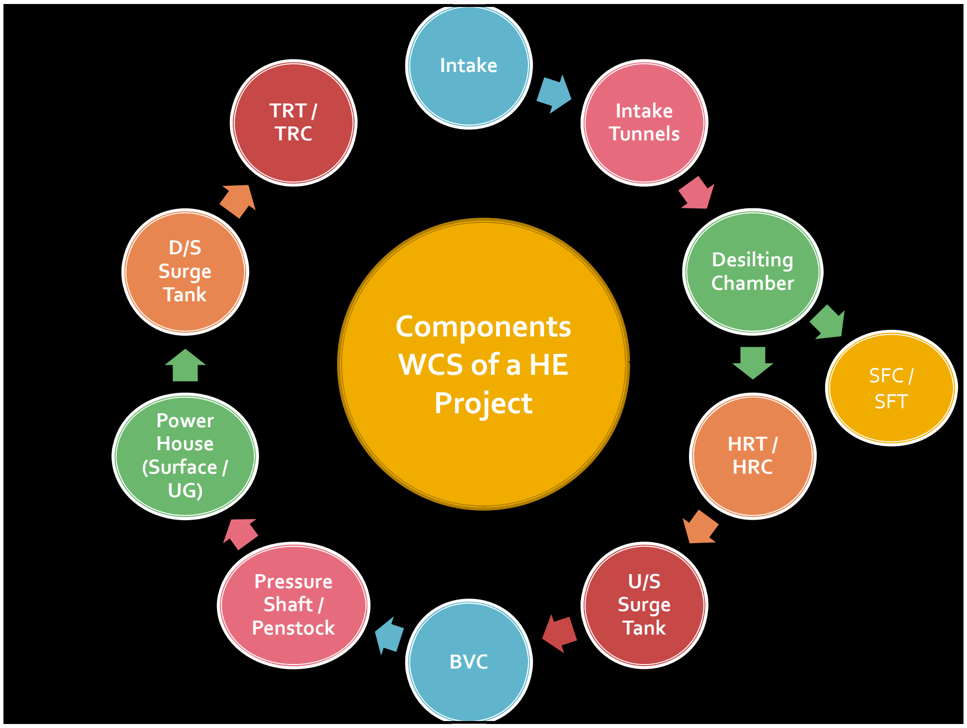
Water Conductor System

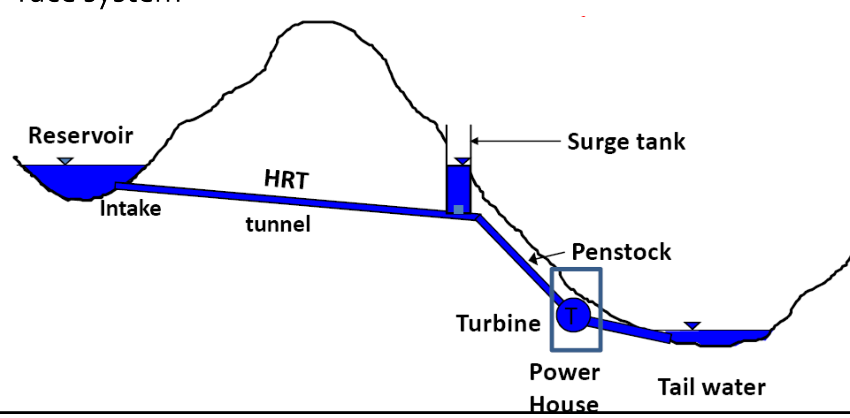
-Manish Rathore
Deputy Director

Introduction



Function

- To carry the water from reservoir to turbines for generating Power and finally joining the river through tail race system



Factors Deciding Type of WC System

- Type of the Scheme
- Maximum utilization of available hydro potential
- Topography of the site
- Geology of the site
- Overall Economy of the scheme

Pre-requisites for WC System

- Should be short to Reduce cost and Increase efficiency
- Capable of utilizing the maximum hydro potential
- Hydraulically Stable
- Structurally Stable
- Shorter length of steel lined section
- Overall Economical

Sample Layouts

Nathpa Jhakri H.E. Project General Layout Plan

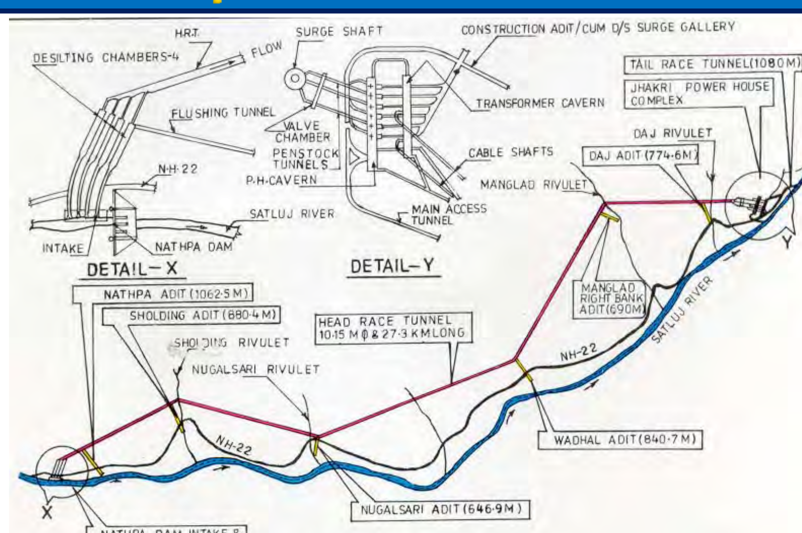
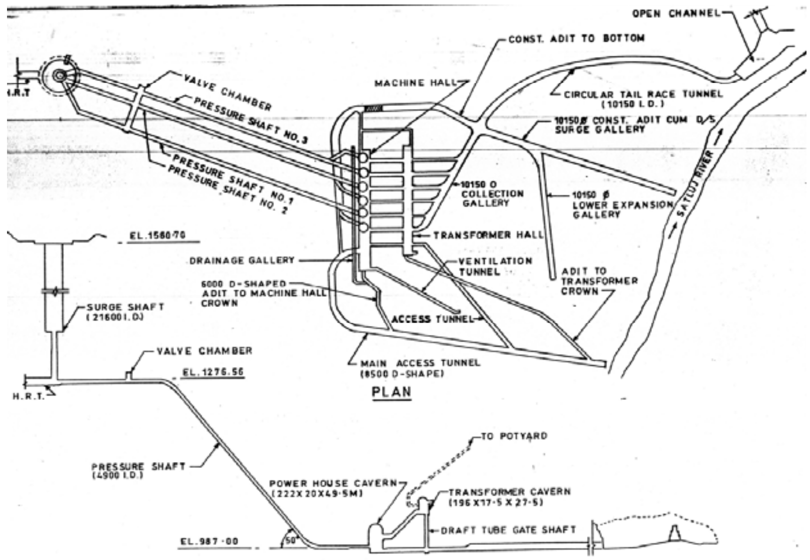
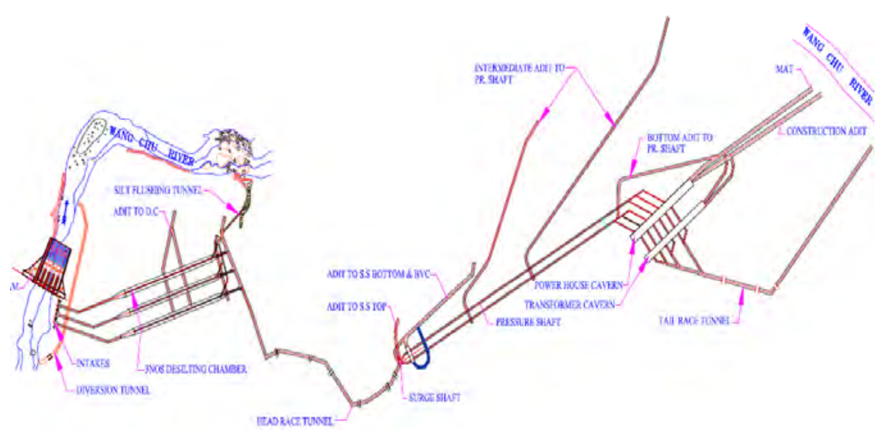


FIG.2 LAYOUT PLAN

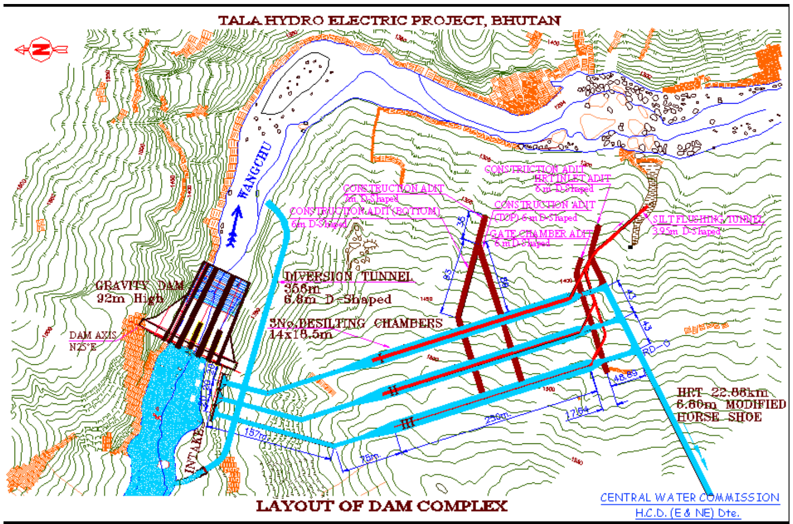
Nathpa Jhakri H.E. Project
Power House Complex



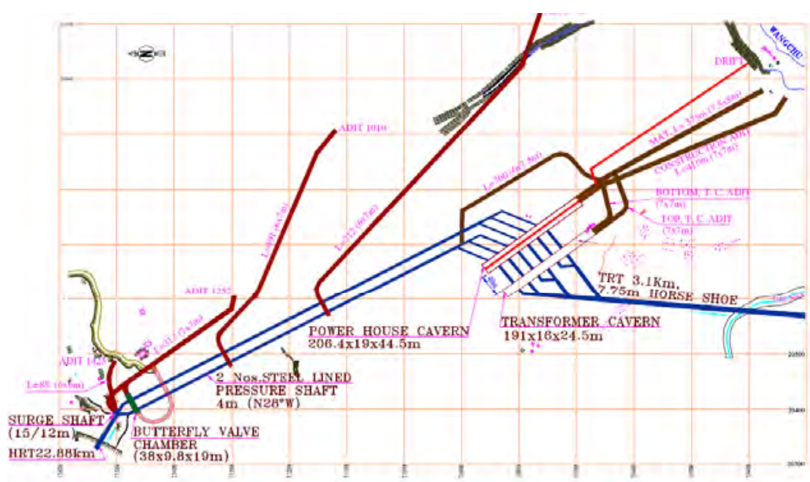
Tala H.E. Project
General Layout Plan



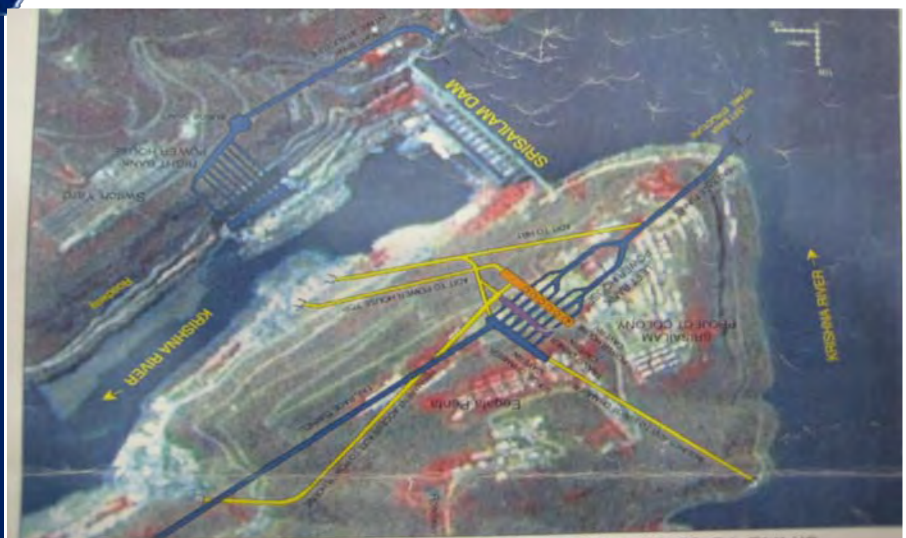
Tala H.E. Project
Dam Complex

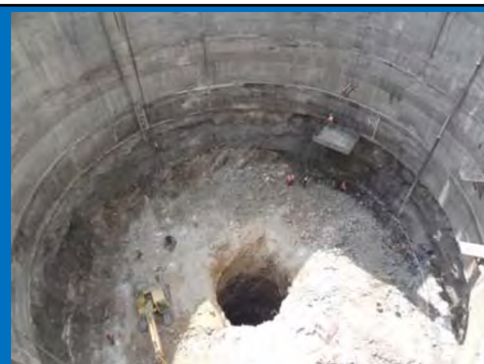


Tala H.E. Project
Powerhouse Complex



Srisailam
Left and Right Power House





Surge Tanks

Surge Tanks

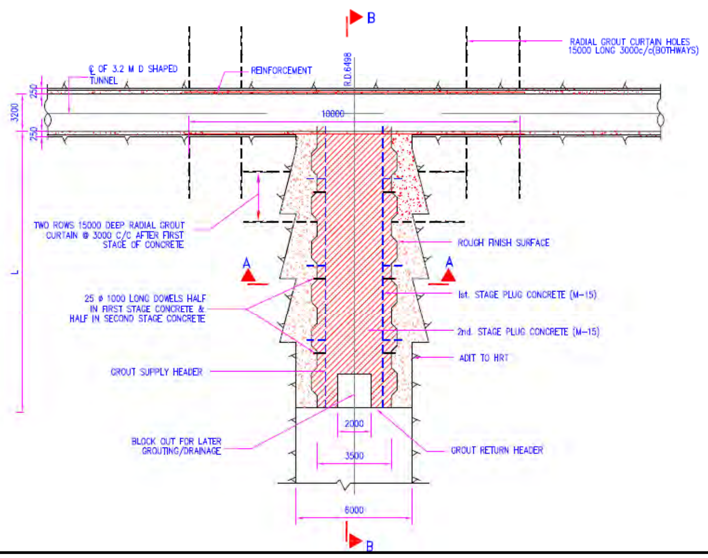
- Surge tank is the forebay provided for water hammer relief, pressure regulation, flow regulation and improvement in speed regulation of the machines.
- Functions
  - Reflects incoming pressure waves
  - Decreases cycle time of pressure wave in the penstock
  - Start-up/shut-down time for turbine can be reduced (better response to load changes)
- Design
  - Hydraulic
  - Structural

Construction Adits

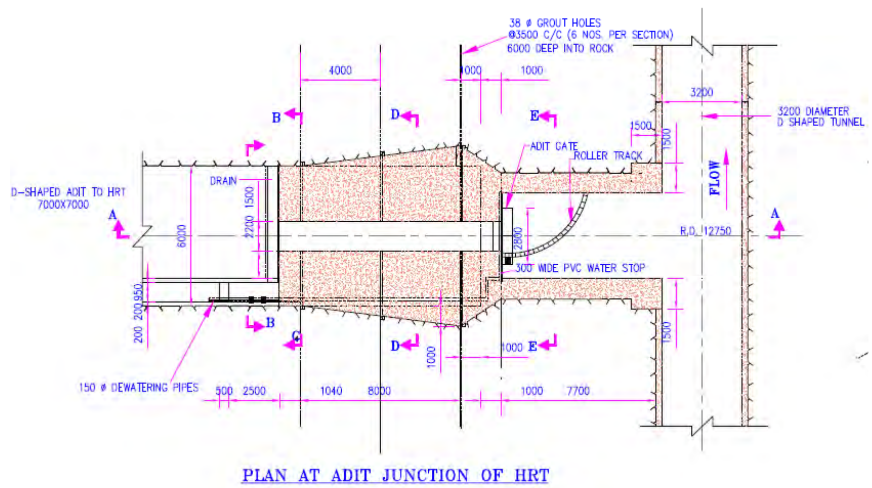
Construction Adits

- Purpose
  - To speed up excavation process
  - Ventilation
  - To shorten travel path for trucks carrying muck
  - Increase in overall economy
- Requirements
  - Length should be kept as less as possible
  - Must be of size so as to accommodate construction machinary
- Must be plugged after construction in case of Hydraulic tunnels

Construction Adit - Plugs



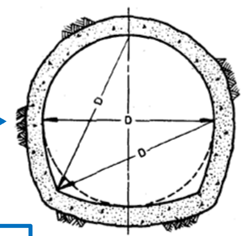
Construction Adit – Plug with Gate

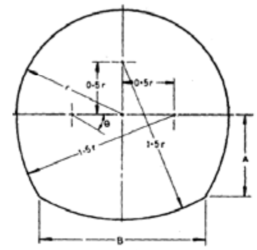


Design Aspects

Shape of Tunnel

- Shape of tunnel
  - Circular
  - D-shape
  - Horse shoe
  - Modified horse shoe





- Factors deciding shape of tunnel
  - Geological conditions along the alignment
  - Hydraulic requirements
  - Structural considerations
  - Functional requirements

Tunnel Alignment

- Tunnel alignment should be kept straight as far as possible. And if it is not possible to avoid the curves in the tunnel, the radius of the curve should be at least in the range of 2.5 D to 5 D.
- The weak zones, foliations and major joint sets should be as oblique to the alignment as possible
- Alignment should be set such that minimum horizontal and vertical rock cover is available on all points along the tunnel

Tunnel Lining

- Hydraulic tunnels are concrete lined to prevent weathering effect on surrounding rock and get smooth hydraulic conditions and not to take external loads.
- However sometimes in very poor rock conditions RCC Lining may be provided to counter the external rock and water pressures.

- Concrete Lining
  - For HRT, Surge Tanks, TRT and other low pressure tunnels
  - Max velocity allowed 3 to 5 m/s
- Steel Lining
  - For high pressure tunnels
  - Max velocity upto 6.5 m/s allowed

Hydraulic Design of WCS

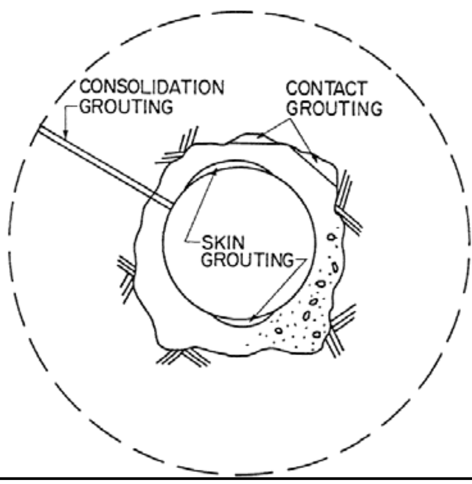
- Cross section of tunnel
- Number of penstock / HRT
- Diameter of penstock / HRT
- Head Losses calculations:
  - At trash rack
  - At intake entrance
  - Friction losses
  - At bends
  - At branches / Y-pieces
  - At reducers and expansions

Structural Design of Lining

- Plain cement concrete
  - PCC not cracked ; Rock not cracked
  - PCC cracked ; Rock not cracked
  - PCC cracked ; Rock cracked
- Reinforced cement concrete
  - RCC not cracked ; Rock cracked
  - RCC cracked ; Rock cracked
- Crack parameters
  - Crack spacing
  - Crack width

Grouting

- Consolidation grouting
- Contact grouting



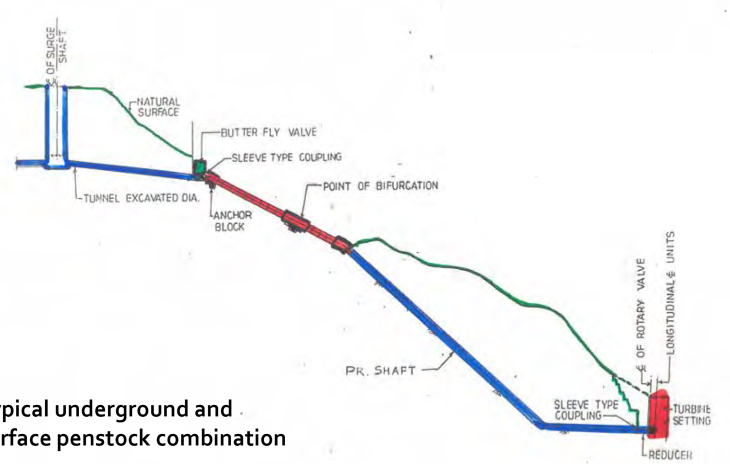
Penstock

- Types
  - Based on location
    - Surface penstock
    - Embedded penstock
    - Buried penstock
    - Penstock in tunnels
  - Based on fabrication
    - Riveted
    - Welded
- Design
  - As per CWC manual

MANUAL ON DESIGN FABRICATION ERECTION AND MAINTENANCE OF STEEL PENSTOCKS

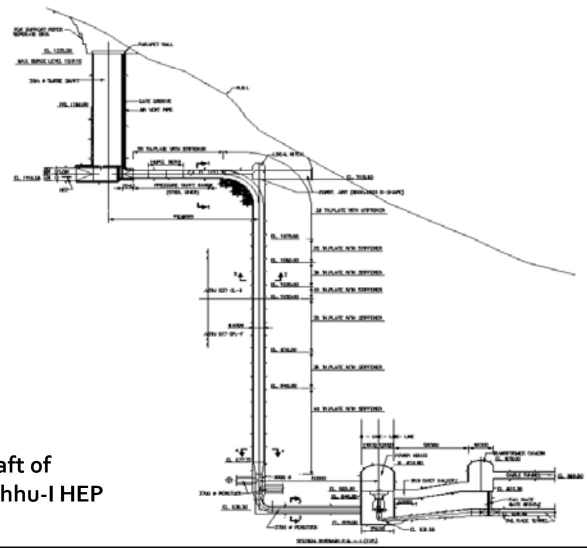
HYDEL CIVIL DESIGN DIRECTORATE-I
CENTRAL WATER COMMISSION
NEW DELHI

Penstock



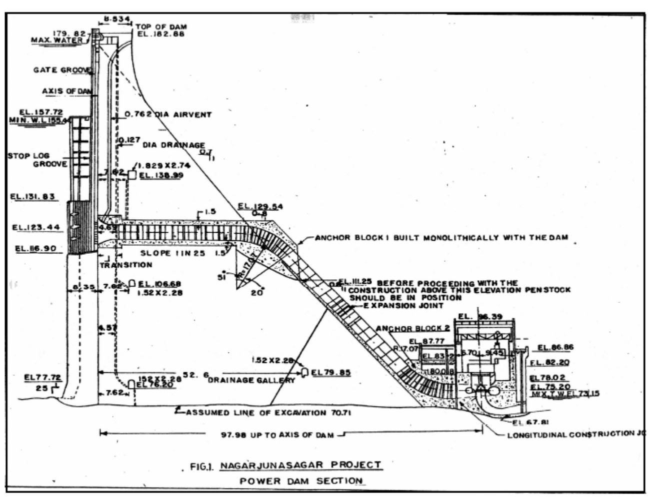
Typical underground and surface penstock combination

Penstock



Pressure shaft of Punatsangchhu-I HEP

Penstock



Penstock of Nagarjuna Sagar HEP

Penstock Design

- It should stand against maximum internal pressure including water hammer and surge effects.
- It should stand against external pressure, if any, without buckling.
- It should have required impact strength to be able to deform plastically in the presence of stress concentrations at notches and bends.
- It should have good weldability without preheating.
- It should not require any stress relieving after welding.

Operating Conditions

- Normal
  - All operations that are likely to occur several times during the life of the penstock are termed as normal.
- Intermittent
  - Intermittent condition includes those during filling and draining of penstocks and maximum surge in combination with pressure rise during normal operation.
- Emergency
  - The emergency conditions are those in which one of the pressure control equipment malfunctions.
- Exceptional
  - Those conditions in which various control devices malfunction in the most unfavorable manner are called exceptional

Operating Conditions: Design Stress

- Normal

  σ_{design} < $\sigma_u/3$ OR $0.6\sigma_y$ whichever is less.

- Intermittent

  σ_{design} < $0.4\sigma_u$ OR $2\sigma_y/3$ whichever is less

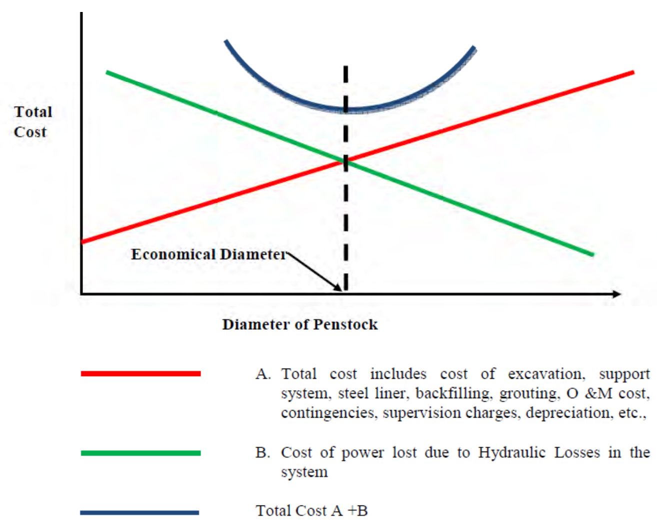
- Emergency

  σ_{design} < $2\sigma_u/3$ OR $0.9\sigma_y$ whichever is less

- Exceptional

  σ_{design} < σ_y

Most Economical Diameter Studies



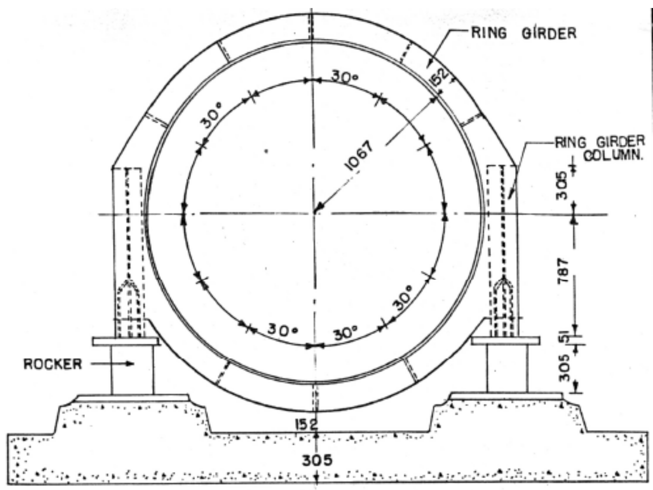
Penstock Appurtenance

Penstock Fabrication

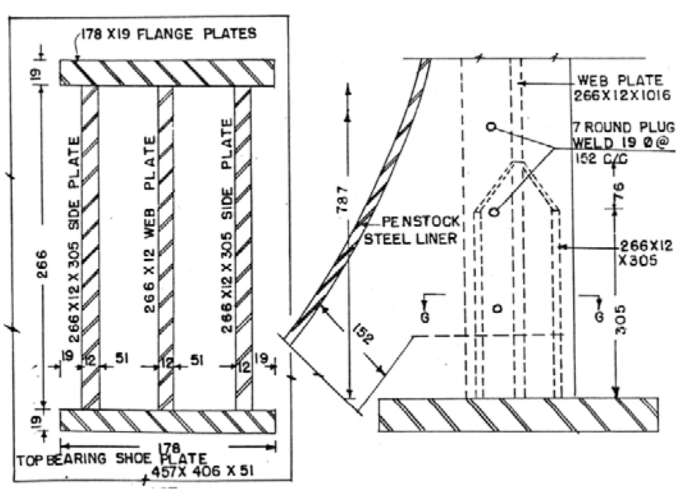
Penstock Fabrication

Penstock Fabrication

Ring Girder Support



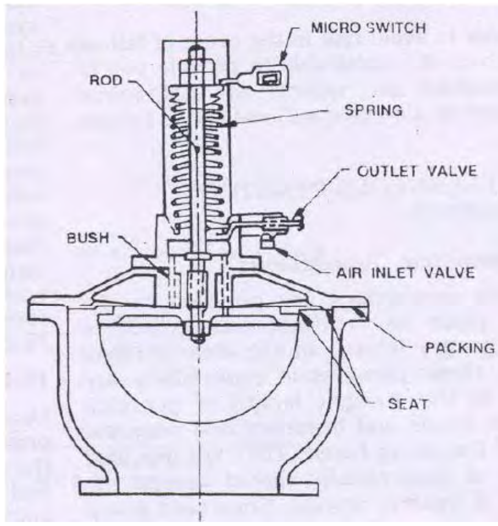
Ring Girder Support



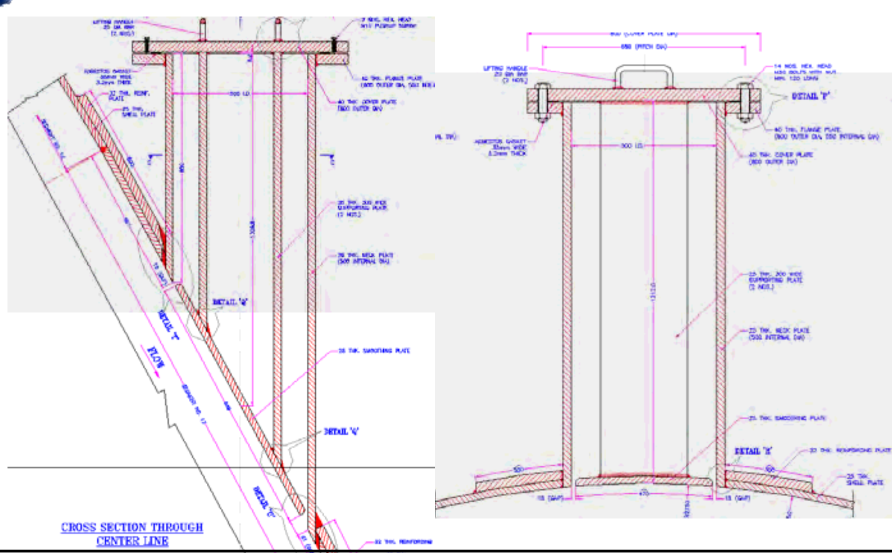
Ring Girder Support

Bulkhead

Air Valve



Manhole

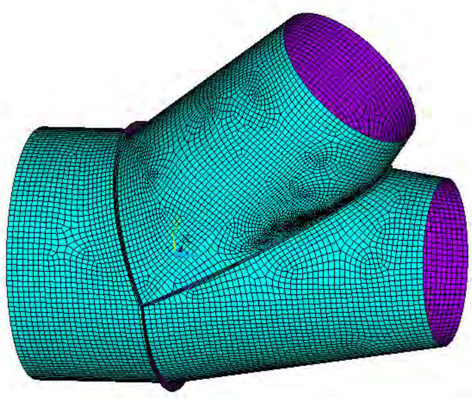


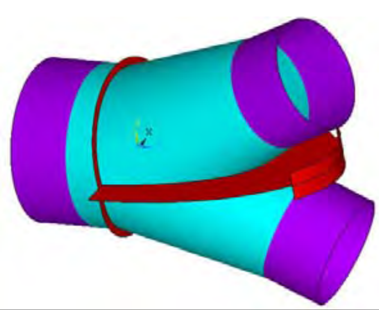
Bends

Thrust Collars

Thrust Collars

Bifurcation Piece



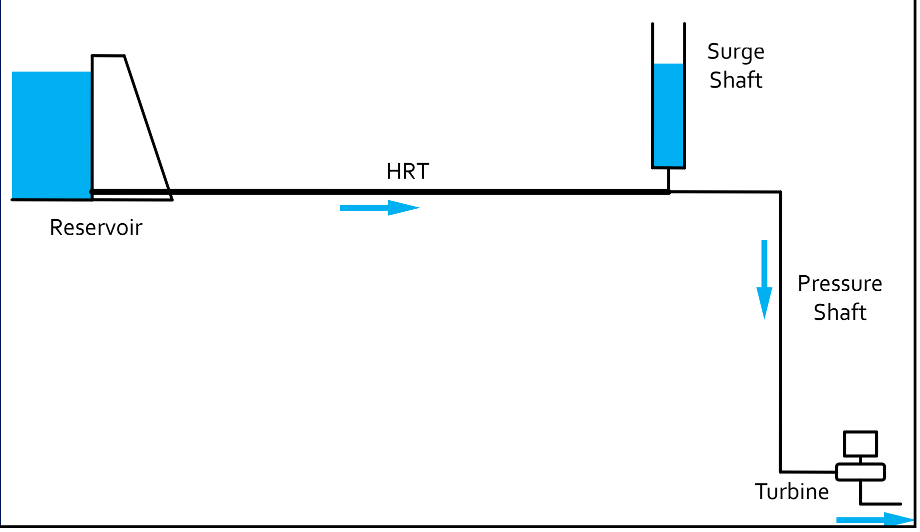


Thank You

Hydraulic Transient Studies

-Manish Rathore
Deputy Director

Layout of Run of the River Scheme



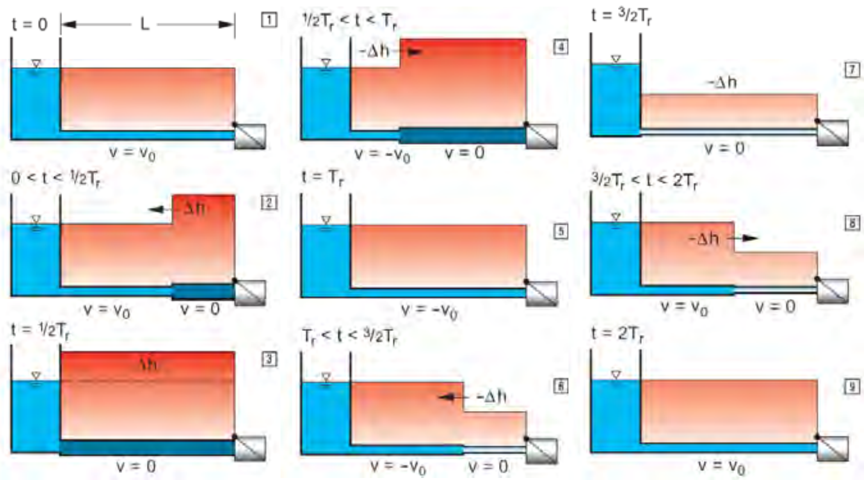
Transients

- What is transient condition…??
  - Flow condition during change from one steady state to another steady state
  - Rapidly varying Unsteady flow
  - During transients, fluctuation of pressure & velocity occurs at a point
- Transients die out due to
  - friction in the system

Causes & Concerns

- In hydroelectric projects, transients are caused by change in boundary conditions at turbines.
  - Sudden load acceptance
  - Sudden load rejection
  - Tripping of turbines
- During transients, pressure rise/drop is the main concern for the design of water conductor system & turbines

Transients in Reservoir – Tunnel – Turbine System



Wave Velocity
Water Hammer
Pressure Surge

Wave Velocity

$$a = \sqrt{\frac{K/\rho}{\left[1 + \frac{K}{E}.\frac{D}{e}\left(1-\mu^2\right)\right]}}$$

Where,	a	=	Wave velocity
	K	=	Bulk modulus of water
	ρ	=	Water density
	E	=	Modulus of elasticity of conduit
	D	=	Diameter of conduit
	e	=	Wall thickness of conduit
	μ	=	Poisson's ratio of conduit material

For rigid conduits a = 1440 m/s

In tunnels, a = 1200 – 1400 m/s

In surface penstocks, a = 700 – 1000 m/s

Water Hammer Analysis

- Equation of motion & continuity in a pipe are

$$\frac{gA}{c^2}.\frac{\partial H}{\partial t} + \frac{\partial Q}{\partial X} = 0$$

Eq. of Continuity

$$\frac{\partial Q}{\partial t} + gA\frac{\partial H}{\partial x} + \frac{fQ.|Q|}{2DA} = 0$$

Eq. of Momentum

- Most common method for solving equations is method of characteristics.
  - The method of characteristics is a technique for solving partial differential equations. Typically, it applies to first-order equations. The method is to reduce a partial differential equation to a family of ordinary differential equations along which the solution can be integrated from some initial data.

Method of Characteristics

PD Equations are transformed into ODEs

$$\frac{gA}{a}\frac{dH}{dt}+\frac{dQ}{dt}+\frac{fQ\,|\,Q\,|}{2DA}=0$$

C+

$$\frac{dx}{dt} = +\ a$$

$$\frac{-gA}{a}\frac{dH}{dt}+\frac{dQ}{dt}+\frac{fQ|Q|}{2DA}=0$$

C-

$$\frac{dx}{dt} = -a$$

and finally into algebraic equations

C+: Hp = $(\frac{a}{gA}Q_A+H_A)-\frac{a}{gA}Q_P-\frac{f\Delta t}{2DA}Q_A.|Q_A|$

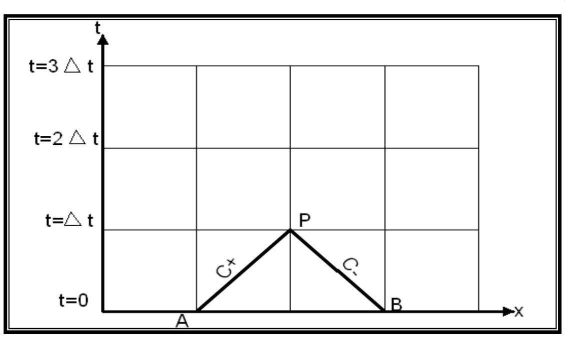
C-: Hp = $\frac{a}{gA}Q_P+\left(H_B-\frac{a}{gA}Q_B\right)+\frac{f\Delta t}{2DA}Q_B.|Q_B|$

H_P = $C_P + BQ_P$

H_P = $C_M - BQ_P$

Where B= $\frac{a}{gA}$ C_P= $H_B-BQ_B+\frac{f\Delta t}{2DA}.Q_B.|Q_B|$

C_m= $H_A+BQ_A-\frac{f\Delta t}{2DA}.Q_A.|Q_A|$



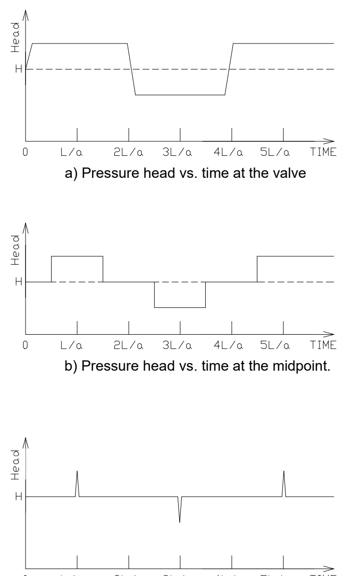
Pressure Surge Analysis

Equation of motion in pipe

$$h = \frac{L}{gA}\frac{dQ}{dt} + \frac{fLQ.|Q|}{2gDA^2}$$

$$Q_S = A_S \frac{dH}{dt}$$

Equations have to be solved using numerical technique.



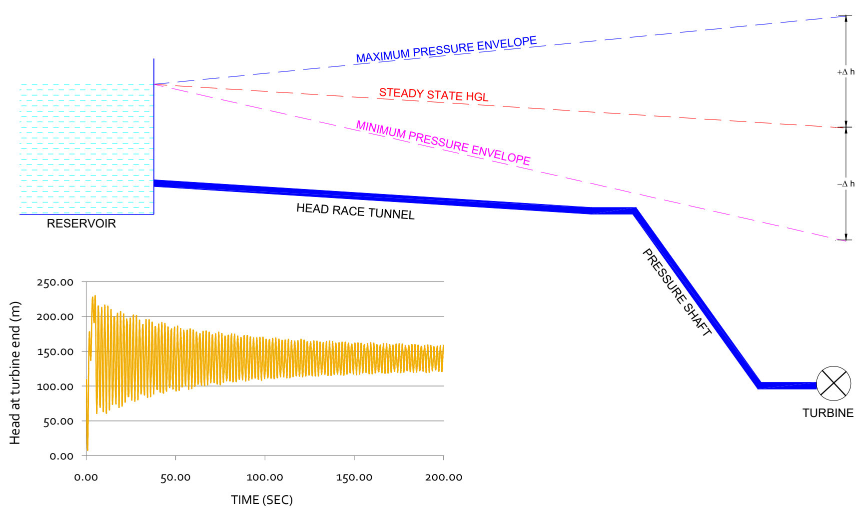
a) Pressure head vs. time at the valve

b) Pressure head vs. time at the midpoint.

c) Pressure head vs. time just inside the pipe at the reservoir.

Figure 2-3. Pressure head vs. time at three locations along the pipe.

Pressure Variations During Transients



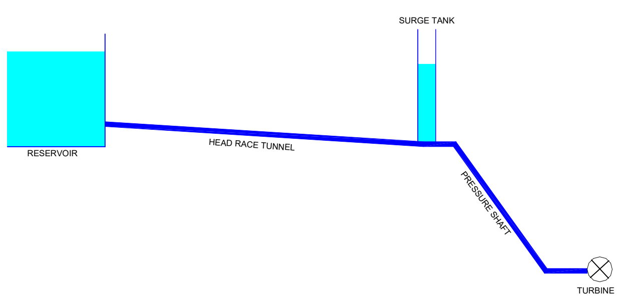
Pressure Rise at Turbine

- For $T_c > 2L/a$ $\Delta H = \frac{2.L.V}{g.T_c}$
- To control speed rise of turbines, maximum pressure rise at turbine is restricted
- Permissible pressure rise
  - Francis turbine 35% - 40%
  - Pelton turbine 15%- 20%
- What if
  Actual Pressure Rise > Allowable Pressure Rise ???

Surge Tanks

Need of Surge Tank

- To control pressure rise, surge tank is provided in the water conductor system, between turbine and reservoir
- To minimise the pressure rise, surge tank shall be provided as close to the Power House as possible



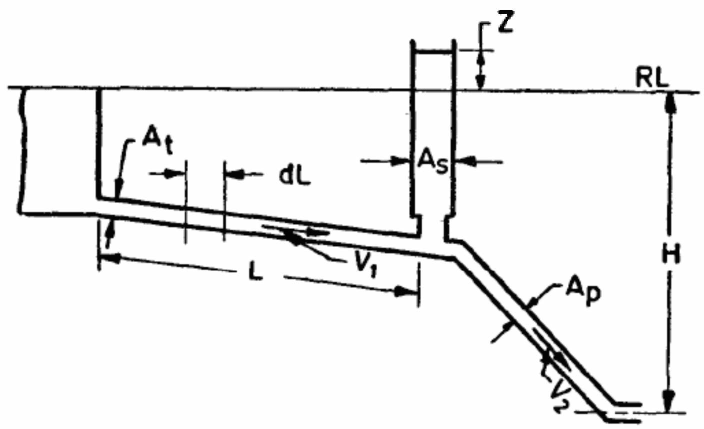
Functions of Surge Tanks

- Pressure and speed regulation in turbine
- Flow regulation
- And to protect tunnel upstream of surge tank from water hammer pressures

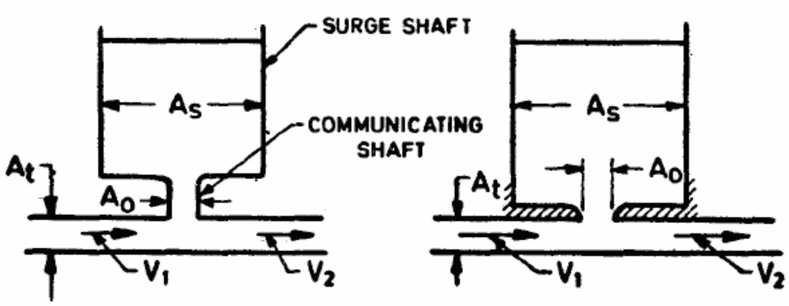
Types of Surge Tanks

- Simple
- Restricted Orifice
- Differential
- Surge tank with expansion galleries
- Air cushion surge tank

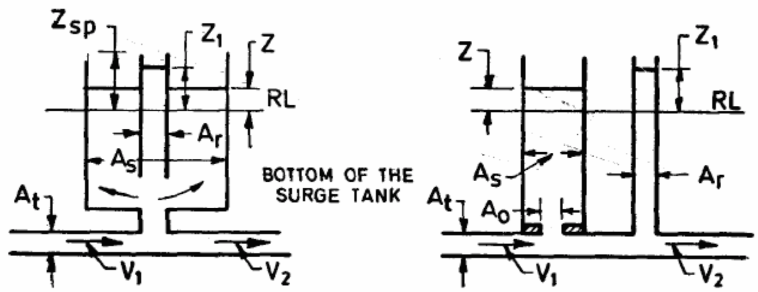
Simple Surge Tank



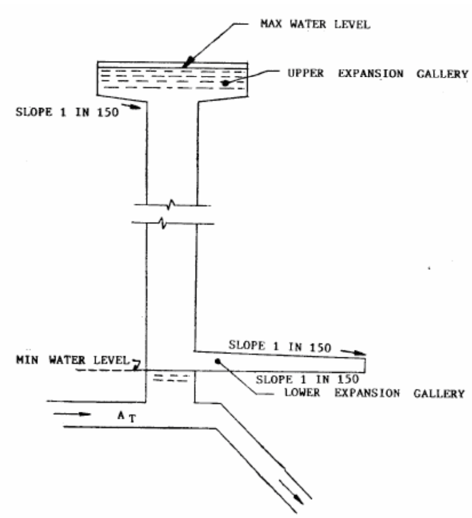
Restricted Orifice Surge Tank



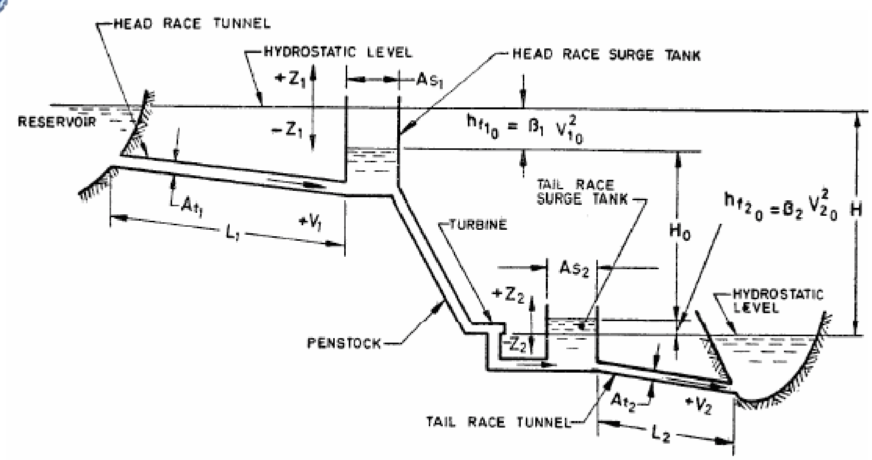
Differential Orifice Surge Tank



Surge Tank with Expansion Galleries



Head and Tail Race Surge Tanks



Hydraulic Design of Surge Tank

- For stability of oscillations, Thoma criteria should be satisfied

$A_s > A_{th}$

$$A_{th} = \frac{L.A_t}{h_f.H_o} \cdot \frac{V_1^2}{2g}$$

L = Length of tunnel

A_t = area of tunnel

h_f = head loss in tunnel

V_1 = Velocity in tunnel

H_o = Net head on turbine

Pressure Surges Analysis

- To know water level oscillations in surge tank, surge analysis can be carried out separately for reservoir-HRT-surge tank system

  Equation of motion in pipe

  $$h = \frac{L}{gA}\frac{dQ}{dt} + \frac{fLQ.|Q|}{2gDA^2} \; ; \qquad Q_S = A_S \frac{dH}{dt}$$

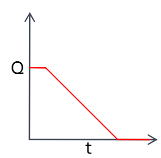
- Equations are solved using numerical technique.
- Generally, transient analysis of complete water conductor system is carried out to know pressures at different points including water level oscillations in surge tank

Maximum Upsurge

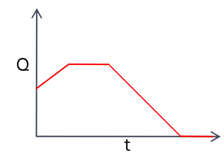
- Surge tank shall be designed to accommodate maximum and minimum water levels anticipated under worst conditions
- To obtain maximum upsurge level the worst of following two conditions shall be considered
  - Full load rejection at FRL (100 – 0 – 0)
  - Specified load acceptance followed by full load rejection at the instant of maximum velocity in HRT at FRL (66 – 100 – 0)
- Lower value of coefficient of friction shall be taken during analysis

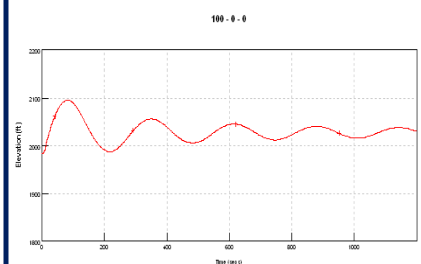
Maximum Upsurge

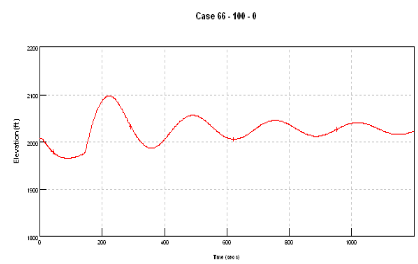
Condition 100 – 0 – 0



Condition 66 – 100 - 0





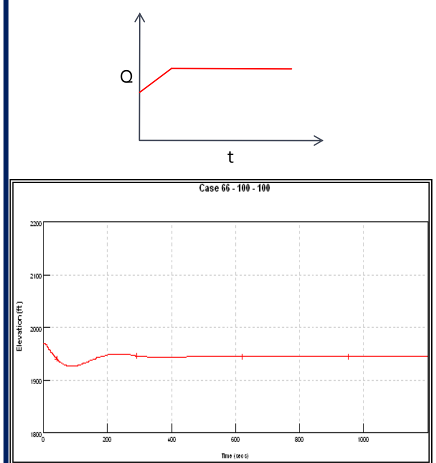


Maximum Down Surge

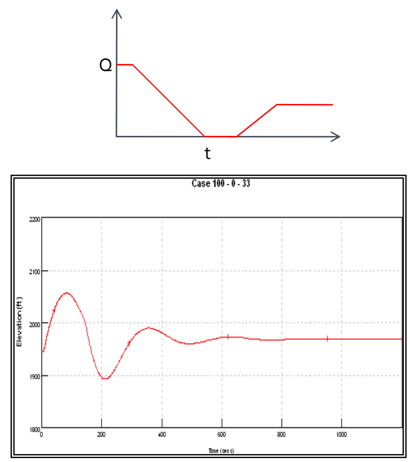
- To obtain minimum down surge level, the worst of following two conditions shall be considered
- Specified load acceptance at MDDL (66 – 100 – 100)
- Full load rejection at MDDL followed by specified load acceptance at the instant of maximum negative velocity in HRT (100 – 0 – 33)
- Higher value of coefficient of friction shall be taken during analysis

Maximum Down Surge

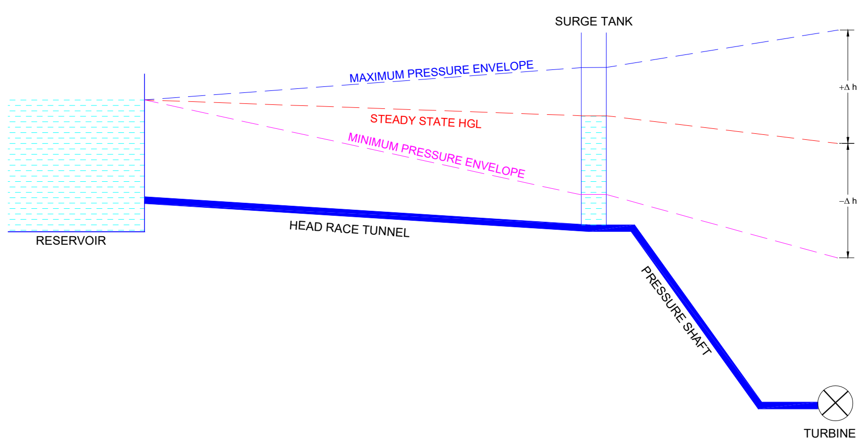
Condition 66 – 100 – 100



Condition 100 – 0 - 33



Pressure Envelopes



Orifice Size in Restricted orifice Surge Tank

Head loss through orifice is given by

$$h_{or} = \frac{Q_o^{\,2}}{Cd^2 . A_o^{\,2} . 2g}$$

Where

Q_o = Maximum discharge through turbines

A_o = area of orifice

Cd = Coefficient of discharge (varies from 0.6 to 0.9)

Area of orifice is so chosen as to satisfy the condition of Calame and Gaden for maximum flow

$$\frac{Z^*}{\sqrt{2}} + \frac{1}{4} h_f \le h_{or} \le \frac{Z^*}{\sqrt{2}} + \frac{3}{4} h_f$$

$$Z^* = V_o \sqrt{\frac{L}{g} . \frac{At}{As}}$$

Hydraulic Design of Tail Race Surge Tank

For stability of oscillations, Thoma criteria should be satisfied

As > Ath_{ds}

$$Ath_{ds} = \frac{L . At}{hf . H_o} . \frac{V^2}{2g}$$

L = Length of tail race tunnel

At = area of tail race tunnel

hf = head loss in tail race tunnel

V = Velocity in tail race tunnel

Ho = Net head on turbine

Head &Tail Race Surge Tanks

1. Calculate Thoma Areas for U/s and D/s Surge Tanks

$$Ath_1 = \frac{At_1 \cdot L_1}{2\ g\ \beta_1\ H_0}$$

$$Ath_2 = \frac{At_2 \cdot L_2}{2\ g\ \beta_2\ H_0}$$

2. Calculate As_1 and As_2 by assuming C_t

$$As_1\ min = (1 + C_t r)\ Ath_1$$

$$As_2\ min = \left(1 + \frac{C_t}{r}\right) Ath_2$$

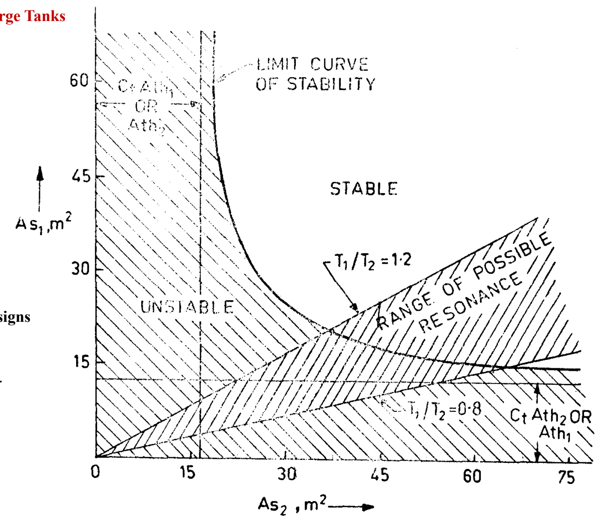
$$r = \frac{As_1\ min}{As_2\ min}$$

C_t may be assumed as 0.7 for preliminary designs

3. Calculate ratio of time periods T_1/T_2

$$\frac{T_1}{T_2} = \sqrt{\frac{L_1\ At_2}{L_2\ At_1} \times \frac{As_1}{As_2}}$$

To avoid resonance, $1.2 < T_1/T_2 < 0.8$



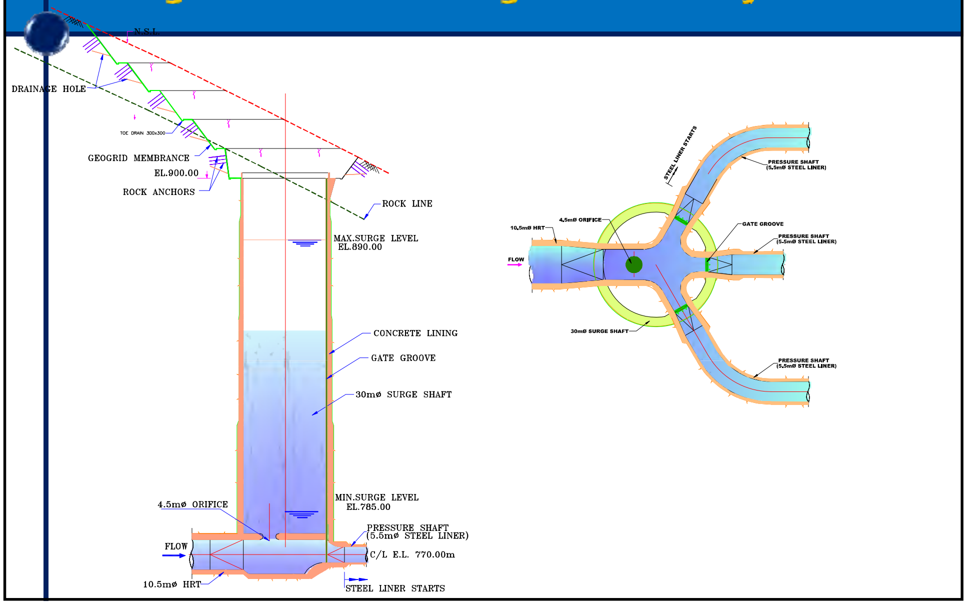
Structural Design of Surge Shaft

- Lining should be designed for internal pressure
- Lining should be safe under external pressure when shaft is empty

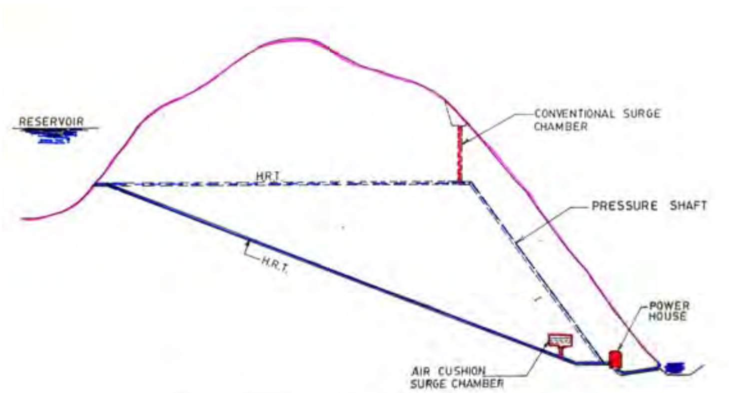
Internal Pressure = Corresponding to maximum water level

External pressure = Higher of existing water table or saturation level in rock around Surge shaft corresponding to static water level in surge shaft.

Surge Shaft of Punatsangchhu II HE Project

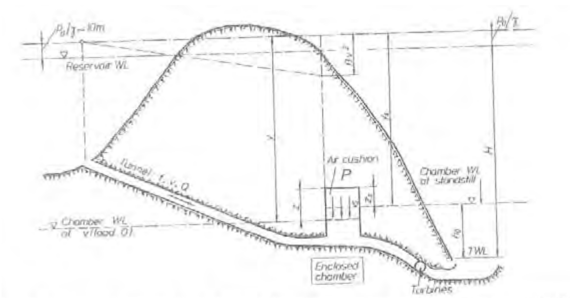


Air Cushion Surge Chamber



USE OF COMPRESSED AIR CUSHION SURGE CHAMBER
AS COMPARED WITH A CONVENTIONAL DESIGN

Air Cushion Surge Chamber



15/85. Air-cushion enclosed surge chamber. Absolute air pressure at standstill: $p_s = y_{s0}$ at steady state v velocity: $p = (y_s - z_s + z - u_n^2)$

Air Cushion Surge Chamber

- A pressure vessel with a back up compressor that maintains the volume of gas in the unit to cater for pressure changes
- The air follows the reversible polytropic relation

  $$H\,V^n = \text{Constant}$$

  (for isothermal process, n = 1.0 & for adiabatic process, n = 1.4)
- Size of air volume to control maximum pressure and the size of liquid volume to avoid emptying the tank is determined by transient analysis.
- For stability, minimum area of chamber shall be

  $A = n.H.A_{th}$ where H is absolute pressure head

Advantages of Air Cushion Surge Chamber

- Can be installed close to turbines
- The water hammer pressures induces by sudden changes in load are small as the penstock length from air chamber to turbines is less.
- Length of water conductor system reduces.

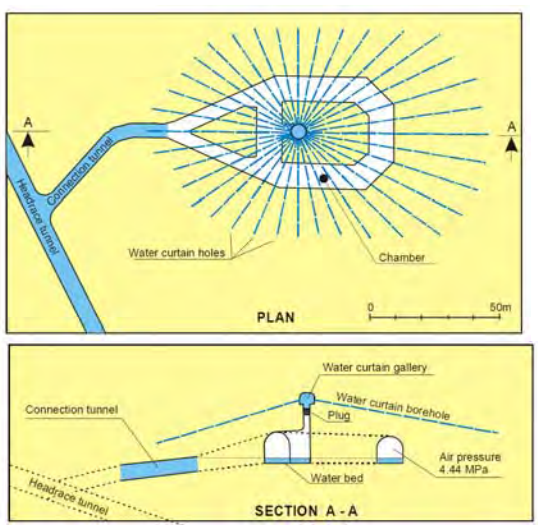
Practical Aspects

- The air cushion surge chambers require an air compressor to compensate for possible air leakage
- Air may get dissolved in turbulent water mass and the chamber may prove to be a source of cavitation damage to the machine
- The air in the chamber may slowly become poisonous (low oxygen content, possible content of H2S from the deposits of organic material). This will in no way damage the machine but care should be taken that the gases form chamber are not cleared through the machine hall.

Practical Aspects

- The chamber does not completely reflect the water hammer waves.
- Unlike conventional surge tanks, air cushion surge chambers need regular monitoring of air volume & pressure.
- To avoid air leakage, surrounding rock mass needs to be properly grouted and SFRS / concrete lining is generally required.

Air Cushion Surge Chamber At Torpa Hydropower Plant



Thank You

SURVEY AND INVESTIGATION FOR PLANNING OF HYDROPOWER PROJECTS

S K Das
Director, HCD(NW&S) Dte.

OBJECTIVE

- Assessment of Feasibility of Project.
- Finalize the location of components - To evaluate and optimize the layout of the project.
- Data for designing the project components - To collect sufficient quantitative and qualitative geological and geotechnical information.
- Data to plan suitable construction methodology.

As per BIS 12182:1987, the planning of Hydro-electric projects is done in such a way that full service life of the project should not be less than 25 years and feasible life shall not be less than 70 years.

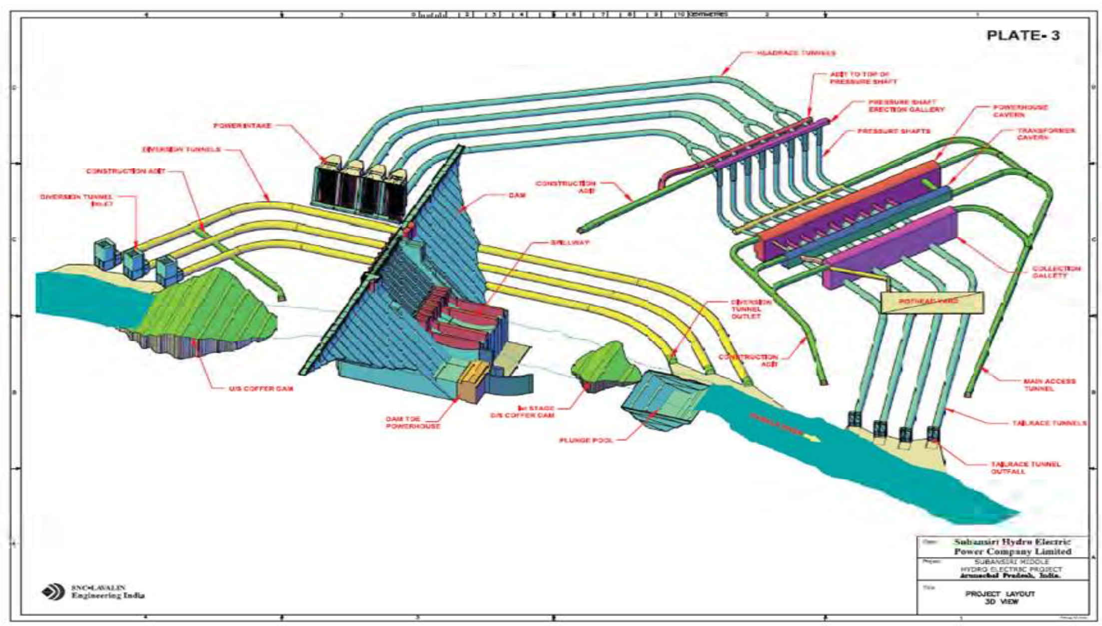
- Survey and Investigation of the proposed hydro power project site is the preliminary requirement that needs to be met by using state-of the-art scientific tools and techniques
- Hydropower Project completion time is estimated approximately 6-8 yrs from inception. But, generally it takes 10 -20 yrs in completion.
- The process of preparation of Detailed Project Report shall be completed in a period of 30 months from inception of the project.
- Construction period is generally taken 5 yrs.

Importance of Survey & Investigation

- Delays are very common in hydropower projects due to various reasons which produces cost overrun of the project in lacs per day.
- A large number of HE projects are delayed due to geological surprises.
- Geological surprises take place in hydro projects mainly due to inadequate investigation.
- However, geological surprises cannot be completely ruled out even after adequate investigation.
- Thorough survey and investigation would minimise geological surprises and delay of the project may be minimised and thus cost overrun also.

The main components of a typical hydroelectric scheme are:

- Diversion structure dam/weir/trench weir, etc.
- Intake
- Desilting basin
- Water conductor system
- Surge shaft
- Pressure shaft/penstock
- Power house
- Tail race system



DETAILED PROJECT REPORT (DPR) Stage S & I:

After establishment of the feasibility of the Project, DPR stage Survey & Investigation is done and included in the Inception Report. The Inception Report is submitted for approval of concerned chapters by CWC/CEA/CSMRS/GSI. All these approved chapters are included in the DPR.

Survey

Types of Survey

- **Topographical surveys** of river, reservoir, head works, colony layout, head race tunnel/channel, power house, switchyard, surge shaft, tail race tunnel/channel, adits, penstock etc. considering different water levels Extent of surveys, scales and contour intervals for various components. Detailed topographic surveys of the project area, reservoir area, quarry & burrow areas, infrastructure and muck disposal areas & river cross section and power house area.Remote sensing studies and satellite imageries also provide land use pattern, geo-morphological features and drainage system of the basin.
- **Archaeological surveys** in the reservoir area.
- **Mineral surveys** in the catchment areas.
- **Right of way surveys** for the reservoirs. These shall cover survey for right of approach roads, which may be claimed by owners to various structures above FRL.
- **Communication surveys**
- **Geology & geo-technical**
- **Geophysical**

  Geophysical methods are employed as an aid to geological investigations for assessment of in-situ conditions and engineering properties of the rockmass mainly by using seismic and electrical methods. These methods provide subsurface information which include depth of overburden, depth and quality of rockmass, major faults, folds, dykes and water saturation conditions.

- **Seismicity**
  Regional and local seismicity is to be discussed with relevant plates. Site specific seismic parameters are to be determined. MEQ studies and active fault studies are to be conducted, if required.
- **Foundation investigations** of different structures/components of the project indicating boreholes details, soil/rock strata etc.
- **Construction materials** survey
- **Hydrological and meteorological** River gauging stations on the river and the tributaries.
- **Ecological and Environment** including wild life, fish culture etc.

TOPOGRAPHICAL SURVEY

River Surveys

L-Section (1:10,000 horizontal scale and 1:100 vertical scale) upto MWL+5m or to a point up to which the back water effect is likely to extend, whichever is less, from the dam axis in the upstream direction and 10 km downstream from the axis giving HFL, deep pools, rapid rock outcrops etc.

Dam site

Dam site are covering up to 500m u/s and 500m d/s of the dam axis extending up to an elevation of dam top + ¼ of dam height, depending on the geological requirement and slope stability including abutment stripping. The scale of the maps may vary from1:500 to 1:2000 depending upon the size of the area. The contour interval should be 1m to 5m depending on the topographical characteristics of the valley.

Survey for tunnel

- The project area maps are used for initial geological mapping and fixing of alignment of tunels, adits etc. in consideration with the required rock cover.
- For detailed studies and layout finalization 100 m to 400 m wide strip along the tunnel alignment in a scale of 1:2000 to 1:5000 is considered.
- Survey for Adit portals may be carried out in a scale of 1:200 in 50 M width on either side of Adit alignment.
- Adit portal may be located in areas where sufficient space is available for provision of infrastructure facilities for the works.

Other Surveys

Generally for power house, switch yard,construction lay down area, quarries, surge shaft area, generally topographical maps on a scale of 1:1000 to 1:2000 are considered adequate depending upon the size of area

Project area map

- If the project area is spread upto 5 km, a scale of 1:5000 is adequate.
- For project spread upto 10 km, normally a scale of 1:10,000 is considered and for bigger area, a scale of 1:15,000 is adequate.

Table 1.
(CWC guidelines for DPR preparation SURVEYS: Extent, Scales, Contour intervals, etc.)

Sl.No	Description	Area to be covered / Extent of Surveys	Scale		Contour Interval	Remarks
			Horizontal	Vertical		
1.	River Surveys L- section	Upstream L-Section upto MWL +5m or to a point up to which the back water effect is likely to extend from the axis of the structure, whichever is less . In case of any head works situated upstream within MWL + 5 m or the farthest point affected by back water. Downstream 10 km from the axis of the structure	1:10000 1:10000	1:100 1:100		Leveling at 50m or less interval. Maximum historical observed HFL Deep pools and their bed level, rock outcrops. -do-
2.	Reservoir	i) Contour plan covering an area upto an elevation of MWL+5m	1:2000 to 1: 10000 (depending on the total area)	-	1 or 2 or 3 m 5	Contour intervals for, slope less than 10^o to horizontal – 1 m or less, slope 10^o to 30^o – 2 m, and slope more than 30^o – 3 m.

2.	Dam and Dyke	Topographic plan of the site with contours, covering the area upto 4H on upstream and downstream of the axis OR a minimum of 250m on the upstream and 500 m on the downstream of the axis, and extending upto MWL+2H where H is the height of dam.	1:1000	-	1 or 2 or 3 m	As per item 2 above. Leveling to be at least at 10m grids.

Sl.No	Description	Area to be covered / Extent of Surveys	Scale		Contour Interval	Remarks
			Horizontal	Vertical		
1.	Canal and Water Conductor System	i) L-Section ii) Cross-Section at 50 m interval iii) Strip Contour Plan to cover 150 m on either side of the centre line of the canal or depending upon the requirement whichever is more.	1:2000 1:2000 1:1000	1:100 1:100	- - 0.5	Leveling at 50 m or less interval -do- Block leveling on 50 m or less grid basis depending upon the slope of the land.
2.	Power House, Switch Yard, Surge Shaft, Tailrace etc,	Contour plan of the site to cover full area of the component(s) alternative layouts.	1:1000	-	0.5 or 1 or 2 or 3 m	Contour intervals for, slope less than 10^{o} to horizontal – 1 m or less, slope 10^{o} to 30^{o} – 2 m, and slope more than 30^{o} – 3 m. Block levelling on 50 m or less grid basis depending upon the slope of the land.
3.	Tunnel & Adit	i) Contour Plan of the area covering the length of the tunnel & 500 m on either side of the centre line of the tunnel/adit including approach, portal and dump areas. ii) L-Section	1:1000 to 1:10000	-	1 or 2 or 3 m	Contour interval as per Item 2 above. Block leveling as per Item 1 (iii) above in case of ground surveys. Vertical scale depending upon steepness of the slope and drop
4.	Penstocks	i) Contour Plan of the area covering the length of the structures and 150 m on either side of the centre line of penstocks.	1:1000 1:1000	- 1:100 or 1:200 or 1:500 or 1:1000	1 or 2 or 3m	Contour interval as per Item 2 above. Block leveling as per Item 1 (iii) above. Vertical scale depending

		ii) L-Section				upon steepness of the slope.

Hydrological & Meteorological Survey

These surveys are carried out to establish

- Rainfall
- Gauge
- Discharge
- Sediments
- Water quality
- Evaporation
- Availability of water for the benefits envisaged
- Design flood for various structures

Hydrological studies

- Gauge and Gauge and discharge observation data for at least 10 yrs at project site.
- Sediment data for the period of minimum 3 yrs for suspended load, bed load and catchment characteristics.
- Chemical and petrographic analysis of river water.
- Design flood, diversion flood, sedimentation studies, Tail water rating curve, Area capacity Curve, design storage and key reservoir levels for each site.

Hydrological Data required for civil design

- Design flood discharges at locations of diversion structure and confluence of tailrace & river.
- Design discharge in power Intake
- PMF, FRL, MDDL and TWL
- Sediments data (size, shape and concentration) for dead storage of reservoir, design of silt flushing arrangement and turbine

Environment and Forest surveys

These surveys / studies are carried out on the following aspects

- Environmental survey
- Forest area involved
- Wild life
- Likely displaced persons
- Environment impact assessment
- Environment management plan

INVESTIGATION

GEOLOGICAL AND GEOTECHNICAL INVESTIGATION

To evaluate:

(a) Ground water conditions

(b) Engineering properties of the overburden and rocks.

(c) Stability of slope adjacent to the excavations.

(d) Landslides, rock falls and avalanches.

(e) Groutability of the geological medium.

(f) Liquefaction and settlement aspects.

Regional Geology, Geomorphology and tectonics of the project area and its vicinity.

The site specific **geological mapping** is prepared from existing GSI map and by intensive surface traverses of the project area and also with the aid of aerial photographs & satellite imagery, for coverage of inaccessible area comprehensively.

Geological mapping required to:

- Assess the physical and structural characteristics of the rock mass.
- Delineate rock overburden contact.
- Collect geotechnical parameters for rock mass characterization.

Surface mapping and sub-surface geological exploration to provide information regarding;

- Types of soil and rock mass
- The location, sequence, thickness and aerial extent of each soil/rock stratum, including a description and classification.
- The depth and type of bedrock as well as the location, sequence, thickness, aerial extent, altitude, depth of weathering, soundness, and description of rock in each rock stratum within the depth of exploration.
- Joints, weakness zones, shear zones, folds and faults and their trends.
- Likely existence of hot or cold springs, presence of any gas.

LABORATORY AND IN-SITU TESTS AS REQUIRED

LAB TESTS ON SOIL SAMPLES	**LAB TESTS ON ROCK SAMPLES**	**IN-SITU TESTS**

Manual examination **Natural moisture content** **Liquid and plastic limits** **Specific gravity** **Particle size distribution** **Bulk density** **Permeability** **Consolidation characteristics** **Swelling test** **Strength characteristics** **Compaction test** **Relative density** **Minerological composition** **Chemical analysis**	**Unconfined compressive strength** **Modulus of elasticity** **Specific gravity and porosity** **Water absorption** **Slake durability index** **Shear and ultrasonic velocities** **Shear strength of infilling material** **Swelling** **Joint stiffness** **Petrographic study**	**Shear strength parameters** **Modulus of deformation** **Seismic wave velocity** **Bearing capacity test** **Permeability test** **In-situ stress** **Blasting test** **Vibration study** **Groutability test**

- To determine the geotechnical parameters like unconfined strength, shear strength, bearing capacity, permeability, water tightness of rock, principal stress, modulus of elasticity, poisson ratio, deformation modulus etc for designing dam, tunnel, underground cavern,adits and slope protection work.
- To know the bedrock configuration and rockmass condition of the foundation media.
- To determine characteristics of foundation material.
- To know the nature of riverine material, identify liquefiable zones (if any) present below the foundation.
- To out out the presence of huge boulder/rock blocks as fluvio-glacial deposit/ landslide debris etc
- To work out the support system along the cut slope for the dam foundation on both the banks.
- To know the details on bedrock configuration, rockmass condition and depth of overburden/ cutoff along axes etc. of both u/s and d/s cofferdam.
- To assess bedrock depth, nature of bedrock .
- To assess in situ stress
- Orientation of the tunnel with respect to regional strike and weak zones of the rock formation, maximum and minimum cover over tunnel alignment, joint sets, low cover and highcover zones, weak/shear/fault zones likely to be encountered, vertical and lateral rock covers at all nala crossings etc
- consolidation and curtain grouting.
- To assess the rockmass condition, depth of grout curtain and detect any adverse features like fault/shears etc below the dam body that may endanger the stability
- To know the efficacy of grouting as well as to determine spacing, orientation of grout holes for

Exploratory Drilling (Bore holes):

- Nos. and depths of drill holes for different components required as per CWC guidelines.
- Assessment of depth of overburden and quality of bedrock.
- Permeability tests for overburden are done in drill holes.

- Groutability test is done to determine the groutability characteristics at the site.
- Geological logging of Drill holes by geologist.

Preparation of Test Specimens

- The rock core & boulder specimens for various laboratory tests were prepared in accordance with relevant provisions of IS: 9179-1979. Core specimens of 54mm dia were drilled out from rock boulders for testing purpose.
- Rock cores of 54/42 mm dia. were cut to proper length, meeting the requirements of the requisite length/diameter ratio and their ends were planed and polished using polishing and lapping Machine

Grain Density

- The test is conducted as per IS: 13030-1991 "Method of Test for Laboratory Determination of Water Content, Porosity, and Density & Related Properties of Rock Material".
- The rock core samples were crushed and ground to a grain size not exceeding 150 microns.
- Three representative specimens of about 15 g each of the pulverized material were used for evaluation of grain density. Test results based on average of 3 tests.

Point Load Strength Index

- The test was conducted as per IS: 8764-1998 "Method for the Determination of Point Load Strength Index of Rocks". The cylindrical cores were tested axially keeping the length to diameter ratio of 0.30 to 1.0. The load was applied to the specimen such that failure occurred within 10-60 seconds and the failure load P was recorded

Elastic Parameters(Modulus of Elasticity & Poisson's Ratio)

The test is conducted as per IS: 9221-1979 "Method for the Determination of Modulus of Elasticity & Poisson's Ratio of Rock Materials in Uniaxial Compression". The test specimens comprised of right circular cylinders with length to diameter ratio of 2. Axial and circumferential deformations were determined using data obtained by electrical resistance strain gauges. Tangent Modulus and Poisson's Ratio were determined at 50% of the ultimate stress.

Triaxial Compression Tests for determination of material constant 'mi'

The test was carried out as per IS: 13047-1991 "Method for Determination of Strength of Rock Materials in Triaxial Compression". The test specimens comprised of right circular cylinders with length to diameter ratio of 2. The rock core specimens were tested at different confining pressures.The purpose of conducting triaxial test was to determine the material constant mi for intact rock as

Geophysical Explorations:

- Seismic refraction surveys to ascertain the depth and type of overburden and to assess the quality of subsurface strata.
- The guidelines for site specific studies as outlined by National Committee on Seismic Design Parameters (NCSDP) may be adhered.
- Site specific seismic parameters are finalized (CWPRS)

Construction Stage S &I

- To assess the actual parameters of specific component and to carry out required modification in Design.
- Large scale foundation grade mapping for dam, progressive 3D geological logging of ongoing tunnels, probe holes on the tunnel face etc are carried out during this stage.
- Foundation grade geotechnical mapping of earth & rockfill dam is done on 1 :500 scale
- 3-D geological logging of diversion tunnel is carried out on 1: 100 scale.For Concrete Dam, Power House Excavation, Tunnel Excavations, foundation grade geotechnical mapping and geological logging is done on 1 :100 scale

Table-2

MoWR/CWC Guidelines for Preparation of Detailed Project Reports

(a) Earth and Rock-fill Dam

Minimum Pattern of Drilling

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts

(i) Drill holes along the axis 150 m or less apart, with intermediate pits to delineate weak and vulnerable strata with a minimum number of 3 to 5 holes in the gorge portion and additional two on each abutment parallel to the flow. Drift on each abutment at about 60 m elevation interval at a minimum of one each on each abutment.	Depth equal to half the height of Dam at the elevation of the hole or 5 m in the fresh rock whichever is less. About two holes to be extended deep (equal to the maximum height of the dam in the absence of rock at higher elevation or 5 m in fresh rock whichever is higher), in the gorge portion and one each in abutments. Drifts to be extended 5m in geologically sound strata for keying the dam in the absence of rock.

(b) Masonry & Concrete Dam
Minimum Pattern of Drilling

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts
(i) Drill holes along the axis at 100 m interval or less apart to delineate weak and vulnerable strata with a minimum number of 3 to 5 holes in the gorge portion and additional two on each abutment parallel to the flow. (ii) 2-3 drill holes down stream of spillway. (iii) Drifts on each abutment at about 60m elevation interval with a minimum of one on each abutment.	10 m in the fresh rock (proved by geophysical or any other suitable method). About two holes to be extended deep (equal to the maximum height of the dam in the absence of rock at higher elevation), in gorge portion and one each in abutments. 10m in high rock or equal to maximum height of dam in absence of rock. 10 m in the fresh rock (proved by geophysical or any other suitable method).

(c) Tunnels

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts
(i) Drill holes one at each of the portal and adit sites and additional at least one every 1-5 km interval depending upon the length of the tunnel (ii) Drifts on each abutment at about 60m elevation interval with a minimum of one on each abutment.	Drill holes 5-10m below the tunnel grade of maximum possible depth 10 m in the fresh rock or upto tunnel face.

(d) Barrage and Weirs

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts
(i) Drill holes along the axis 150 m or less apart, with intermediate pits to delineate weak and vulnerable strata with a minimum number of two additional holes on each abutment parallel to the flow.	Drill hole 1.5-2 times maximum head of water below the average foundation level or 5m in the fresh rock whichever is less. Rock to be proved by geophysical or any other method.

(e)Power House

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts
Two to four or more drill holes and/or drifts covering the area to satisfactorily portray the geological condition and delineate weak and vulnerable zone, if any.	Drill holes one to two times the maximum width of the structure or 5-10 m in the fresh rock proved by geophysical or any other method whichever is less. For underground Power house the strata shall be examined by the explorations with adequate number of drill holes. If found feasible and necessary accordance to the site conditions, one drift with cross cuts may be excavated at the roof level to prove fresh rock conditions along the length and breadth of the cavity structure.

PUMPED STORAGE SCHEME

It is another viable alternative to meet the peak demand. Energy is required to pump water from lower reservoir to an upper reservoir. So operation of a pumped storage scheme is possible only when off-peak surplus power is available in the system.

In this type of development water is lifted from lower reservoir into an upper reservoir using surplus energy during low demand and is released from the upper reservoir during high demand to generate power. Either separate turbine and pump are used or a reversible turbine which can act as a pump also may be used. Reversible reaction turbines upto a head of 500 m are available. The efficiency of pumped schemes is low (of the order of 67 %) hence this peak power is costly.

To find suitable sites for two reservoirs is an important factor in planning such a scheme. The principal types of pumped storage schemes developed so far can be grouped into three groups.

(i) Recirculating type

(ii) Mixed or multi use type

(iii) Water transfer type.

These are shown in enclosed figure. Type II are more common. Tehri, Sardar Sarovar, Nagarujan Sagar, Puralia projects are under construction. The pumped storage potential in India is estimated as 93000 MW. Power plants for 4000 MW are under construction. A Detailed note on pumped storage development is annexed.

ANNEXURE – I

The advent of an ever-increasing number of large base-load generating units and the growth of high peak load demand has greatly increased the need for flexible peak load generating capacity. This peak load generating capacity could be provided by either quick start generating units such as combustion turbines, combined cycle units, diesels, and small common header compressed air, and batteries.

Another effective alternative is to convert low cost surplus off peak electric power into peak power. Pumped hydro is one of the methods to do this conversion, although it is relatively expensive to install and the number of suitable sites is also limited.

The use of hydro pumped storage facilities to meet peaking capacity has been growing at an increasing rate in some countries. In addition, these schemes can contribute to the overall electric power system by performing the function of load regulation, quick response reserve capacity to off-set short term generation or transmission outage and increase overall system energy.

Because of their high effectiveness in utilization of power system, large capacity, high head, high speed, pumped storage power plants (PSPP) are now being constructed in the world. It is also gaining momentum in India.

Definition of Pumped Storage:

Pumped storage scheme is a combined pumping and generating plant; hence it is not a primary producer of electrical power but, by means of the conversions, stores the surplus power of the network and returns it in peak load periods.

In this scheme water generates power during peak demand, while the same water is pumped back in the reservoir during lean demand period. The provision is based on economics of operation and the availability of enough spare capacity in the grid to operate the machines as pump in the low load period.

Types of Pumped Storage Scheme

a. **Storage or Re-circulating type**: water from reservoir A (Fig a) is used to produced power at station P and the discharge through tailrace is stored in reservoir B. During lean periods, the water from reservoir B is pumped back to A which later produces energy in time of need.

Fig. a

b. **Pondage or Multi-use type:** in addition to the pumping back of reservoir B to reservoir A for producing peak power at station P_1, some water of reservoir B is also used to produce power at station P_2 in the downstream (Fig b). Release from P_2 will meet other water needs in the downstream.

Fig b

c. **Part Head or Water Transfer type:** water of reservoir B is pumped to an elevated reservoir A in another valley. The water of reservoir A is utilized to produce power at station P in a third valley (Fig. c).

Fig c

Here we consider the type of Multi-use Pumped Storage, It means there are some release from down stream reservoir to meet the requirement of several other interests. This type of pumped storage schemes is more common.

Role of Pumped Storages Schemes

The role of pumped storage schemes in a power system is described in the following paragraph. These have all the merits of a hydropower station plus some additional advantages as is evident from the following.

Most Efficient and Practical means for Storing Large Quantity of Energy

Pumped storage hydroelectric schemes have proved technologically the most attractive method of storing excess system thermal energy during off peak hours and returning it to the system during peak hours. Although net extra energy production is not involved in pumped storage schemes, the conversion of otherwise excess night energy into high productive peak energy has provided the economic base for these schemes. Thus these schemes provide the most efficient and practical means for storing large quantity of energy. These plants spread in value between base power and peak power.

Reduction in system Minimum Loading

During off peak hours, when the load is low, some of the base load units may have to run at part load. If the minimum load is low enough, the sum of minimum operating levels of the committed base load units may still exceed the minimum load level. In this situation, the base load energy will be dumped or is forced to sell to the neighboring utilities at very low rates. If pumped storage scheme is available in the system, this dump energy can be used for pumping operation. Even if the system does not need that stored energy during the peak, having stored that energy allows the utility to sell it on peak to utilities at a profit.

Improvement in Efficiency and Economy of the Thermal Power Stations

The operation of thermal and nuclear power plants at very low load (during off peak period) would adversely affect the efficiency of the power system, power plant life and fuel consumption per KWh generated. Thus increasing number of large base load thermal and nuclear power plants, which cannot be operated at low loads due to economic reasons would need construction of pumped storage power projects. The pumped storage schemes improve the plant load factor, efficiency and durability of thermal units and reduce severe cycling of these units and hence they improve the operational performance of thermal power plants. These allow thermal generating units to operate at nearly constant output all the time and at the best efficiency instead of ramping up and down for the daily load cycle. This translates into lower thermal and mechanical stresses on the boiler and turbine. Reduction in wear and tear on an expensive base load unit can prolong its life, lower its maintenance costs, and improve its availability. All these benefits can result into significant cost savings. Frequent starting and stopping of thermal generating unit's results in the economic losses as well as adverse effects of the equipments. Thus the pumped storage hydroelectric schemes help in improving the operational and economic performances of thermal plants by creating favorable conditions in the power system, which facilitate base load operation of thermal/nuclear power plants.

Spinning Reserve Benefits

Spinning reserve benefits come from the fact that pumped storage schemes have a fast response time. Spinning reserve provided by thermal units is costly because it forces a thermal unit to be operated at part load, which is inefficient, when pumped storage system is not generating at full load, its unused generating capacity can be used to meet the utility's spinning reserve requirements. Efficiency for pumped storage is not as sensitive to part load as that of thermal plant. Even if the pumped storage units are not generating, as long as there is water in the upper reservoir, the full rating of storage generating capacity is credited towards spinning reserve.

Thus, these schemes provide system-spinning reserve at no cost by operating the installation at partial load. When operated in this manner, the PSPP in many cases can achieve overall power system savings by redacting the portion of the required spinning reserves assigned to operating units and standby in steam electric plants. These schemes serve extremely well with their reserve capacity to maintain the power supply in case of failure of thermal and nuclear generating units or in case where there is an unanticipated high demand for electric power due to the availability to come rapidly on line at full load from no-load synchronized spinning reserve in as a little as 15 seconds.

Power Generation Load Leveling

As there may be severe fluctuations in the daily power consumption patterns, it has become increasingly necessary to optimize the various type of electric power generation in order to achieve the most cost efficient supply of electricity. Pumped storage hydroelectric schemes can make this possible by utilizing power from thermal and nuclear generation facilities at night (off peak hours) to pump water from lower reservoir to upper reservoir, where it can repeatedly be used to generate power during peak demand hours.

In a power system without pumped storage generation, electricity power generation like thermal and nuclear are also primarily to meet peak demand. When the fluctuation in the demand become severe, the repeated start up and shut down required to make the required generation output adjustment result in high fuel costs and lower performance level for these power plants.

In a power system with PSPP, the power needs during peak demand periods can be met by these pumped storage schemes. In this way the system enables to modify the high and low extremes in operating levels of thermal and nuclear power plants, thus facilitating a load leveling effect, which may result in an increased operating efficiency and reduced costs. Therefore, although the pumped storage schemes consume more energy than they generate and do not increase power system energy supply unless there is natural inflow into upper reservoir, these schemes level the peak and valleys of a typical electricity utility load curve and provide emergency power. These schemes create an artificial demand in the power system at low load periods by pumping water from lower pond to upper pond, which is then utilized to generate power to meet peak demands.

Voltage and Power Factor Correction

Voltage and power factor correction are additional operating benefits of pumped storage schemes, when the pumped storage units can operate in a voltage regulation mode like a synchronous condensor and can reduce losses as well as maintain the quality of services to customers.

Improved System Reliability

System operating reliability can be improved by having a pumped storage scheme in the system. The availability of a pumped storage plant is much higher than that of combustion turbines. The speed at which a reversible unit can be brought on line makes it ideal for covering for forced outages of thermal units in the system.

Output Adjustment Capabilities

Another continually growing importance/role of the pumped storage hydroelectric schemes is there availability for providing a quick employable reserve source of power and possibility for enabling load frequency regulations. Electricity is normally supplied at a frequency of 50 Hz. However this frequency is not constant, as it declines when supply capacity falls short of demand and increases in demand when supply is in excess of demand in the power system. Failure to keep supply and demand of the electricity can have an adverse effect on electrical and electronic equipment connected as the loads and may even cause power failures. Therefore the adjustment of generation output in response to demand fluctuations is an important way of ensuring the supply of high quality power at suitable frequency. The important characteristics of PSPP (to reach maximum output within a very short period, say 3 to 5 minutes of start up and adjustment in their output within a matter of seconds) make them more flexible than thermal and nuclear power plants.

Environmentally Friendly

Pumped storage hydroelectric schemes are environmentally friendly. These schemes generate electric power by regulating water between upper and lower regulating reservoir. Once the water has been stored, it can be used repeatedly without changes in water levels downstream, except during the initial filling of the reservoirs. Thus the pumped storage hydroelectric schemes have got very limited, if any adverse effect on environment of the region and help in reducing pollution and others adverse effects which are otherwise likely to results from the development of alternative sources of energy for meeting the peak requirements. Further configuration of pumped storage schemes in mountain regions often provides for underground location of powerhouse and other key elements of the system, thus avoiding disruption of scenery and minimizing the esthetic impact of the new structure.

Other Attributes Offered by Pumped Storage Power Plant

The pumped storage hydroelectric schemes are economically advantageous because they convert low value, low cost, off peak energy to high value, on peak capacity energy and highly flexible peaking power. In addition these schemes are comparatively more flexible from the considerations of selection of their sites. These schemes require much less capital investment. Low investment, ruggedness in nature, easiness in maintenance due to simplicity in mechanism, flexibility in operation and design (short term peaking, spinning or ready reserve), high reliability, and hydrologic independence etc. are other attributes offered by PSPP.

DEVELOPMENT PUMPED STORAGE IN INDIA

Trend of Development

It was Sixth Plan, when the first pumped storage hydroelectric scheme Nagarjunasagar (7 x 100 MW) in Andhra Pradesh was installed in India. First six units of 100 MW each of the project were commissioned in the Sixth Five Year Plan in the existing conventional Nagarjunasagar powerhouse (having one conventional unit of 110 MW already in operation). However, the units of the scheme are being operated as conventional units due to non-availability of the tail pond dam. Paithon (1 x 12 MW) in Maharashtra was the second pumped storage hydroelectric scheme installed in India during the same Plan. Thus, the total Hydro installed capacity under pumped storage schemes was 612 MW at the end of Sixth Five Year Plan.

The seventh unit of 100 MW at Nagarjunasagar pumped storage scheme and all the four units of Kadamparai pumped storage scheme (4 x 100 MW) in Tamil Nadu, aggregating to 500 MW were commissioned during Seventh Five Year Plan. Thus, the total hydroelectric installed capacity under pumped storage schemes was stepped up to 1112 MW at the end of Seventh Five Year Plan.

Kadana Stage I (2 x 60 MW) pumped storage scheme in Gujarat and reversible Panchet Hill, second unit of 40 MW under Damodar Valley Corporation in Bihar were commissioned in 1990-91. Thus the total hydroelectric installed capacity under pumped storage schemes in operation as on 31st March 1992 (at the end of the Annual Plans 1990-91 and 1991-92) rose to 1272 MW.

During the Eight Five Year Plan, two pumped storage scheme, viz. – Ujjani (1 x 12 MW, commissioned), Bhira PSS (1 x 150 MW, rolled on 29th March 1995) and unit of 1 of 60 MW of Kadana Stage II (rolled on 31st March 1996). These on adding raising pumped storage installed capacity to 1494 MW on 31st March 1997.

Future Development

Four sanctioned pumped storage schemes and unit 2 of 60 MW Kadana Stage II in the existing Kadana Stage I PSS in Gujarat with an aggregate installed capacity of 3310 MW are under various stages of construction in India. These schemes are Ghatgar (2 x 125 MW) in Maharashtra, Sardar Sarovar RBPH (6 x 200 MW) in Gujarat, Srisailam LBPH (6 x 150 MW) in Andhra Pradesh, Purulia PSS (4 x 225 MW) in West Bengal.

In addition to these Central Electricity Authority has cleared two pumped storage scheme, viz.- Bhivpuri PSS (1 x 90 MW) in Maharashtra and Tehri Stage II (4x 250 MW) in Uttaranchal with an aggregate installed capacity of 1090 MW.

Further, Central Electricity Authority has identified 56 additional potential sites for development of pumped storage schemes with a probable installed capacity of 93,920 MW.

World Scenario of Pumped Storage Schemes Development

Pumped storage hydroelectric schemes has its beginning in Switzerland, where the first pure pumped storage plant was constructed in 1904. This system of pumped storage power generation had separate turbine/generator and pump motor installation. The generator-motor system (tandem type) for combined usage as generator and motor was developed in 1910 and used in Italy.

In country like Germany where thermal power was main, the pumped storage power schemes were completed for the purpose of daily or weekly regulations so as to make effective the midnight surplus capacities of thermal power station and of imparting adaptability to be quick coping with fluctuations in power demand.

In 1925 a plant having greater capacity began operating in Federal Republic of Germany. In 1930s the first pumped storage plant in Japan was built.

To improve the economics of PSPP, reversible pump-turbines were developed in 1930s. Research and development of this type were carried out in various countries resettled in the introduction of high efficiency, high head, large capacity of pumped storage power stations. Today practically all pumped storage power stations have adopted this type.

The first significant progress in pumped storage (the beginning of single runner reversible pump/turbine development) was identified on the early 1940s in Brazil. The early 1950s saw the advent of the era of increased pumped storage construction. Canadian power authority developed the Niagara Falls installations, the first truly daily/nightly cycle pumped storage plants with reversible units. This plant was converted from existing hydroelectric installations demonstrating the technological advancement in pumped storage developments.

Before 1960, there were about 40 PSPP with an aggregate approximate installed capacity of 2700 MW. At the end of 1974 additional 90 number of pumped storage power plants with a combined total installed capacity 10 times greater were in operation and many were in various stages of construction.

There are 290 PSPP with an aggregate installed capacity of about 82800 MW in operation in the world. USA has the credit of having the largest numbers of PPSP (38 number with 18091 MW total installed capacity). The second largest numbers of PSPP in operation are in Japan (38 number with 17005 MW total installed capacity). The third largest producer of PSPP is Italy (20 number with 6449 MW total installed capacity), followed by Germany (35 number with 5688 MW total installed capacity).

The unit capacity-wise, World's largest capacity of pump-turbine unit (398 MW) is installed at Helms PSPP in USA, installed in 1984. The second largest unit size (386 MW) is also in USA at Racoon Mountain installed in 1979.

There are 42 PSPP with an aggregate installed capacity of about 27400 MW in various stage of construction in the World. Maximum numbers of PSPP under construction are in Japan (8 number with 5480 MW total installed capacity).

About 550 number of PSPP aggregating to about 392,000 MW installations have been planned allover the world. Japan has planned the largest number of PSPP (440 number with 329,116 MW total installed capacity) for future development.

Reservoir Operation Study

The reservoir operation table is the result of reservoir operation study, which is also termed as working table shows the pattern of the inflow of water into the reservoir, and demand for the corresponding period, surplus to be stored or released during period. This is to be studied for the optimum utilization of the runoff A correctly prepared operation table can show at a glance how the reservoir is going to behave after it has been put into operation. Operation table is very useful part of the project and should not be over looked. It will also prove the adequacy of the capacity of the reservoir tentatively fixed earlier.

The reservoir operation plan is devised to achieve the greatest value or benefit from the storage capacity. The plan must be based on:

a. Knowledge of the flow characteristic of the stream.

b. The purpose or purposes of the reservoir must be analyzed to determine, how the hydrograph of flow should be altered to produce the greatest benefits.

c. Special considerations, such as the effect of sudden releases of stream flows and long sustained flows from the reservoir on agricultural developments and other interests in the valley below the reservoir.

The cooperation of two or more water uses generally permit savings large enough to make multi purpose storage reservoir economically attractive.

The manner in which the coordination of different purposes is realized is controlled by the most important utilization of storage character of the sites determines this in turn. For instance, power generation is usually considered a secondary benefit in comparison to the other uses, but in some instances, power is the main objective. In such cases, the flood moderation capacity is provided as the surcharge capacity, so that any restriction on power generation should be precluded.

A multi purpose operation does not differ essentially from the single purpose one. Identical principles apply to each water use without any regard to other uses, which are served by the same reservoir. The main difference is the necessity of allocating the specific capacity for each of the uses served.

For pumped storage power project, operation is based on the principle of power generation by exchange of the water between two impoundment. These impoundments are often man made, resulting from the damming of a free flowing stream. This scheme uses the energy of stored water to generate and store electric power.

Decision on Operating Criteria

The success of the pumped storage power development depends on the availability of the pumping up energy. Therefore the pumping up energy should be carefully studied to ensure the availability of it.

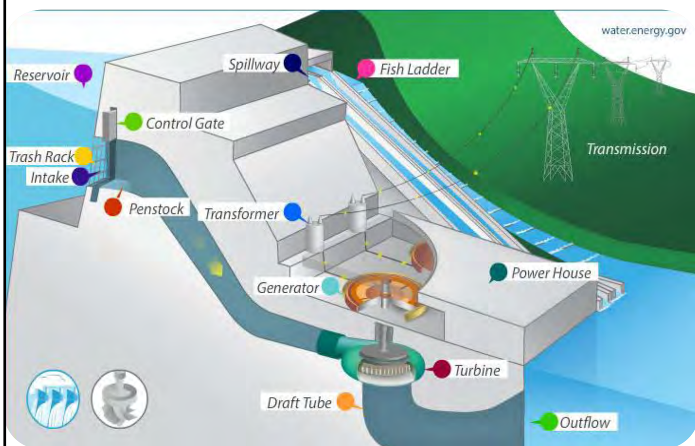
When total firm output of the base load power stations linked to the power system exceeds the off period demand, the difference becomes the availability of pumping up energy.

The pumping energy resources can be calculated based on demand projection, the power development projects and daily load curve on the typical highest load days and the lowest load days and the lean period. The difference between the daily loads curved produced for future demand and the base load during the night can be evaluated as pumping energy resources.

From projected typical daily load curve characteristic and availability of electric power, decision can be taken regarding:

- The hours during which and time when the surplus energy would be available in a day for pumping water from lower reservoir to the upper reservoir utilizing this surplus energy and therefore to decide number of pumping hours.
- The load curve would also indicate the deficit energy duration in hours and its period in order to use the available amount of water for the peak power generation.

Hydropower Projects
Planning and Design Aspects



Abhinav Srivastava
Deputy Director
HCD(E&NE)
CWC

1

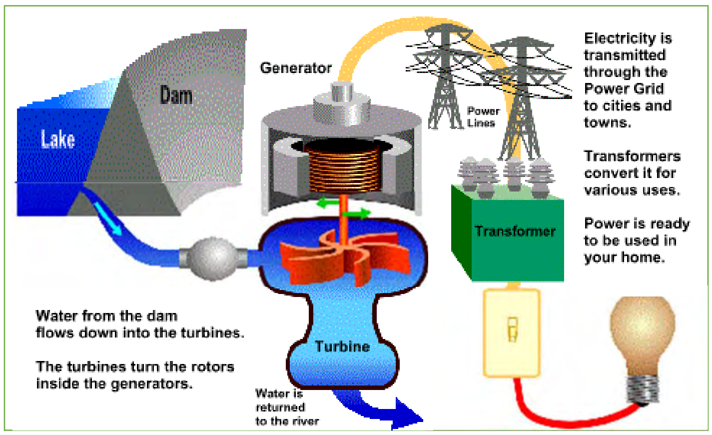
WHAT IS HYDRO POWER?

- The term '**hydro**' is the Greek word for water and hydropower is the energy contained in water .
- Hydropower is considered a **renewable energy source.**
- Hydropower transforms the potential energy of a mass of water flowing in a river or stream with a certain vertical fall (termed the "head").
- The power output from the scheme is proportional to the flow and to the head

2

HOW A HYDROELECTRIC POWER SYSTEM WORKS

The mechanical energy produced by the turbine is converted into **electric energy** using a turbine generator. Inside the generator, the shaft of the turbine spins a magnet inside coils of copper wire. It is a fact of nature that moving a magnet near a conductor causes an electric current.



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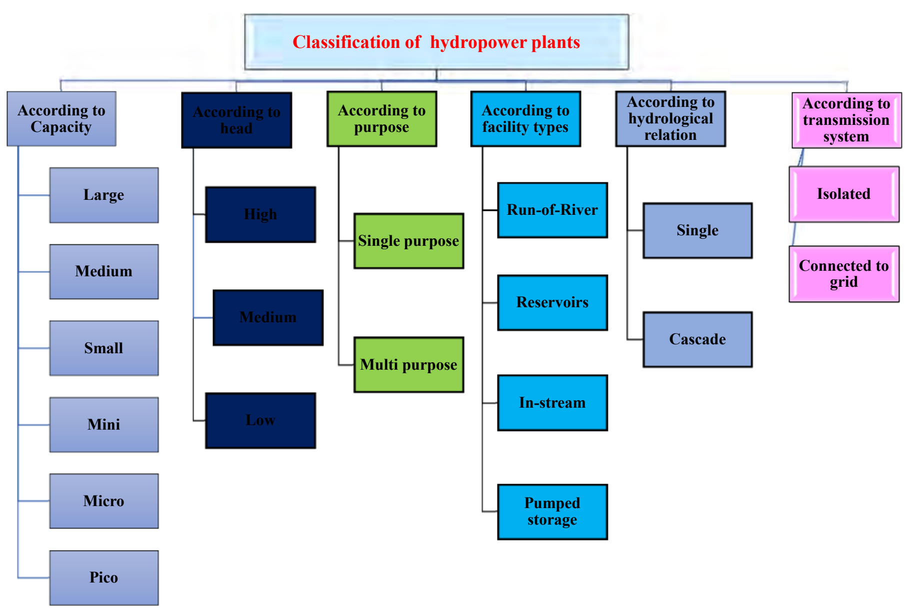
CLASSIFICATION OF HYDROPOWER PLANTS :



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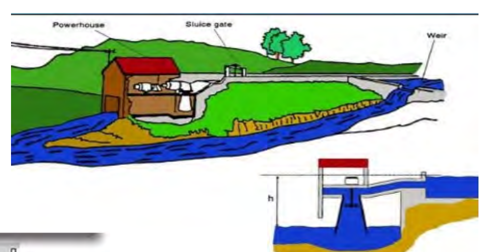
CLASSIFICATION ACCORDING TO CAPACITY:

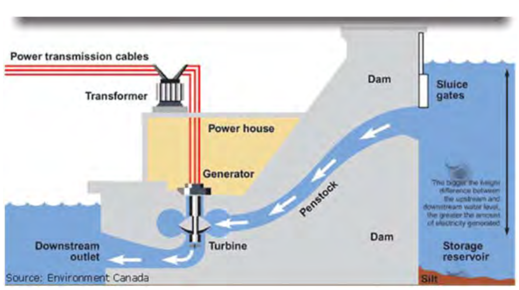
- LARGE: >100 MW
- MEDIUM: 25 – 100 MW
- SMALL: 1-25 MW
- MINI: 100 KW - 1MW
- MICRO: 5 – 100 KW
- PICO: < 5 KW

5

CLASSIFICATION ACCORDING TO HEAD

- LOW HEAD < 30 m





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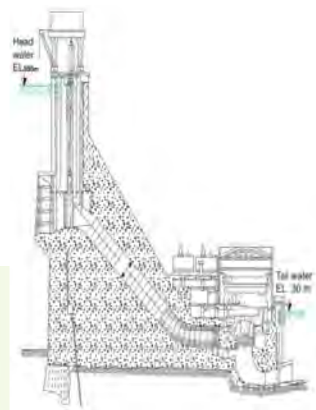
CLASSIFICATION ACCORDING TO HEAD

- MEDIUM HEAD 30 – 300 m



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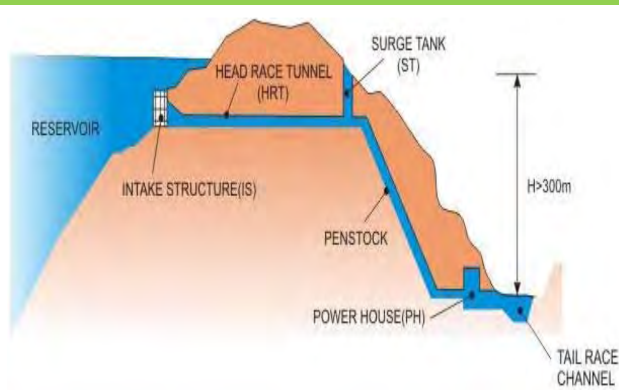
CLASSIFICATION ACCORDING TO HEAD

- HIGH HEAD > 300m



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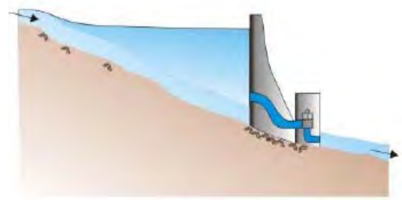
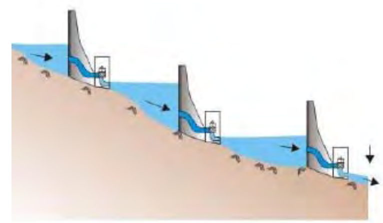
CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

SINGLE STAGE- When the run off from a single hydropower plant is diverted back into river or for any other purpose other than power generation



(a)

CASCADE SYSTEM- When two or more hydropower plants are used in series such that the runoff discharge of one hydro power plant is used as the is a intake discharge of the second hydro power plant



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CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

Multi-purpose Project

- Power generation may be one of the benefits along with Flood Control, Irrigation, Navigation, Drinking Water Supply etc.

Purely Hydro-electric Project

- Project is conceived exclusively for power generation.

10

CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

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 water conductor system and finally water after generation of power is thrown back to the river at a lower level on down stream.
- It takes advantage of the drop in elevation that occurs over a distance in the river and does not involve water storage.
- 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.

11

CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

Storage Project

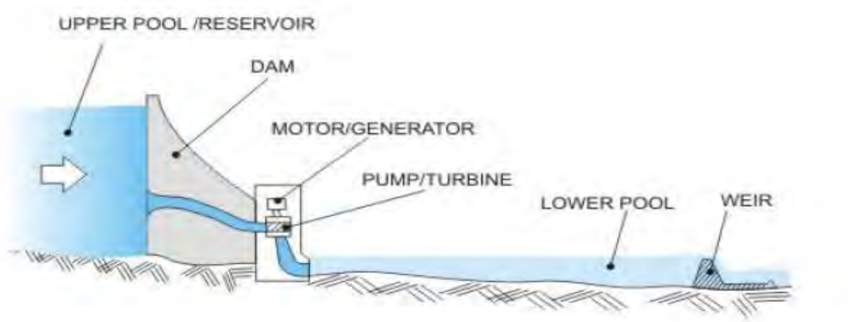
- Storage projects provide storage or pondage and thereby, evens out stream flow fluctuations and enhances the water head.
- It increases firm power and total power generation by regulating the flow.

12

CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

Pump Storage project

- Pump storage projects involve reversible turbines, which can generate power from water of upper reservoir during peak hours and pump back water from lower reservoir to the upper reservoir during off peak hours.



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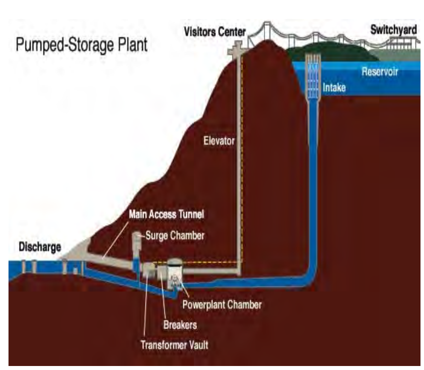
CLASSIFICATION ACCORDING TO HYDROLOGICAL RELATION

Pump Storage project

- These projects are advantageous in power system of mix type, which have thermal and nuclear power houses in addition to hydro power projects.
- Pump storage project utilizes the off peak surplus power of the grid in
- lifting the water from
- lower reservoir to higher
- reservoir and generates
- power during peak hours
- thus flattening the load
- curve.



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HYDRO DEVELOPMENT- IMPORTANT TERMS

- **FRL (FULL RESERVOIR LEVEL)**

  FRL is the Upper level of the reservoir (selected based on techno-economic& submergence considerations)

- **MDDL (MINIMUM DRAWDOWN LEVEL)**

  Lowest level up to which the reservoir level could be drawn down to withdraw waters for energy generation (selected from considerations of silt & turbine operational limits)

- **GROSS STORAGE**

  Total storage capacity of the reservoir

15

HYDRO DEVELOPMENT- IMPORTANT TERMS

- **DEAD STORAGE**

  Reservoir storage which cannot be used for generation and is left for silt deposition(below MDDL)
- **LIVE STORAGE**

  The storage in the reservoir which is available for power generation (between FRL & MDDL)
- **FIRM POWER**

  Continuous power output in the entire period of hydrological data at 90% dependability
- **FIRM ENERGY**

  Energy generated corresponding to firm power

16

HYDRO DEVELOPMENT- IMPORTANT TERMS

- **Peak Energy**

  Electric energy supplied during periods of relatively high system demands.

- **Off-peak Energy**

  Electric energy supplied during periods of relatively low system demands.

- **Load Factor**

  Ratio of the average load over a designated period to the peak-load occurring in that period

- **Diurnal Storage**

  Storage required to meet daily variations in load demand. It depends upon the minimum flows and peak discharges.

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HYDRO DEVELOPMENT- IMPORTANT TERMS

- Design Head

  The head at which the turbine will operate to give the best overall efficiency under various operating conditions.

- Gross Head

  The difference of elevations between water surfaces of the forebay/ dam and tailrace under specified conditions.

- Net Head

  The gross head chargeable to the turbine less all hydraulic losses in water conductor system.

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COMPONENTS

1. Diversion structure dam/weir
2. Intake Structure
3. De-silting arrangement
4. Power channel or Head Race tunnel
5. Surge shaft or forebay
6. Pressure shaft or penstock
7. Power House Complex
8. Tail Race system

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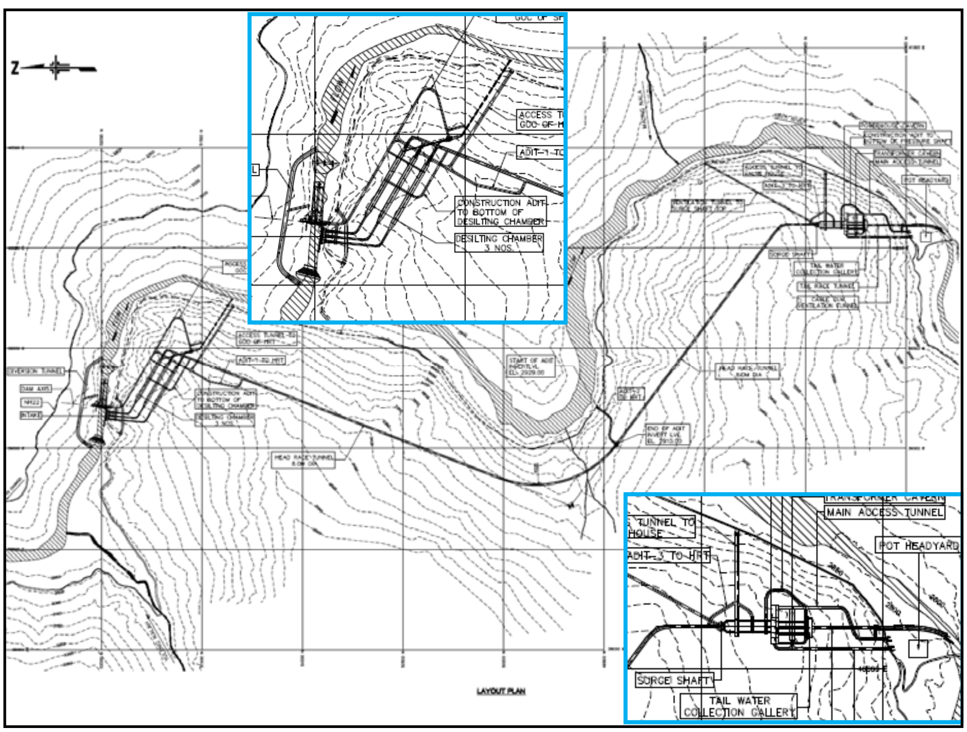

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Dam/Barrage

- The dam/ barrage is used for delivering/diverting the water to the water conductor system for power generation



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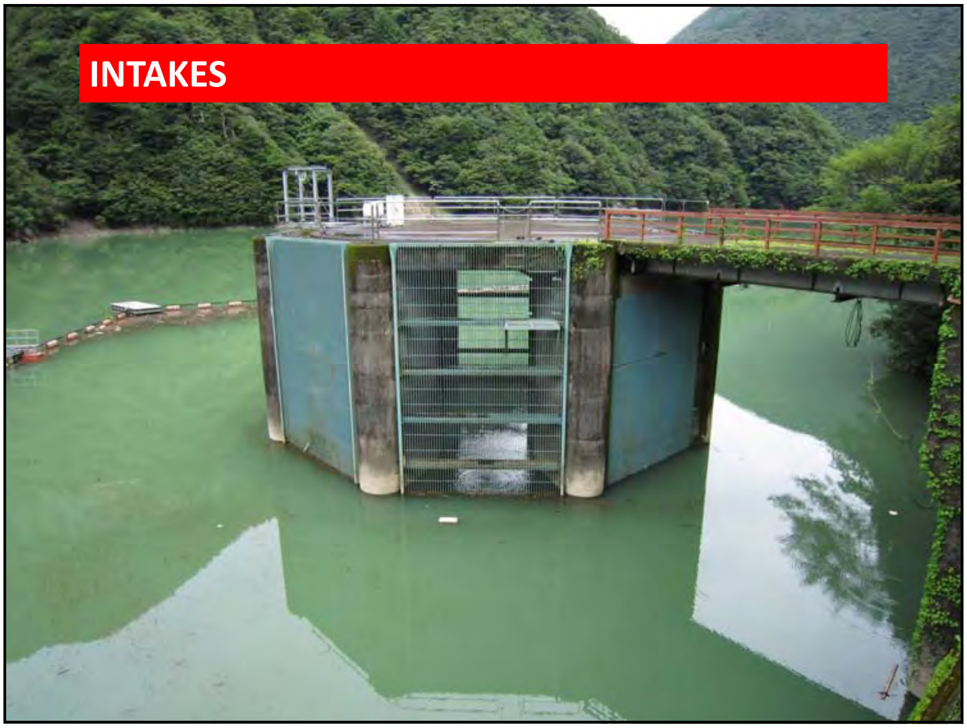
INTAKES



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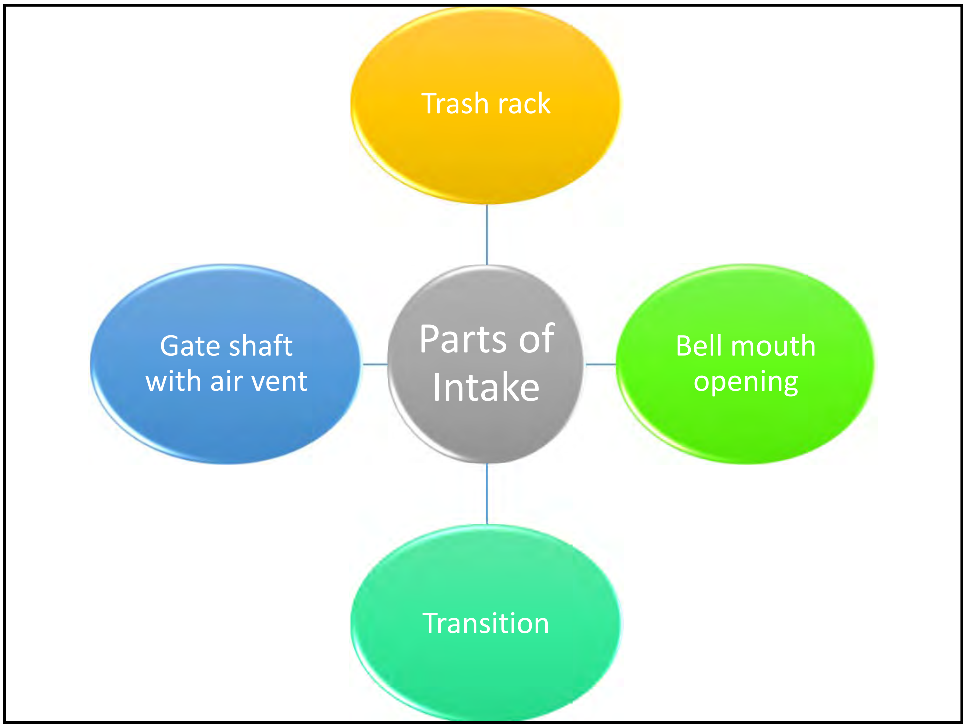

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Intake

- The intake is an arrangement to divert the water from the reservoir to the water conductor system. The objectives of intake design are:
  - to prevent entry of trash, debris and other floating matter in the water
  - to prevent or reduce the silt when water is required for power generation which is likely to damage the runner blade.
  - to ensure smooth hydraulic conditions and to minimize the head loss

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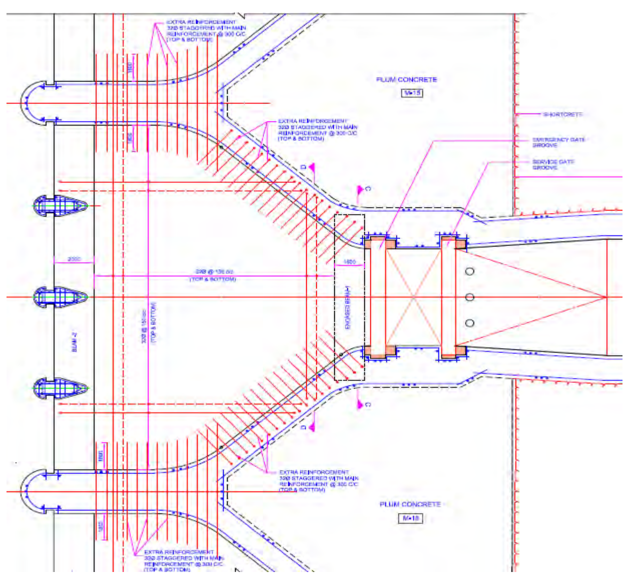
Intake Parameters

- Shape / Size
  - The shape should follow the profile of the jet entering the opening
  - Normally shape is an ellipse
  - Shape is usually rectangular
  - Size is set to keep entry velocity 0.6 to 1 m/s
- Location
  - Has to be above the dead storage level
  - It should be sufficiently below the MDDL

25



PLAN OF INTAKE

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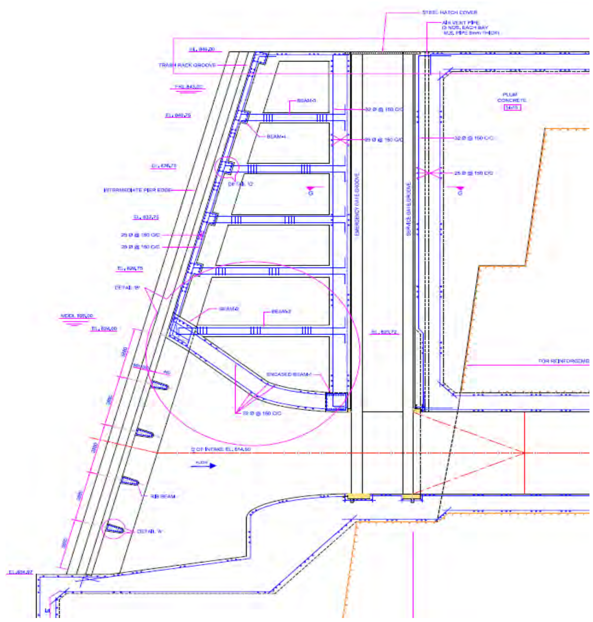

ELEVATON OF INTAKE

27

Trash Rack

- What is trash rack
  - screen provided at the entrance of the intakes
  - to prevent the entry of any trash that would not pass through the smallest opening in the machine

- Hydraulic considerations
  - Flow through velocity 0.6 –0.9 m/sec.
  - Spacing of bars less than opening between turbine blades.
  - Piers and ribs to be streamlined
  - Designed for a differential head of 1/2H or 6m. whichever is minimum.

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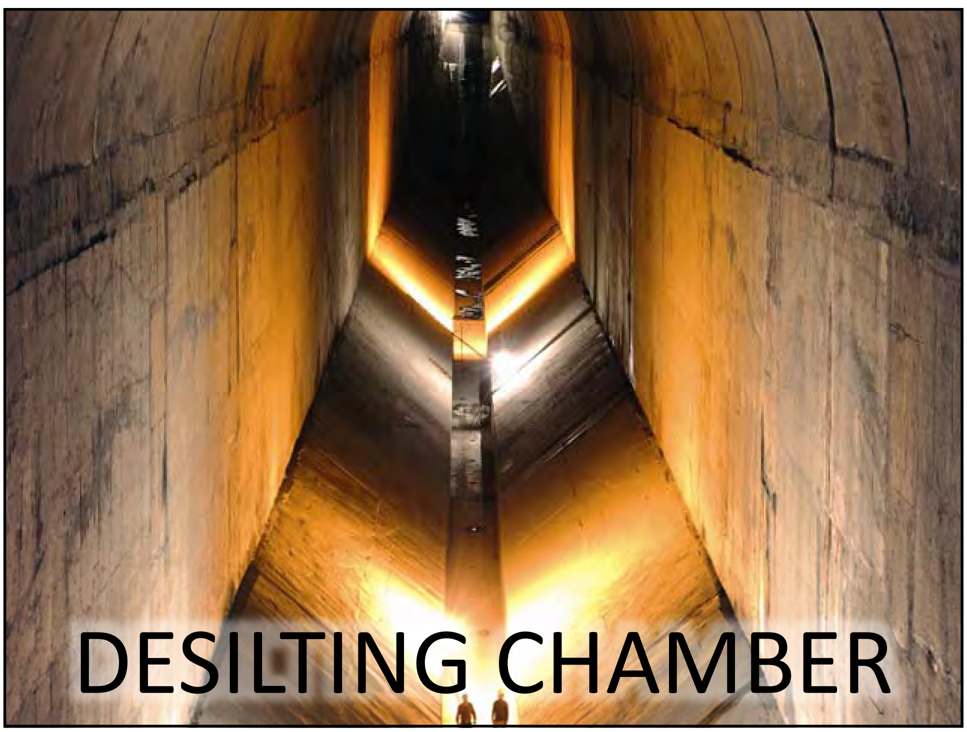
DESILTING BASIN

29

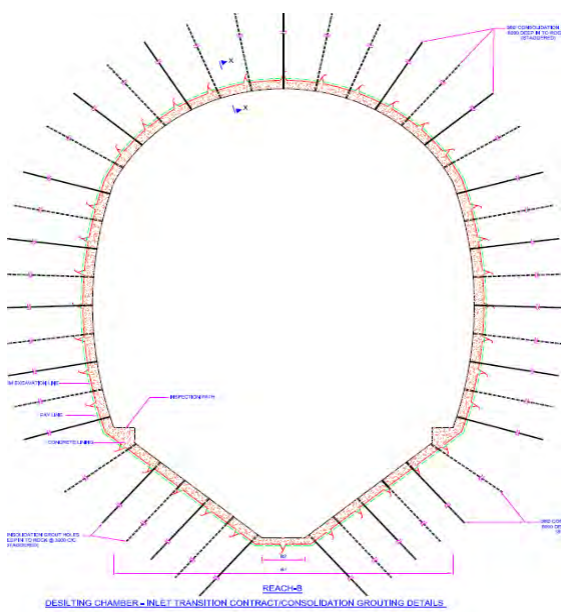


30

Desilting Basin

- Purpose
  - To remove the suspended silt load from the water
  - To prevent the damaging effect on the turbines
- Designed to
  - Settle particles of a size larger than particular size
- Location
  - Usually provided just d/s of the intake structure

31



De-silting Chamber Cross Section

32

WATER CONDUCTOR SYSTEM

33

Water Conductor System

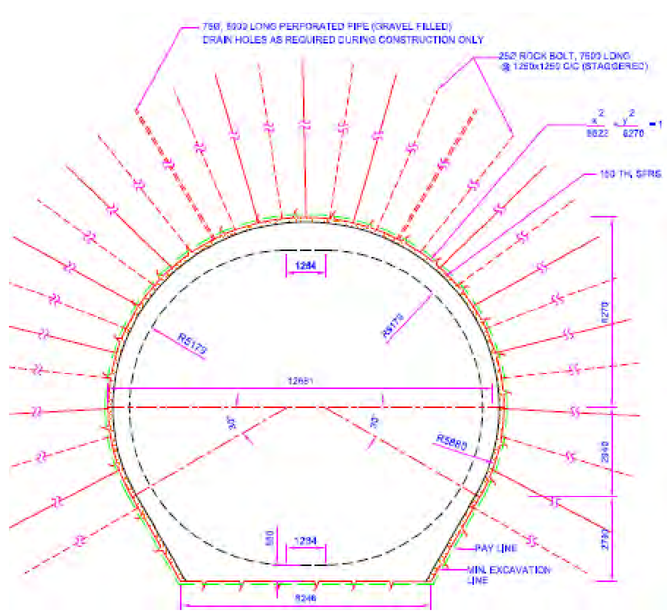
- Purpose
  - To carry the water from de-silting basin to the surge tank / forebay
- WCS Types
  - Channels
    - Trapezoidal / Rectangular
  - Tunnels
    - Circular / D-shape / Horse shoe / Modified horse shoe

34

Tunnel Design

- Hydraulic Design
  - Pressure flow-Velocity should not be excessive
  - Roof of tunnel should always be below energy gradient line
- Structural Design
  - Rock surrounding the tunnel should be stable
  - Supported using rock bolts / shotcrete / steel ribs / lattice girders

35



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Surge Tanks

- Purpose
  - Water hammer relief
  - Pressure regulation
  - Flow regulation
  - Machine speed regulation
- Design
  - Hydraulic design
  - Structural design

38

Surge Tanks

- Purpose
  - Water hammer relief
  - Pressure regulation
  - Flow regulation
  - Machine speed regulation
- Design
  - Hydraulic design
  - Structural design

1. To determine the shape, size and water level fluctuations, during upsurge and downsurge.
2. The height of the surgetank provided should be atleast 1.5m above the highest surge level and 2.0m below the lowest surge level.
3. It shall be made sure that there is no vortex formation at the lowest water level in the surge tank and the Head race tunnel does not get exposed for entry of air during the downsurge.

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Surge Tanks

- Purpose
  - Water hammer relief
  - Pressure regulation
  - Flow regulation
  - Machine speed regulation
- Design
  - Hydraulic design
  - Structural design

Surge tank is normally an open excavated concrete lined cylinder. It should be designed for

1. lateral rock cover
2. Internal loads
3. External loads including front pressure

40

PENSTOCK / PRESSURE SHAFT

41

Penstock / Pressure Shaft

- Used to transfer the water from fore-bay or surge tank to the powerhouse
- Design of penstock
  - Designed to resist the total internal pressure consisting of static and water hammer pressure
  - If subjected to outside pressure, then the thickness shall be checked for buckling
- Penstock specials
  - Bifucation piece, trifrucation piece, bends, expansion joints, reducer, bulkheads, thrust collars, etc.

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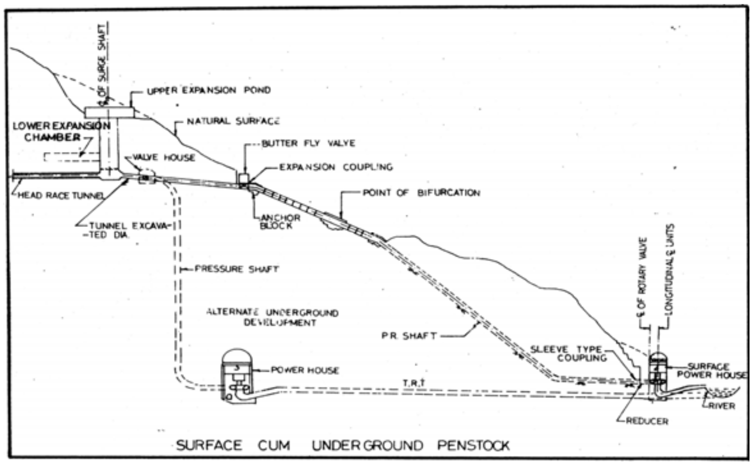
Penstock / Pressure Shaft



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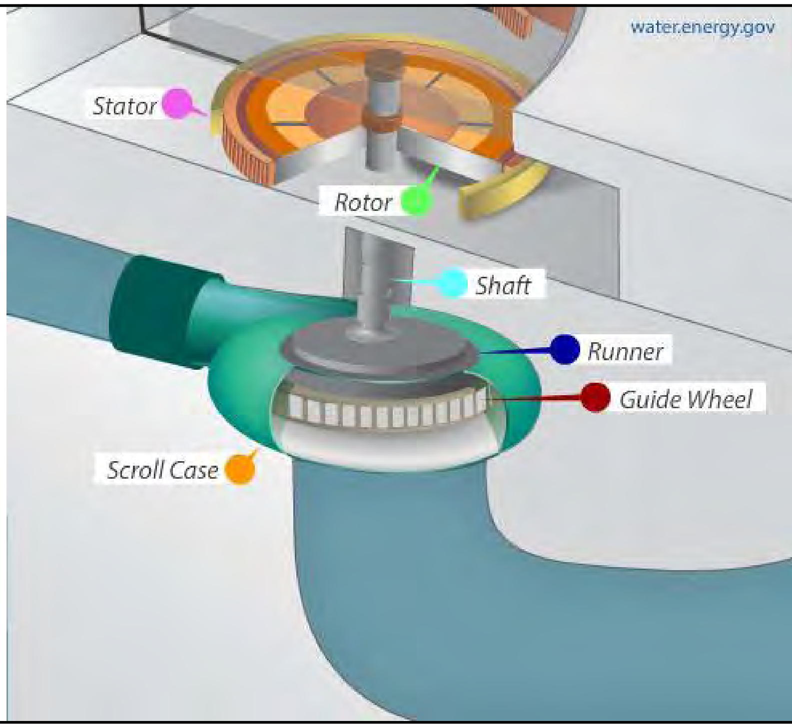
POWER HOUSE

44



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Power House

- Houses all generators and appurtenant machinery
- Components
  - Machine hall / Service bay / Auxiliary bay or control bay
  - Sub structure / Super structure
- Design of power house
  - Design for gentry loads, external loads, machine loads, etc.
- Turbines
  - Impulse, Reaction and Gravity turbines

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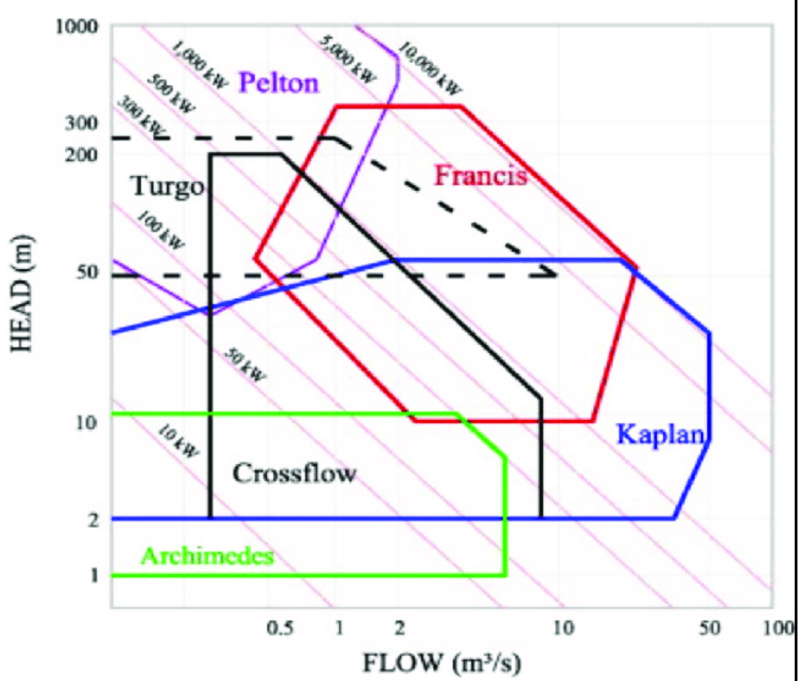
47


1000
300
200
50
10
2
1
HEAD (m)
0.5
1
2
10
50
100
FLOW (m³/s)
1,000 kW
500 kW
300 kW
5,000 kW
10,000 kW
100 kW
50 kW
10 kW
Pelton
Turgo
Francis
Kaplan
Crossflow
Archimedes

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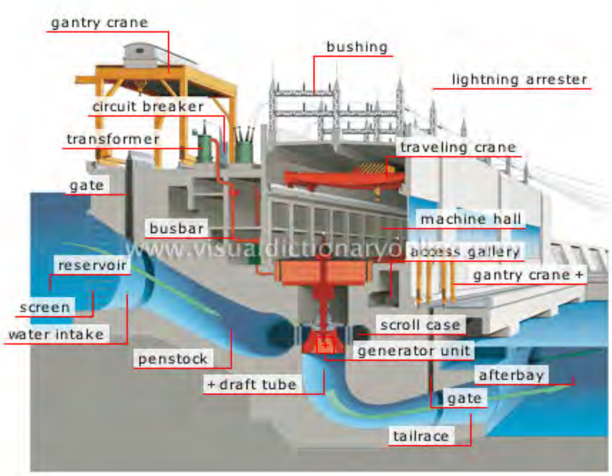
Surface Power House



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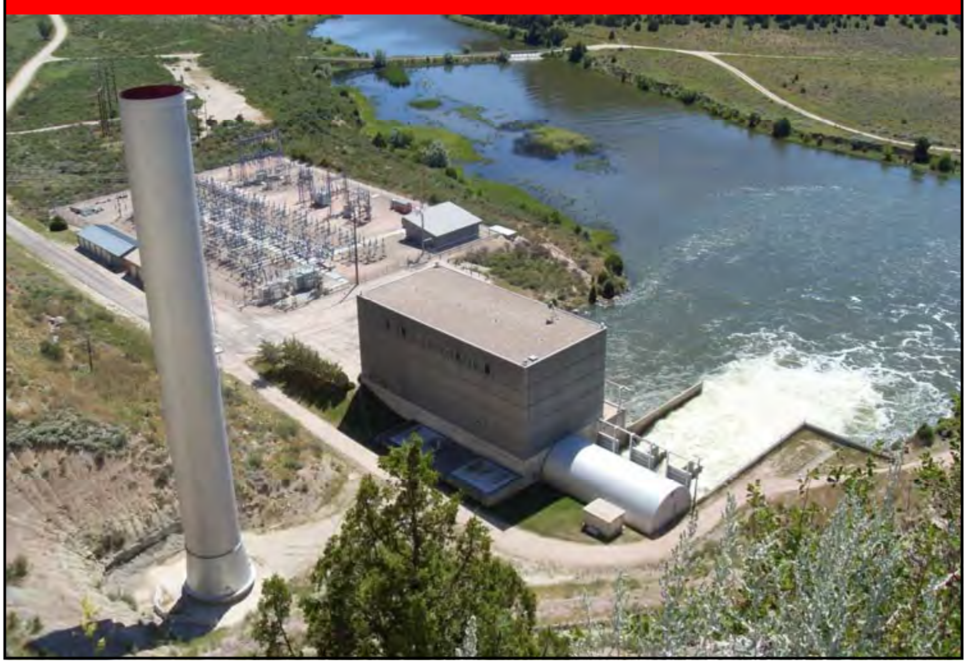
TAIL RACE SYSTEM



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Tail Race System

- Last part of the water conductor system
- Minimum tail water level is critical for design of turbines
- Usually tail water pool is provided
- Downstream surge tank may be required depending on length of TRT

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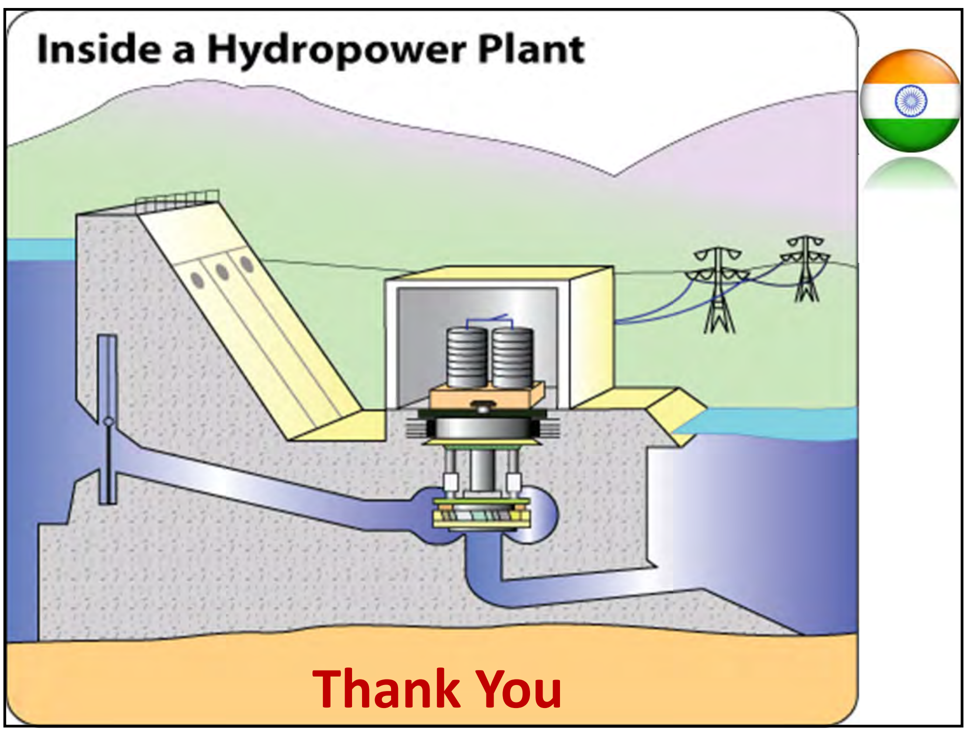

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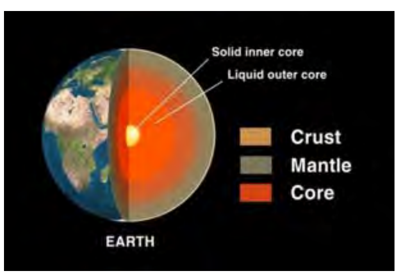
UNDERGROUND EXCAVATIONS –

ROCK FORMATION - GENESIS
CLASSIFICATION OF ROCKS – BASED ON ITS GENESIS
ROCKMASS – DIFFERENT FROM ROCK
ROCK DISCONTINUITY
INTRODUCTION TO JOINTS

ABHINAV SRIVASTAVA
Deputy Director
HCD (E&NE) Dte , CWC (HQ)

1

ROCK FORMATION - GENESIS

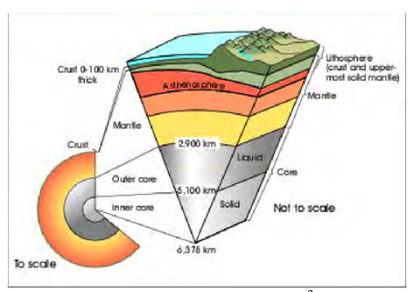


Upper Crust

5-10 km thick
Solid State & Very Brittle
All Phenomenon – Tectonic Activity/
Volcanic Eruption observed here

Lower Crust

10-30 km thick
Liquid Molten State
Movement of this layer causes all Disruptions/Discontinuities in Rocks



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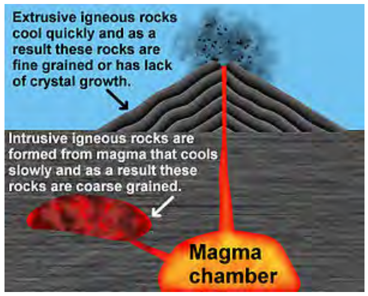
2

ROCK FORMATION - GENESIS



IGNEOUS ROCKS

Toughest , Competent
Sound UCS (>200 MPa) and E (75 GPa)

Intrusive Igneous Rocks >> Extrusive Igneous Rocks

TEXTURE

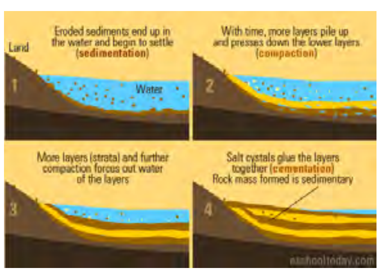
Minerals are Homogenously Mixed

May be Fine and Coarse Grained Depending on Rate of Cooling.

Granite & Basalt

3

ROCK FORMATION - GENESIS



SEDIMENTARY ROCKS

Not Tough Enough
Not So Sound UCS (50-100 MPa) and E (40 GPa)

Has Weak Planes of Schistosity

TEXTURE

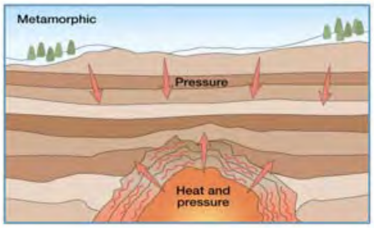
Minerals are bedded in Layers

Non Homogenous and thus Anisotropic

Sandstone and Limestone

4

ROCK FORMATION - GENESIS



METAMORPHIC ROCKS

Toughness
Igneous ><Sedimentary

Good Enough UCS (150-200 MPa) and E (50 GPa)

Shows Weak Planes of Schistosity

Marbel	Gneiss	Slate	Quartzite
			
Marble is used for countertops, statues and decorative stone. Marble is white, gray, or black. Marble has a pearly feel to it.	Gneiss is used for flooring, grave stones, and building materials. Gneiss is red, black, brown or gray.	Slate is used for flooring and roofing. Slate is black and is very durable.	Quartzite is used for building material, roofing, flooring, decorative rocks and tiles

5

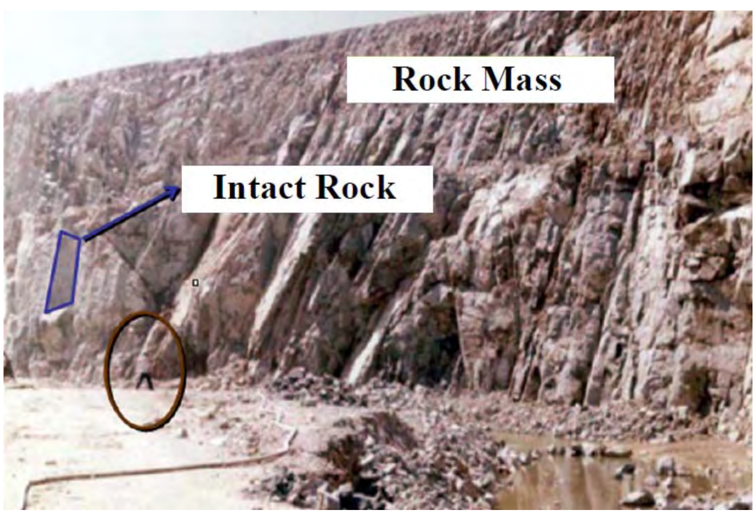
TEXTURE

Non Homogenous and thus Anisotropic

Marble and Slate

5

ROCK AND ROCK MASS



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ROCK AND ROCK MASS

DISCONTINUITY

- Joints
- Fractures
- Faults
- Shear zones

- Makes rock discontinuous
- Makes rock anisotropic
- Makes rock stress dependent

7

FACTORS AFFECTING STRENGTH OF ROCK MASS

- Nature of discontinuities
- Location of discontinuities
- Orientation of discontinuities

  - Deformability
  - Strength
  - Permeability

8

A ROAD CUTTING IN ROCK

9

10

JOINTING

11

SYNCLINE & ANTICLINE

12

13

UNDERGROUND STRUCTURE – FAILURE MECHANISMS

For an underground opening, the failure can occur mainly in two ways:

- **Structurally- controlled and / or**
- **Stress- controlled**

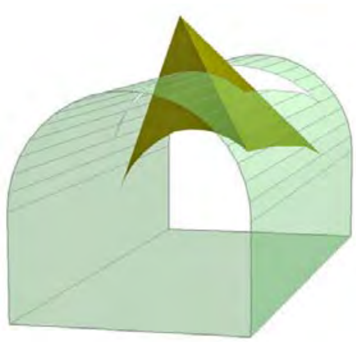
14

STRUCTURALLY- CONTROLLED FAILURE.

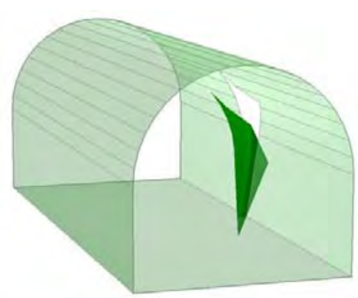
- *jointed rock masses at relatively shallow depth.*
- *wedges falling from the roof or sliding out of the sidewalls of the openings.*
- *wedges formed by intersecting structural features, such as bedding planes and joints.*
- *Unless steps are taken to support these loose wedges, the stability of the opening may deteriorate rapidly and the structure could fail.*
- *unidirectional body force ie., gravity.*

15

STRUCTURALLY- CONTROLLED FAILURE



ROOF WEDGE



SIDEWALL WEDGE

16

STRUCTURALLY- CONTROLLED FAILURE
Design Approach

- Determination of average dip and dip direction of significant discontinuity sets.

- Identification of potential wedges which can slide or fall from the back or walls.

- Calculation of the factor of safety of these wedges, depending upon the mode of failure.

- Calculation of the amount of reinforcement required to bring the factor of safety of individual wedges up to an acceptable level.

17

STRUCTURALLY- CONTROLLED FAILURE

Identification of potential wedges

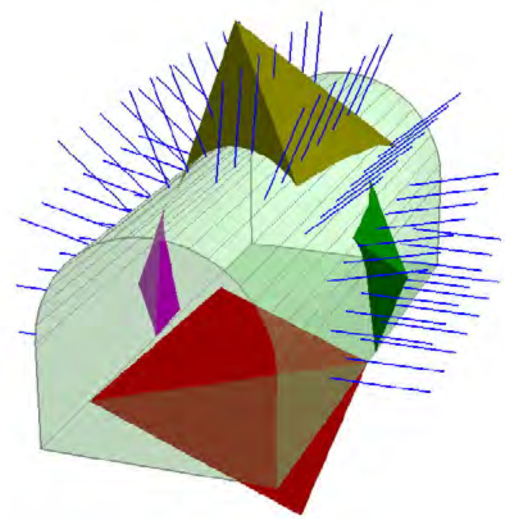
- size, shape and orientation of the opening
- orientation of the significant discontinuity sets.

Calculations tedious due to 3D geometry of the problem

Use of computer programs like UNWEDGE

18

STRUCTURALLY- CONTROLLED FAILURE



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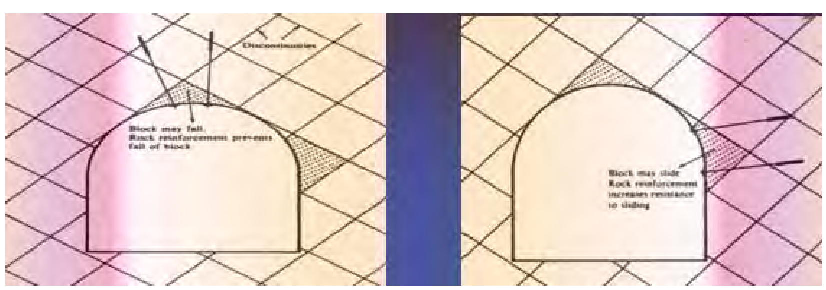
STRUCTURALLY- CONTROLLED FAILURE



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STRESS-CONTROLLED FAILURE.

- ***Excavation at more depth.***
- ***High stresses.***
- ***Rock mass is relatively homogeneous and isotropic.***
- ***The failure is essentially stress related.***

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Closed form solutions for induced stress:

- For simple shapes such as circle, ellipse etc. closed form solutions for induced stress are available.
- For complex 3-dimensional excavation geometry, the induced stresses can be computed by Numerical Methods.

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Circular Excavations -

Stresses and Displacements

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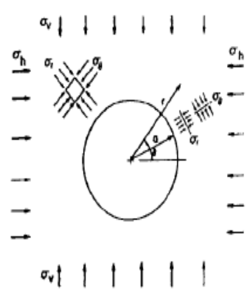
Box 8-1. Stresses Around a Circular Opening in a Biaxial Stress Field



Notation used to describe stresses around a circular opening in a biaxial stress field

a	radius of tunnel shaft
r	radial distance to any point
Θ	angular distance to any point
σ_h, σ_v	original (pre-tunneling) stress field at the tunnel level
σ_θ, σ_r	final (post tunneling) radial and tangential stresses around the tunnel
E	is Young's Modulus of the rock
υ	is the Poisson's Ratio
u_a	is the radial displacement at radius a
v_a	is the tangential displacement at radius a

The stresses are:

radial stress $\sigma_r = 0.5(\sigma_v + \sigma_h)(1 - a^2/r^2) + 0.5\,(\sigma_v - \sigma_h)(1 + 3a^4/r^4 - 4a^2/r^2)\cos 2\Theta$

circumferential stress $\sigma_\theta = 0.5(\sigma_v + \sigma_h)(1 + a^2/r^2) - 0.5\,(\sigma_v - \sigma_h)\,(1 + 3a^4/r^4)\cos 2\Theta$

shear stress $\tau_{r\theta} = 0.5(\sigma_h - \sigma_v)(1 - 3a^4/r^4 + 2a^2/r^2)\sin 2\Theta$

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At the boundary of excavation, $r = a$,

The radial stress, $\sigma_r = 0$,

The circumferential stress,

$\sigma_\theta = \sigma_v$ [(1+ k) – 2(1-k) cos 2θ]

The shear stress , $\tau_{r\theta} = 0$

where , $\sigma_h = k . \sigma_v$

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For the case when the horizontal stress is zero ie., $\sigma h = 0$ and

Hydrostatic stress field ie.,

$\sigma h = \sigma v$,

The radial and tangential stress distribution are shown below.

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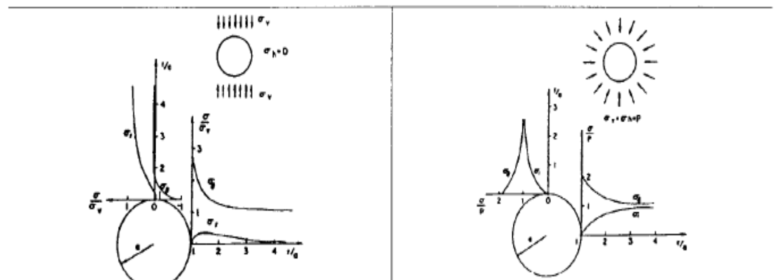

Radial stress (σ_r) and tangential stress (σ_θ) along the vertical and horizontal axes of a circular tunnel (shaft) in a uniaxial stress field (σ_v).

Radial stress (σ_r) and tangential stress (σ_θ) around a circular tunnel (shaft) in a hydrostatic stress field (P).

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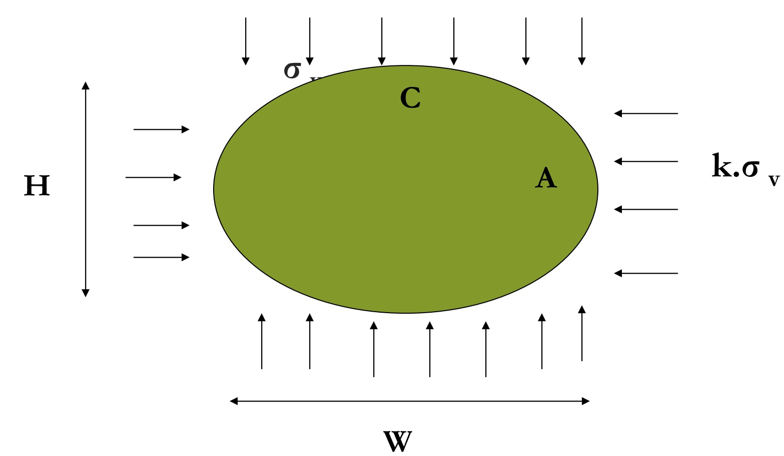
Elliptical Excavations-stress and displacements



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The tangential boundary stresses are given by the following equations:

$\sigma A = \sigma v \; [1 + \surd(2.W/\rho A) - k]$

$\sigma C = \sigma v \; [k\{1 + \surd(2.H/\rho C)\} - 1]$

where σA and σC are the tangential boundary stresses at location A & C.

ρA and ρC are radii of curvature at A & C.

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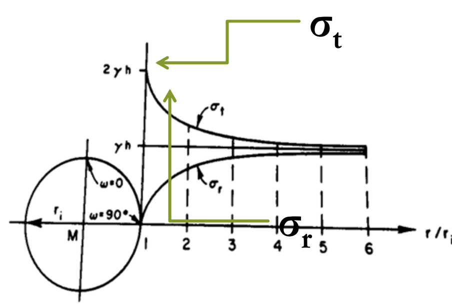
ELASTIC STRESS ANALYSIS SOME IMPORTANT OBSERVATIONS

From the elastic stress analysis it can be seen that:

When distance(r / r1) more than 4 , the stress increment is very less. Hence practically the extent of the disturbed zone can be taken as twice the width of the opening.

The stress around the opening are independent of the elastic constants of rock material and radius of the opening. It depend on (r / r1) only.



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GEOLOGICAL & GEOTECHNICAL DATA

- **Faults**
- **Shear zones**
- **Discontinuities**
- **Joint pattern, spacing, roughness etc.**
- **In-situ Data**
  - ➢ **In-situ stress**
  - ➢ **Pore water pressure**
- **Deformation modulus**

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IN-SITU STRESS:

The pre existing state of stress in the rock mass before any excavation is carried out is called the in-situ stress.

The common methods for determination of magnitude and directions of in situ stress are:

- **Flat jack technique**
- **Overcoring technique**
- **Hydro-fracturing method**

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Stresses around multiple openings.

- The zone of readjustment can include points that are located upto 5 times the diametrical distance from the centre of the opening.

- Thus if another tunnel is located such that the pillar thickness equals 9 (R1 + R2), where R1 & R2 are the radii of the two adjacent tunnels, then the readjustment of the stresses and displacements will be independent of the second tunnel.

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In actuality, rock is not that homogeneous and as such elastic theory may not be fully applicable. In practice, if the pillar thickness equals the largest adjacent tunnel, the multiple openings behaves as a single opening and the stability of the intervening pillar should be analysed in detail.

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ROCK PILLAR BETWEEN CAVERNS

- For weak rock masses the pillar **should not be less than the height of the larger cavern**
- In very poor rock masses, in which the overstressed zones are larger, **the pillar thickness should be 1.5 times the height of the larger cavern.**
- These multiple caverns should be subjected to numerical analysis for confirming the adequacy of the design.

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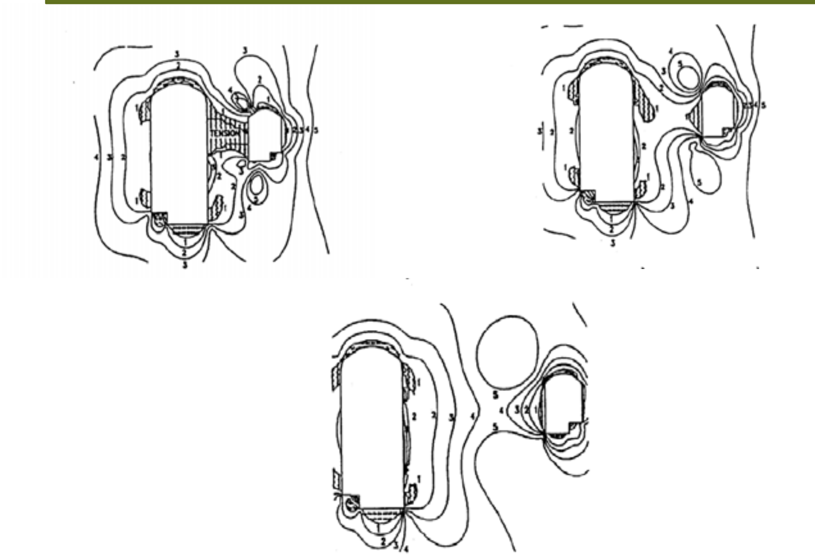
ROCK PILLAR BETWEEN CAVERNS



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NUMERICAL METHODS FOR STRESS ANALYSIS

- Used for complicated non-uniform or non-geometric shapes, tunnel intersection, bifurcation, stacked tunnels, power house caverns etc.
- Accommodates different material properties including joints, shear zones & faults.
- Linear or non- linear behavior
- Time dependent behavior

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NUMERICAL METHODS FOR STRESS ANALYSIS

- INPUT PARAMETERS
- IN-SITU STRESS – vertical / horizontal
- ROCK MASS – modulus of deformation
- Poisson's ratio
- uniaxial comp. strength
- density
- mi,mb,s,a - for Hoek & Brown failure criteria
- c , Ø - for Mohr- Coloumb failure criteria
- JOINT PARAMETERS - c , Ø , normal & shear
- stiffness
- SUPPORT PARAMETERS- for rock bolt, shotcrete etc.

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NUMERICAL METHODS FOR STRESS ANALYSIS

- COMMONLY USED SOFTWARES:
- EXAMINE 2D / 3D , PHASE 2 -
- FEM, B.E METHOD
- FLAC 2D/ 3D – FDM
- UDEC / 3-DEC - DISTINCT ELEMENT METHOD

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FAILURE CRITERIA

Generalised Hoek-Brown criterion

The Generalised Hoek-Brown failure criterion for jointed rock masses is defined by:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \qquad ($$

where σ_1' and σ_3' are the maximum and minimum effective principal stresses at failure,

m_b is the value of the Hoek-Brown constant m for the rock mass,

s and a are constants which depend upon the rock mass characteristics, and

σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

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FAILURE CRITERIA

- **MOHR- COLOUMB FAILURE CRITERIA:**

- **$\sigma 1f = \sigma 3 \tan 2\,(45+\text{Ø}/2) +$**
- **$2\,c \tan (45+\text{Ø}/2)$**

- **FACTOR OF SAFETY = $\sigma 1f / \sigma 1$**

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Importance of selection of failure criteria:

- **The Hoek- Brown failure criterion, which assumes isotropic rock and rock mass behaviour, should only be applied to those rock masses in which there are sufficient number of closely spaced discontinuities. When the structure being analysed is large and the block size is small in comparison, the rock mass can be treated as a Hoek-Brown material.**

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Importance of selection of failure criteria:

- Where the block size is of the same order as that of the structure being analysed or when one of the discontinuity sets is significantly weaker than the others, Hoek - Brown criterion should not be used. In these cases the stability of the structure should be analysed by considering failure mechanisms involving the sliding or rotation of blocks and wedges defined by intersecting structural features.

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DESIGN OF UNDERGROUND STRUCTURES:

- Steps involved are:

- Planning of Layout
- Shape / Size of Opening:
- Circular, Modified Horse Shoe, Horse
- Shoe, D-Shape Etc.
- Design of Rock Support System:
- Rock Bolts / Rock Anchors / Cable Anchors
- Sohtcrete (Plain / Wiremesh / Steel Fibre
- Reinforced) Steel Ribs Etc.

- Design of Lining, if Required:
- Steel
- Concrete (Plain / Reinforced)

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PLANNING OF LAYOUT

Considerations:

- **geology,**
- **minimum rock cover above the roof of the tunnel**
- **easy accessibility by provision of construction adits.**
  - The approximate length of main tunnel from each face of excavation should be kept to a maximum of 3000 m.
  - adit length also should be kept to a minimum
  - slope for proper natural drainage of water

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PLANNING OF LAYOUT

Location of portal for adits and tunnels in consultation with the geologist

Rock cover of minimum 2 x diameter of tunnel is necessary at the portal face of tunnel / adit.

stabilize the slopes by suitable treatment by using rock bolts and shotcreting etc.

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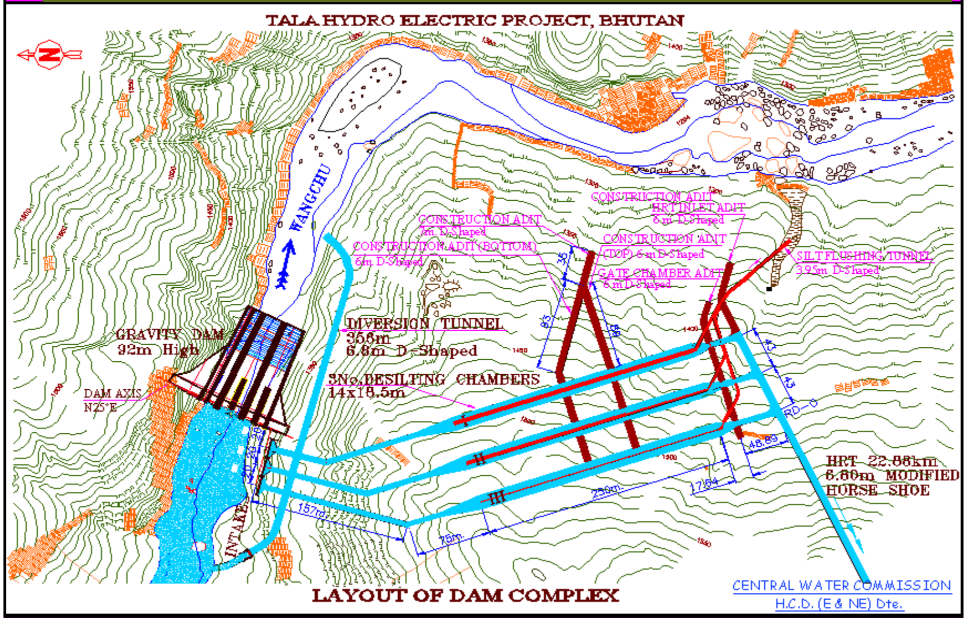
TALA HYDROELECTRIC PROJECT, BHUTAN



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PLANNING OF CAVERNS (POWER HOUSE , TRANSFORMER HALL ETC)

The planning of underground caverns is made considering the following geological & geotechnical data:

- Faults
- Shear zones
- Discontinuities
- Joint pattern, spacing, roughness etc.
- In-situ Data
  - In-situ stress
  - Pore water pressure
- Deformation modulus

Based on the above, the following are decided:

Deciding layout of power house cavern

Deciding orientation

Support system

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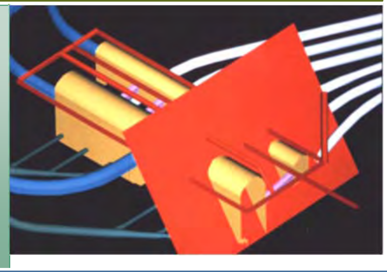
PLANNING OF CAVERNS
(POWER HOUSE , TRANSFORMER HALL ETC)

orientation of caverns

- *structural discontinuities like joint sets*
- *in-situ stress.*



The axis of the cavern is placed perpendicular to the strike of major joint set if structurally controlled failure is expected. In case stress induced failures are of major concern, as in the case of deep seated caverns, the axis of the cavern is oriented along the direction of major principal in-situ stress.

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SUPPORT DESIGN.

- EMPERICAL METHODS
- ANALYTICAL METHODS
- GRAPHICAL METHODS
- OBSERVATIONAL METHODS

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EMPIRICAL METHODS

THE EMPIRICAL APPROACH FOR THE DESIGN OF SUPPORT SYSTEM RELATES THE EXPERIENCE GAINED ON ROCK CONDITION AND SUPPORT REQUIREMENTS AT PREVIOUS PROJECTS TO CONDITIONS ANTICIPATED AT PROPOSED SITE.

THE EMPIRICAL APPROACH MAKES USE OF STATISTICAL ANALYSIS OF OBSERVATIONS.

ROCKMASS CLASSIFICATION WHICH ARE BASED ON SUCH CO-RELATION ARE MOST COMMONLY USED FOR ESTIMATION OF ROCK LOADS AND REQUIREMENT OF SUPPORT SYSTEM

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EMPIRICAL APPROACH

ROCK MASS CLASSIFICATION SYSTEMS

QUALITATIVE

- TERZAGHI'S ROCK LOAD

QUANTITATIVE

- DEERE'S R.Q.D.
- C.S.I.R. (BIENIAWSKI)RMR
- N.G.I (BARTON) "Q"

INVOLVES JUDGEMENT OF ROCK MASS

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QUALITATIVE ROCK MASS CLASSIFICATION

TERZAGHI'S ROCK LOAD CLASSIFICATION :

ONE OF THE EARLIEST ROCK CLASSIFICATION SYSTEMS FOR ESTIMATION OF LOADS TO BE SUPPORTED BY STEEL ARCHES IN TUNNELS

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TERZAGHI'S ROCK LOAD CLASSIFICATION IS DIVIDED INTO NINE ROCKMASS CATEGORIES :

- **HARD AND INTACT**
- **HARD STRATIFIED OR SCHISTOSE**
- **MASSIVE MODERATELY JOINTED**

- **MODERATELY BLOCKY AND SEAMY**
- **VERY BLOCKY AND SEAMY**
- **COMPLETELY CRUSHED BUT CHEMICALLY INTACT**

- **SQUEEZING ROCK MODERATE DEPTHS**
- **SQUEEZING ROCK GREAT DEPTHS**
- **SWELLING ROCKS**

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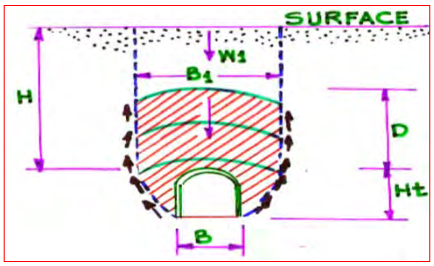
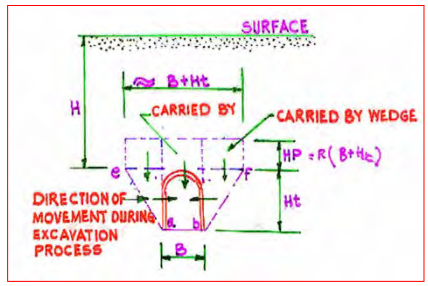
TERZAGHI'S ROCK LOAD CONCEPT





GROUND ARCH **ASSUMED SUPPORT LOADING**

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TERZAGHI'S ROCK LOAD CLASSIFICATION

S.No.	Rock Condition	Rock Load factor H_p	Remarks
	HARD AND INTACT	ZERO	LIGHT LINING REQD. ONLY IF SPALLING OR POPING OCC.
	HARD STRATIFIED OR SCHISTOSEC	0 TO 0.5 B	LIGHT SUPPORT
	MASSIVE MODERATELY JOINTED	0 TO 0.25 B	LOAD MAY CHANGE ERRATICALLY FROM POINT TO POINT
4	MODERATELY BLOCKY AND SEAMY	0.25B TO 0.35 (B+Ht)	NO SIDE PRESSURE
5	VERY BLOCKY AND SEAMY	0.35 TO 1.10 (B+Ht)	LITTLE OR NO SIDE PRESSURE
6	COMPLETELY CRUSHED BUT CHEMICALLY INTACT	1.10 (B+Ht)	CONSIDERABLE SIDE PRESSURE SOFTENING EFFECT OF SEEPAGE TOWARDS BOTTOM OF TUNNEL REQS. EITHER CONTINUOUS SUPPORT FOR LOWER ENDS OF RIBS OR CIRCULAR RIBS
7	SQUEEZING ROCK	1.10 TO 2.10 (B+Ht)	HEAVY SIDE PRESSURE
8	SQUEEZING ROCK GREAT DEPTH	2.10 TO 4.50 (B+Ht)	INVERT STRUTS REQD. CIRCULAR RIBS ARE RECOMMENDED
9	SWELLING ROCK	UPTO 250 FT. IRRESPECTIVE OF VALUE OF (B+Ht)	CIRCULAR RIBS REQD. IN EXTREME CASES USE YIELDING SUPPORT

B-tunnel span(m); Ht- Ht of the opening (m) ; Hp-Ht of loosened rock mass above tunnel crown developing load

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- Terzaghi's method though most commonly used tends to be subjective
- The geotechnical engineer has to use a considerable amount of judgement while interpreting rock outcrops or borings
- It is left to the judgement of the user as to how he interprets a particular rock type and interpretation may differ from person to person

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To assist the engineer in quantitatively classifying rock for engineering purpose i.e. for estimation of rock loads, the following systems have been found to be more useful :-

- Deere's R.Q.D. Classification
- Bieniawski (C.S.I.R) RMR Method
- Bartons (NGI) "Q" System

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DEERE'S R.Q.D. CLASSIFICATION

- **The existence of discontinuities is an important feature of rock as a geologic material and Deere (1970) proposed a relationship between the numerical value of R.Q.D. and rock mass quality and corresponding rock loads expected in the opening.**

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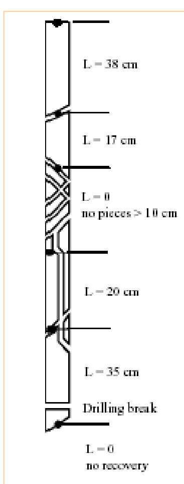
R Q D



Total Length of core run = 200 cms

$$\frac{\sum \text{Length of core pieces} > 10\text{cm length}}{\text{Total length of core run}}$$

$$\frac{38 + 17 + 20 + 35}{200} \times 100$$

$= 55\%$

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WHEN NO CORE IS AVAILABLE BUT DISCONTINUITY TRACES ARE VISIBLE IN SURFACE EXPOSURES OR EXPLORATION ADITS, THE R.Q.D. MAY BE ESTIMATED FROM NO. OF DISCONTINUITY PER UNIT VOLUME

$$RQD = 115 - 3.3\ Jv$$

- RQD IS A DIRECTIONAL DEPENDENT PARAMETER AND ITS VALUE MAY CHANGE SIGNIFICANTLY DEPENDING UPON THE BOREHOLE ORIENTATION.

- THE USE OF VOLUMETRIC JOINT COUNT CAN BE QUITE USEFUL IN REDUCING THIS DIRECTIONAL DEPENDENCE.

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SUPPORT RECOMMENDATIONS FOR TUNNELS IN ROCK (20 Ft to 40 Ft DIAMETER) BASED ON R.Q.D.(CONVENTIONAL TUNNELING) From Deere, et al28

RQD	ALTERNATIVE SUPPORT SYSTEMS Steel Sets	Rock Bolts	Shotcrete
EXCELLENT RQD > 90	None to Occasional light set Rock Load (0.0,0.3)B	None to Occassional	None to Occassional Local Application 2 - 3 in.
GOOD 75<RQD<90	Light Sets 5Ft to 6Ft CTR. Rock Load (0.3 to 0.6)B	Patterns 5 Ft to 6 Ft Centers	Occasional Local Application 2 to 3 inch
FAIR 50<RQD<75	Med. To Heavy Sets on 2 – 4 Ft CTR. Rock Load (0.6-1.3)B	Pattern 3-5Ft CTR	4 inch or more on crown & sides
VERY POOR RQD< 25 Excluding squeezing or swelling ground	Heavy circular sets on 2 ft CTR Rock Load (2.0 to 2.8)B	Pattern 3 Ft Center	6inch or more whole section combine with med to Heavy sets
VERY POOR Squeezing or swelling	Very Heavy Circular Sets on 2ft CTR Rock load upto 250ft	Pattern 2ft to 3ft	6inch or more whole section combine with med to Heavy sets

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- The R.Q.D. approach has limitations in areas where the joints contain thin clay fillings or weathered materials. These clay seams result in unstable rocks though the joints may be widely spaced giving a high R.Q.D. and low loads.

- In addition, R.Q.D. does not take direct account of other factors such as joint orientation which may result in Rock Loads in the form of " *STRUCTURALLY CONTROL - LED INSTABILITIES*" even for very good rock with very high R.Q.D.

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BIENIAWSKI (C.S.I.R.) RMR METHOD

The following six parameters are used to classify a rock mass using the *RMR* system:

- **Uniaxial compressive strength of rock material.**
- **Rock Quality Designation (*RQD*).**
- **Spacing of discontinuities.**
- **Condition of discontinuities.**
- **Groundwater conditions.**
- **Orientation of discontinuities.**

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A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of Values						
1	Strength of intact rock material	Point-load Strength index	>10 Mpa	4-10 MPa	2-4 Mpa	1-2 Mpa	For this low range uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90%-100%	75%-90%	50%-75%	25%-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2m	0.6 - 2 m	200-600mm	60-200mm	< 60mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not Continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1mm Highly weathered walls	Slickensided surfaces Or Gouge < 5 mm thick Or Separation 1-5m	Soft gouge > 5mm thick Or Separation > 5mm Continous		
	Rating		30	25	20	10	0		
5	Ground Water	Inflow per 10m Tunnel length (l/m)	None	< 10	10-25	25-125	> 125		
		(Joint water press)/(Major principal σ)	0	< 0.1	0.1 -0.2	0.2 – 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

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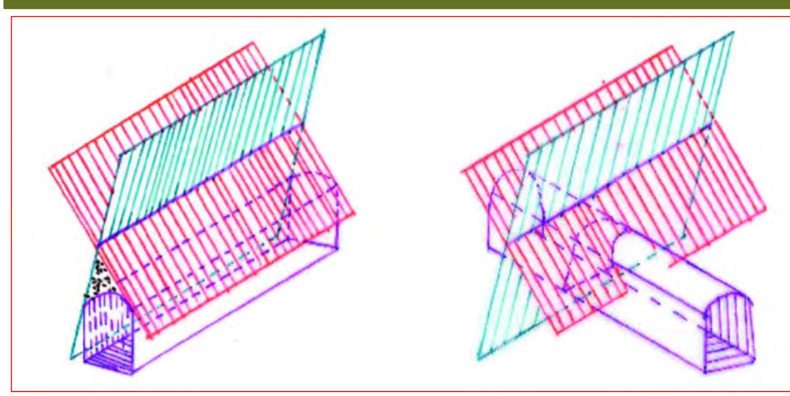
INFLUENCE OF ORIENTATION ON THE STABILITY OF UNDERGROUND OPENING



UNFAVOURABLE ORIENTATION

FAVOURABLE ORIENTATION

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MEANING OF ROCK CLASSES

Class Number	I	II	III	IV	V
Average stand-up time	20 yrs for 15m span	1 year for 10m span	1 week for 5m span	10 hrs for 2.5m span	30 min for 1m span
Cohesion of rock mass (kPa)	> 400	300-400	200-300	100-200	< 100
Friction angle of rock mass (deg)	> 45	35-45	25-35	15-25	<15

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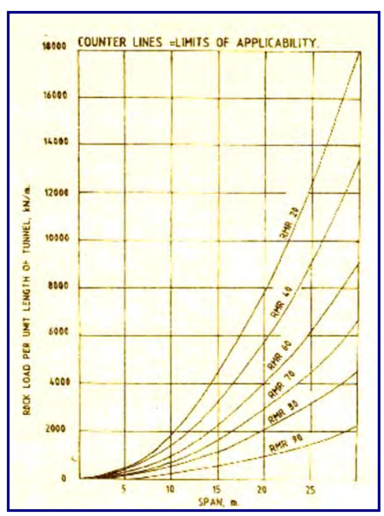
RMR Vs ROCK LOAD

Bieniawski himself did not relate RMR value to the prediction of rock load for design of tunnel support.

Unal (1983) suggested following formula to calculate rock load

$$P = \frac{[100 - RMR]}{100}\,\gamma B$$

Where
P = rock load or support pressure
γ = rock density
B = tunnel width



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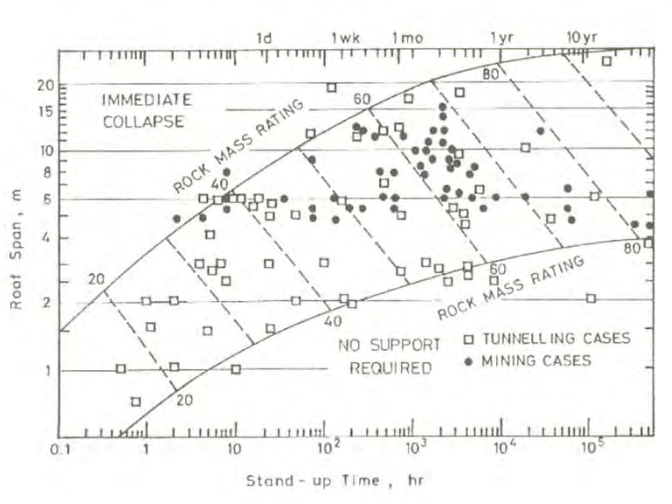

Stand-up time Vs roof span for various rock mass classes as per CSIR geomechanics classification (Bieniawski)

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GUIDELINES FOR EXCAVATION AND SUPPORT OF TUNNELS
(Size 10M and Construction by drill & Blast)

Rock mass Class	Excavation	Rock bolts (20mm diameter, fully grouted)	Shotcrete	Steel Sets
I – Very Good Rock RMR: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting		
II – Good rock RMR: 61-80	Full face,1-1.5m advance, Complete support 20m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required.	None.
III – Fair rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each Blast. Complete support 10m from Face.	Systematic bolts 4m Long, spaced 1.5-2 m In crown and walls with wire mesh in Crown.	50-100 mm in crown and 30 mm in sides	None.
IV – Poor rock RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m Long, spaced 1-1.5 m In crown and walls With wire mesh	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor Rock RMR : < 20	Multiple drifts 0.5-1.5 m Advance in top heading. Install support concurrently withExcavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75m with steel lagging and fore poling if required. Close invert.

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BARTON'S (N.G.I.) "Q" SYSTEM

Barton prepared an index for tunnelling quality of a rock mass and relate this rock mass quality "Q" to six parameters.

- RQD
- Number of joint sets J_n
- Joint roughness J_r
- Degree of joint alteration J_a
- Water inflow J_w
- Stress condition SRF

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$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

$\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{J_w}{SRF}$
↓	↓	↓
Block Size	**Inter Block Shear**	**Active Stress**

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DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	*RQD*	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use (3.0 × J_n)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use (2.0 × J_n)
J. Crushed rock, earthlike	20	

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3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	minimum strength.
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

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4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

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4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
b. Rock wall contact before 10 cm shear		
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12
c. No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed	6.0	
L. rock and clay (see G, H and J for clay	8.0	
M. conditions)	8.0 - 12.0	6 - 24
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0	
O. Thick continuous zones or bands of clay	10.0 - 13.0	
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0	

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5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		*SRF*	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

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DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

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ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\,J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

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ROCK MASS QUALITY FOR TUNNELLING

Q	Group	Classification
10 - 40		Good
40-100	1	Very good
100-400		Extremely good
400-1000		Exceptionally good
0.1-1.0		Very poor
1.0-4.0	2	Poor
4.0-10.0		Fair
0.001-0.01		Exceptionally poor
0.010-0.10	3	Extremely poor

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"Q" vs SUPPORT PRESSURE

$$P_{\text{roof}} = \frac{2 J_n^{1/2} \, Q^{-1/3}}{3 J_r} \qquad P_{\text{h}} = \frac{2 J_n^{1/2} \, Q_w^{-1/3}}{3 J_r}$$

Proof = ultimate roof support pressure in Kg/cm2

Ph = ultimate wall support pressure in Kg/cm2, and

Qw = wall factor

Range of Q	Wall Factor Qw
> 10	5.0 Q
0.1 - 10	2.5 Q
< 0.1	1.0 Q

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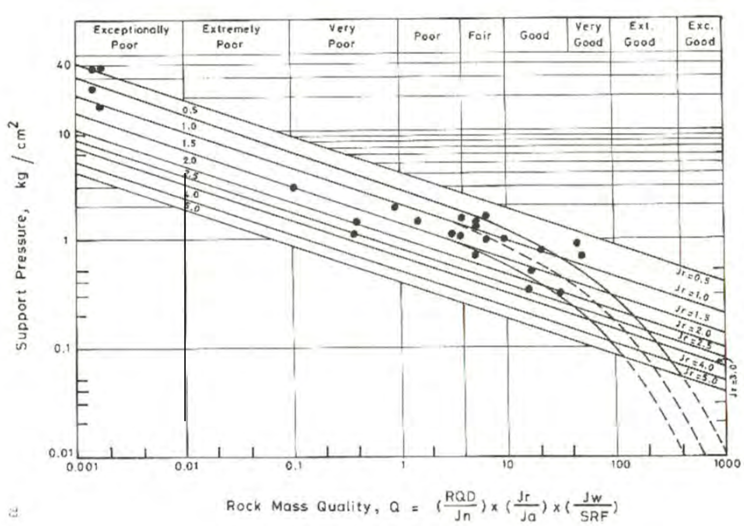
CORRELATION BETWEEN SUPPORT PRESSURE & "Q"



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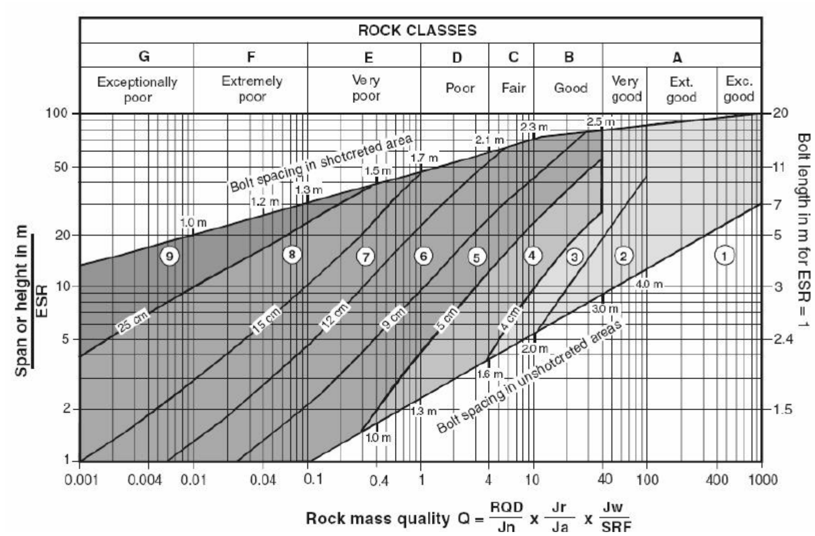

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REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9 - 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

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A SAMPLE PROBLEM

A permanent mine opening with span 15.0m

RQD 90

Jn 4

Jr 3

Ja 1.0

Jw 1.0

SRF 15

From the above data , find (i) Rock mass quality Q value

(ii) Rock support rquired

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<u>SOLUTION</u>

$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF)$

$= (90/4) \times (3/1) \times (1/15)$

$= 4.5$ (Fair Rock)

<u>Rock Support Required</u>

For permanent mine opening ESR = 1.6

Equivalent Dimension ,De= (Span/ESR) = 15/1.6 =9.4

For this De=9.4 & Q=4.5, from Barton's chart

The support category (4) with rock bolt spacing 2.3 m and unreinforced shotcrete 40mm to 50mm.

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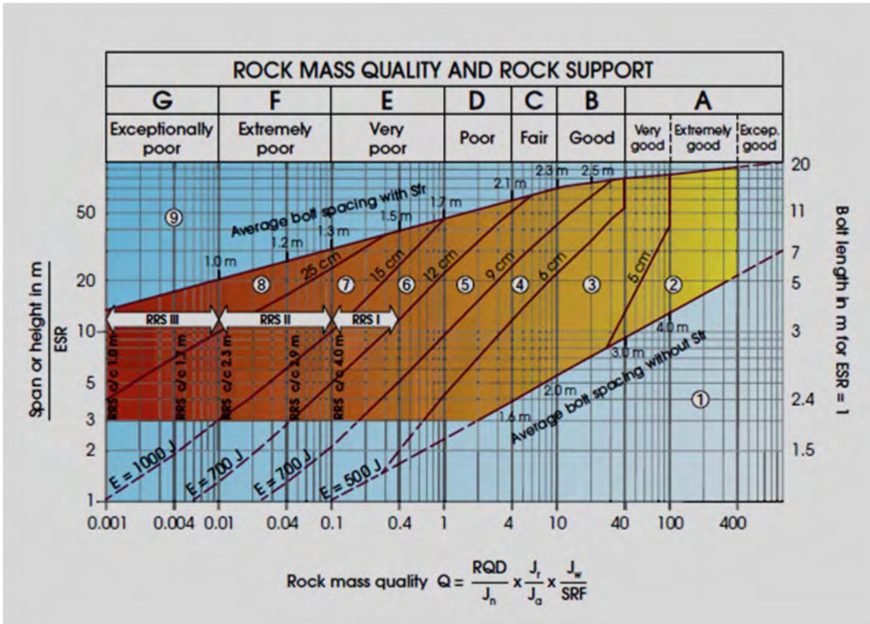

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Support categories

1. Unsupported or spot bolting
2. Spot bolting, **SB**
3. Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B+Sfr**
4. Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500)+B**
5. Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700)+B**
6. Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E700)+RRS I +B**
7. Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000)+RRS II+B**
8. Cast concrete lining, **CCA** or **Sfr (E1000)+RRS III+B**
9. Special evaluation

Bolts spacing is mainly based on Ø20 mm

E = Energy absorbtion in fibre reinforced sprayed concrete

ESR = Excavation Support Ratio

Areas with dashed lines have no empirical data

RRS - spacing related to Q-value

I
Si30/6 Ø16 - Ø20 (span 10m)
D40/6+2 Ø16-20 (span 20m)

II
Si35/6 Ø16-20 (span 5m)
D45/6+2 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

III
D40/6+4 Ø16-20 (span 5 m)
D55/6+4 Ø20 (span 10 m)
D70/6+6 Ø20 (span 20 m)

Si30/6 = Single layer of 6 rebars, 30 cm thickness of sprayed concrete

D = Double layer of rebars

Ø16 = Rebar diameter is 16 mm

c/c = RSS spacing, centre - centre

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– SYSTEM
MPLE
LCULATION

			good	fair
BARTON'S "Q" assume		Q	10	4
Joint roughness		Jr	1.5	1.5
Barton'sMax support pressure	KN/m^2	Pr=(2/Jr)/Q^(1/3)*10	61.89	83.99
Design rock load				
		Good Rock	**61.89**	KN/m^2
		Fair Rock	**83.99**	KN/m^2
2.Design of Rock Bolt (Bolt Capacity, w.r.t design rock load)			Good rock	Fair rock
Design rock load on roof (KN/m^2)			61.89	83.99
Finished Width of Cavern	D		13.9	13.9
Excavation thickness	Te		0.6	0.6
Excavation span of section (m)	B		15.1	15.1
Excavation height of section (m)	H		15.1	15.1
Mean rock density	Wr		27	27
Excavation support ratio	ESR		1	1
Equivalent dimension	De		15.1	15.1
Bolt length in m	Lroof		4.265	4.265
			Provide 8m long Rock bolts	
Yield strength of bolt (KN/m^2)	Fy		500	500
Allowable stress of bolt (KN/m^2)	Fa		275.0	275.0
Spacing of 32 dia bolt (sq parten)<l/2)	S		1.89	1.62

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EXCAVATION SUPPORT RATIO (ESR)

- A.Temporary mine openings, etc. 3 – 5
- B.Vertical shafts - circular section - 2.5
  rectangular / square section - 2.0
- C.Permanent mine openings, water tunnels for hydropower
- (except high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.
  1.6
- D.Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels etc.
  1.3
- E. Power stations, major road & railway tunnels, civil defence chambers, portals, intersections, etc. 1.0
- F.Underground nuclear power stations, railway stations, sports and public facilities, factories etc.
  0.8

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COMMONLY ADOPTED ROCK SUPPORT SYSTEMS ARE THE FOLLOWING

- ROCK BOLTS / ANCHORS
- CABLE SUPPORTS
- SHOTCRETE : PLAIN / MESH / FIBRE REINFOCED
- STEEL RIBS

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LENGTH OF ROCKBOLT

L roof = 2 + (0.15 x SPAN/ESR)

L wall = 2 + (0.15 x HEIGHT/ESR)

- SPACING OF ROCK BOLTS TO BE COMPUTED BASED ON THE SUPPORT PRESSURE

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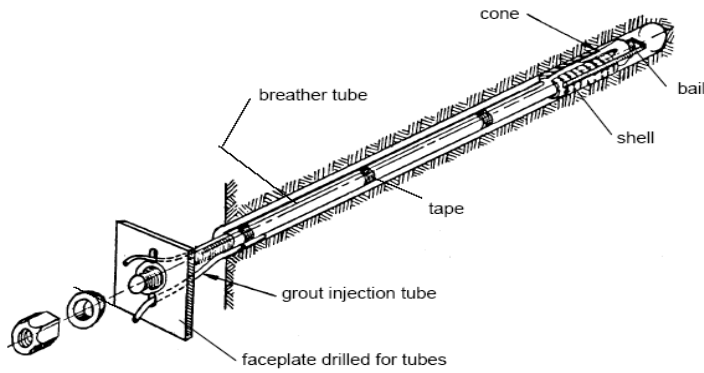

MECHANICALLY ANCHORED ROCK BOLT

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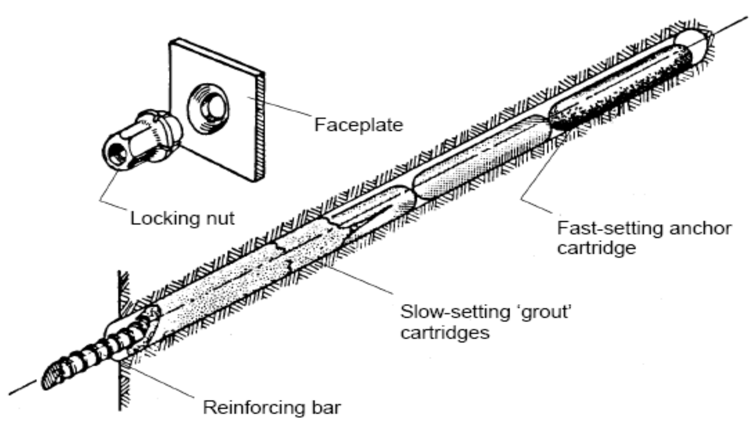

RESIN END ANCHORED ROCK BOLT

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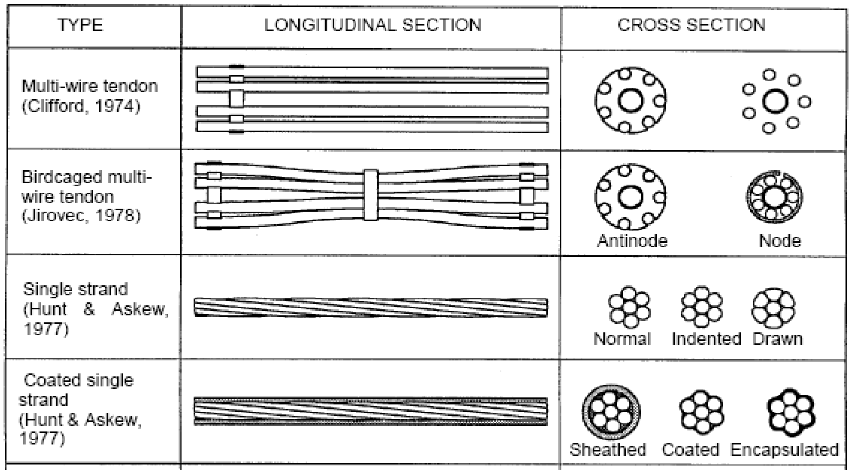

TYPE	LONGITUDINAL SECTION	CROSS SECTION
Multi-wire tendon (Clifford, 1974)		
Birdcaged multi-wire tendon (Jirovec, 1978)		Antinode Node
Single strand (Hunt & Askew, 1977)		Normal Indented Drawn
Coated single strand (Hunt & Askew, 1977)		Sheathed Coated Encapsulated

CABLE SUPPORTS

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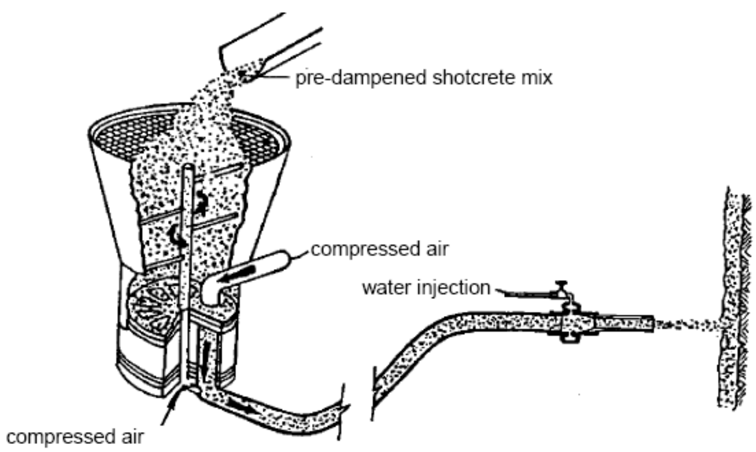

DRY MIX SHOTCRETE

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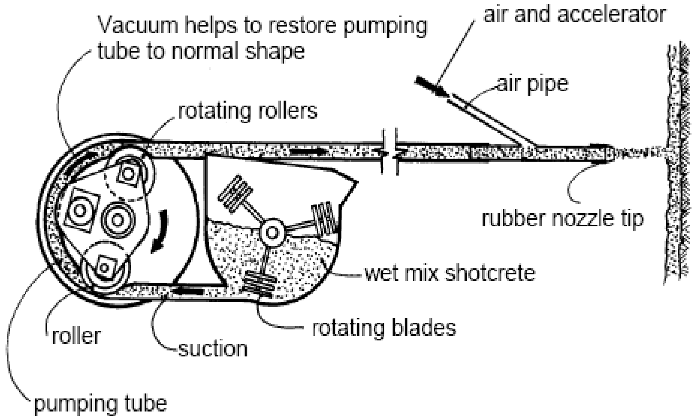

WET MIX SOTCRETE

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Central Water Commission (CWC)

WHERE TO LOOK FOR–

A. WEBSITE OF NORWEGIAN INSTITUTE OF TECHNOLOGY (NGI)
B. ENGINEERING IN ROCKS FOR SLOPES FOUNDATION AND TUNNELS – T RAMAMURTHY
C. IS CODES : 4880_1 TO 7, 5878_1 TO 7, 13365_1 & 2 , 15026
D. HANDBOOK BY NGI ON BARTON'S Q SYSTEM
E. ON-GOING CONSTRUCTION H E PROJECTS PUNATSANGCHU-I & II, BHUTAN (DESIGN CONSULTANCY BY CWC)
F. LIBRARY OF CENTRAL WATER COMMISSION

THANK YOU

ABHINAV SRIVASTAVA
Deputy Director
HCD (E&NE) Dte , CWC (HQ)

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HYDRAULIC MODELLING TECHNIQUES

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ABSTRACT

The construction of dams involves huge capital cost and recurring expenditure of maintenance. The dam hydraulics should be optimized functionally and economically before the execution of construction work. The most reliable method of investigation of flow over spillways is performing experiments on scaled physical models. A physical model is a scaled representation of a hydraulic flow situation. Physical hydraulic models are commonly used during design stages to optimize a structure and to ensure a safe operation of the structure. They have an important further role to assist non-engineering people during the `decision-making' process. A hydraulic model may help the decision-makers to visualize and to picture the flow field, before selecting a `suitable' design.

In civil engineering applications, a physical hydraulic model is usually a smaller-size representation of the prototype (i.e. the full-scale structure). The rapidly varied flows with complex geometry, supercritical velocities due to high heads leading to cavitations damages, intense turbulence causing hydrodynamic forces on the spillway structure are normally investigated by physical models. Various factors influencing the design and selection of scales for Froudian models, construction methodology, measurement techniques adopted and planning of model studies are described in this lecture note.

Hydraulic Modelling

A lack of understanding of physical processes or complex boundary conditions in many fluid mechanics/ hydraulics problems which are not amenable to numerical or analytical techniques are investigated by physical models. Physical model studies are indispensable tools to optimize various components of reservoir and appurtenant structures. The hydraulic design of various components of a river valley project involves two types of problems viz. site specific problems and problems connected with complex hydraulic flow phenomena. The site-specific problems are due to topography at the site, availability of foundation, nature of soil and rock strata etc. The problems associated with complex flow phenomena are many viz. non uniform flow in the approach portion creating vortices, rapidly varied flow because of complex geometry, high velocities due to high heads leading to cavitations damages, high turbulence causing hydrodynamic forces on the structure and erosion of the river bed and banks downstream, flow induced vibration for wide range of operating conditions. These problems at present cannot be dealt analytically and therefore they have to be tackled by conducting studies on physical models of these structures. For number of decades the art of hydraulic modelling has been an important tool in solving complex hydraulic problems. It entails, with a degree of sophistication that varies with the objective of the investigation, the use of a scaled model for replicating flow and fluid-transport processes in diverse natural flow systems and for evaluating the performance of hydraulic structures and hydraulic machines. The following situations are common subjects for modeling: water movement and sediment transport in rivers and coastal zones; the hydraulic performance of water intakes, spillways, and outlets; flow around various objects; flow through, or in, various conduits or flow-regulating devices; performance of turbines, pumps, and other hydro machines; performance of floating structures or ships; and effluent-mixing processes.

An advantage of a hydraulic model is its potential capacity to replicate many features of a complicated flow situation. There are many situations for which there is little recourse other than hydraulic modeling to make design or operational decisions involving expensive and complex hydraulic works. Such situations particularly arise when, for a variety of reasons, complex flow patterns or intricate transport processes are involved, and reliable answers cannot be obtained by means of analytical solution or computer simulation.

Modelling Techniques

- Terms of reference for model studies
- Method of solution
  a. Physical model
  b. Combination of physical and mathematical model
  c. Desk study
- Number and types of models
  a. Models with fixed/movable bed
  b. Three/two dimensional models
- Scale of model
- Factors influencing scale of model
  a. Space
  b. Head
  c. Discharge

Similitude and Design of Spillway Model

Principles of similitude form the basis of designing a model so that model results can be converted to prototypes. Hydraulic similitude is indispensable in physical modelling, regardless of whether the model study involves basic research of fluid flow or hydraulic design of structures such as spillways. Full model –prototype similitude requires satisfaction of the following conditions.

1. Geometric similitude, whereby the ratio of all homologous (geometrically equivalent) length dimensions is equal and where only similarity in form is involved;
2. Kinematics similitude, whereby at geometrically homologous points in model and prototype, velocities and accelerations are in a constant ratio; and,
3. Dynamic similitude, whereby, in addition to kinematics similitude, the force polygons are similar at geometrically equivalent points for model and prototype.

If dynamic similitude is satisfied, kinematics similitude automatically follows. In the following discussion, the subscripts r, m, and p denote ratio, model, and prototype values, respectively. Uniform laminar flow is a relatively uncommon exception for which flow inertia is not important. The discussion pertains to situations of geometric similitude, for which all length scales are equal. The primary parameter for geometric similitude is the length ratio

$$L_r = \frac{L_p}{L_m} \tag{1}$$

which must be constant for all parts of the model and prototype. As a consequence of geometric similitude, the area, A, ratio is

$$A_r = L_r^2 \tag{2}$$

and the volume, $\forall$, ratio is

$$\forall = L_r^3 \tag{3}$$

For kinematics similitude, the velocity ratio, Ur, and the acceleration ratio, Ar must be constant at all homologous points of the model and the prototype. The commensurate ratios are

$$U_r = \frac{L_r}{T_r} \tag{4}$$

$$a_r = \frac{U_r}{T_r} = \frac{L_r}{T_r^2} \tag{5}$$

in which the time ratio, $T_{r,}$ is

$$T_r = \frac{T_p}{T_m} \tag{6}$$

Dynamic similitude involves the force ratio, Fr Forces arise in modeling due to a variety of physical phenomena (friction, surface tension, pressure, gravity, and so on). Inertial force is always important when flows accelerate or decelerate because of changes in flow area or turbulence.

Newton's second law relates inertial force to mass, M, and acceleration, a. Expressed in ratio form

$$F_r = M_r a_r \tag{7}$$

The mass ratio can be written in terms of a density, ρ, ratio and the length ratio

$$M_r = \rho_r \forall_r = \rho_r L_r^3 \tag{8}$$

Thus Newton's second law can be expressed in the following significant form

$$F_r = \rho_r L_r^3 \frac{U_r}{T_r} \tag{9}$$

Because the time ratio can be written from Eq. (4) in terms of the length ratio and velocity ratio, Eq. (9) reduces to

$$F_r = \rho_r L_r^2 U_r^2 \tag{10}$$

The inertial force as expressed in Eq. (10) is relevant to any flow situation, except uniform laminar flow.

When a scaling law is valid, a condition of similarity exists between model and prototype. The simple and fundamental nature of the foregoing scale ratios enable the similitude principles to provide scaling laws with which the data obtained with relatively inexpensive model tests may be extrapolated accurately to aid the design of usually expensive and large prototypes.

Mainly there are four types of forces acting on the fluid, those due to gravitation, viscosity, surface tension and elasticity. Perfect similarity of fluid motions and hydrodynamics between a model and its prototype is practically an impossible task, but results are invariably compatible within the desired accuracy. It has now been established that application of Froude law leads to kinematic and dynamic similitude simulating the fully developed turbulent flow in an undistorted geometrically similar model. Success in achieving the desired results from a model study in the shortest time and with the least cost depends largely on the design of the model. The first and most important step in the design is the careful selection of a model scale. Small models may be used advantageously in preliminary studies to give direction to the primary investigation. Although, a large model is more useful than a small one, and improves the accuracy of measurements, but at some point the cost and difficulty of operation will offset the advantage of large size.

1. Reynolds number, $\mathrm{Re} = \dfrac{U_0 L}{\nu}$
2. Froude number, $F_r = \dfrac{U_0}{\sqrt{gL}}$
3. Weber number, $We = \dfrac{\rho U_o^2 L}{\sigma}$
4. Euler number, $Eu = \dfrac{\rho U_o^2}{p_o}$

Spillway models are built geometrically similar to their prototypes. The force of gravitation causes flow of water in open channel and hence dynamic similitude is closely approximated according to the Froude‟s law. With the same fluid (water) in the prototype and model complete similarity of all forces is not possible resulting in scale effects. Following are the important phenomena for which scale effects are encountered in modelling of spillways:

friction, air entrainment, turbulence, cavitation, fluid-structure interaction and local scour downstream of spillway.

In a Froudian model where viscous and surface tension forces are ignored, scale effects may influence the results if a very small model is used. This is because the effects of viscous and surface tension forces become increasingly important as the scale of a phenomenon reduces. Small Froudian models should be avoided to ensure that viscous and surface tension forces do not distort the Froudian similarity. For example, a model should not be so small in size that a flow, which is turbulent in the prototype, becomes laminar in the model. The Froude number and the Reynolds number each define unique relationships between the scale ratios L_r, T_r and U_r. They cannot be simultaneously satisfied without manipulating fluid properties, which at best is a difficult proposition. In Froudian models, Reynolds number is always smaller than the prototype value. It is established by many investigators that if a model is big enough to simulate large eddies (inertial eddies) so as to ensure turbulent flow conditions in the model, many hydraulic parameters are independent of Reynolds number if $R_e > 5x10^5$. It is believed that a model Reynolds number of at least $5x10^5$ and above will minimize the scale effects.

This essentially requires that the ratio of inertia to gravity forces be the same in model and prototype. It also may be viewed as a ratio of water velocity, U, to shallow-water wave velocity, $(gY)^{1/2}$, in a channel of depth Y. The Froude-number similarity criterion prescribes

$$F_r = \frac{Fr_p}{Fr_m} = \frac{U_r}{\sqrt{Y_r}} = 1$$

Note that, as most models are subject to the same gravitational field that prevails at full scale, g_r = 1. The resultant scales consequent to Froude number criterion (above Eq.) are summarized in the following table. The Froude-number criterion sets the scale ratios, other than geometric scale.

Scale Relationship Based on Froude Number Similitude with ρ_γ = 1

Variable	Relationship	Scale	Scale for Vertically Distorted Model, $G = X_r / Y_r$
length	L = length	$L_r = X_r = Y_r$	horizontal length: $L_{rx} = X_r$ vertical length: $L_{ry} = Y_r$
slope	$S = \frac{\text{horizontal length}}{\text{vertical length}}$	$S_r = \frac{L_r}{L_r} = 1$	$S_r = \frac{Y_r}{X_r} = \frac{1}{G}$
velocity	$U = \frac{\text{length}}{\text{time}}$	$U_r = L_r^{1/2}$	$U_r = Y_r^{1/2} = \left(\frac{X_r}{G}\right)^{1/2}$
time	$t = \frac{\text{length}}{\text{velocity}}$	$t_r = \frac{L_r}{U_r} = L_r^{1/2}$	horizontal motion: $t_{Xr} = \frac{X_r}{Y_r^{1/2}} = (GX_r)^{1/2}$ vertical motion: $t_{Yr} = \frac{Y_r}{Y_r^{1/2}} = Y_r^{1/2}$
acceleration	$a = \frac{\text{velocity}}{\text{time}}$	$a_r = \frac{U_r}{t_r} = \frac{L_r^{1/2}}{L_r^{1/2}} = 1$	horizontal motion: $a_r = \frac{U_r}{t_{Xr}} = \frac{1}{G}$ vertical motion: $a_{Yr} = \frac{U_r}{t_{Yr}} = 1$
discharge	Q = velocity × area	$Q_r = U_r A_r = L_r^{5/2}$	horizontal component: $Q_{Hr} = U_r Y_r X_r = GY_r^{5/2}$ vertical component: $Q_{Vr} = U_r X_r X_r = G^2 Y_r^{5/2}$
Force	F = mass × acceleration	$F_r = \rho_r L_r^2 L_r = L_r^3$	horizontal component: $F_{Hr} = a_{Hr} Y_r X_r^2 = Y_r^2 X_r$ vertical component: $F_{Vr} = a_{Vr} Y_r X_r^2 = Y_r X_r^2$
Pressure and Stress	$p = \sigma = \frac{\text{force}}{\text{area}}$	$p_r = \sigma_r = \frac{\rho_r L_r^2 L_r}{L_r L_r} = L_r$	horizontal component: $p_{Hr} = \sigma_{Hr} = \frac{Y_r^2 X_r}{Y_r X_r} = Y_r$ vertical component: $p_{Vr} = \sigma_{Vr} = \frac{Y_r X_r^2}{X_r^2} = Y_r$
Reynolds number	$Re = \frac{UL}{\nu}$	$(Re)_r = L_r^{1/2} L_r = L_r^{3/2}$	$(Re)_{ry} = Y_r^{1/2} Y_r = Y_r^{3/2} = \frac{X^{3/2}}{G}$

SELECTION OF SCALE FOR A SPILLWAY MODEL

SALMA DAM SPILLWAY, AFGHANISTAN

SL.NO.	SCALE	HEAD	SPAN	DISCHARGE		Depth of overflow MWL-CrestEl
		m	1	CUMEC	CFS	
	PROTO	95.84	8M	2100		12.34m
1	30	3.194667	0.266667	0.426006	15.04429	0.411
2	45	2.129778	0.177778	0.154592	5.459383	0.274
3	50	1.9168	0.16	0.118794	4.195172	0.247
4	55	1.742545	0.145455	0.093608	3.305735	0.224
5	60	1.597333	0.133333	0.075308	2.65948	0.206
6	75	1.277867	0.106667	0.043109	1.522375	0.165
7	90	1.064889	0.088889	0.027328	0.965092	0.137
8	100	0.9584	0.08	0.021	0.741609	0.123

OMKARESHWAR DAM SPILLWAY, MADHYA PRADESH

SL.NO.	SCALE	HEAD	SPAN	DISCHARGE		Depth of overflow MWL-Crest El m
		m	1	CUMEC	CFS	
	PROTO	49.62	20m	88315		20.02
1	50	0.9924	0.400	4.996	176.43	0.400
2	75	0.6616	0.267	1.813	64.02	0.267
3	90	0.551333	0.222	1.149	40.59	0.222
4	100	0.4962	0.200	0.883	31.19	0.200
5	110	0.451091	0.182	0.696	24.58	0.182
6	125	0.39696	0.160	0.506	17.85	0.160
7	150	0.3308	0.133	0.320	11.32	0.133

Surface-tension effects start to become important if *We* is of order 100 or less. This occurs when the radius of surface curvature is small in comparison to liquid thickness or depth, for instance, for liquid drops, bubbles, capillary flow, ripple waves, and very shallow flows in small hydraulic models. The air water flow is a function of Weber number. The hydraulic models constructed according to Froudian criterion would not be able to simulate the air entrainment. In order to simulate air entrainment in the model, the models are required to be constructed to a scale ranging from 1:10 to 1:15 so as to achieve Weber number of about 500.

The scale of the model is chosen depending upon availability of space, discharge and head. Spillway models are scaled to provide flow depth over the crest of at least 75 mm for the design normal operating head to reduce the effect of viscosity and surface tension. In general, large models rather than small models should be built, as permitted by available space, operating head and water supply. Sometimes, cost and operational difficulties dictate the selection of model scale. The model scale for medium sized spillway would be around 1:50 to 1:60.

Classification of Spillway Models

Spillway models can be classified either as two-dimensional sectional model built in a glass sided flume or three-dimensional comprehensive model constructed in a model tray and incorporating entire spillway, non-overflow dam, part of reservoir and river downstream including other structures. The sectional model, usually built to a large size and incorporating 2 to 4 spans is required for determining discharging capacity, pressures on crest and other appurtenances and for testing various alternative designs of energy dissipater. Detailed measurements of discharge, pressure, velocity and force as well as facility of visual observation of flow conditions through glass are the main advantages of a sectional model. Desired modifications could easily be carried out in a sectional model, rather than in a 3-D model. The comprehensive model enables study of general flow conditions upstream and downstream of the spillway, pattern of operation of spillway gates, flow conditions in the vicinity of ancillary structures such as training walls, power house tail race, earth dam toe etc. Studies with erodible bed downstream of spillway also give qualitative indication of scour and requirement of protection.

Data Requirements for Model Studies

For conducting model experiments, it is necessary to obtain correct information from the prototype. The entire operation of the model depends on the equality of the prototype data. The data would help in establishing the model prototype conformity pattern and to enhance the predictability of the model. Generally, the following prototype data would be required for planning, construction of spillway models and conducting model studies.

1. Maximum design outflow discharge for spillway and energy dissipater.

2. Gauge-discharge (Tail Water Rating) curve at about 200 to 300 m downstream of the spillway up to the maximum outflow discharge.
3. Index plan showing location of dam and course of river for about 1 km upstream and 1 km downstream, water spread, tributaries upstream and downstream of the site, important structures etc.
4. Cross sections of the river at about 50 m interval for a distance of 1000 m upstream and downstream of the dam axis. If the approach is curved immediately upstream, the cross section should extend at least 150 m beyond the curve.
5. A plan showing river course, dam complex, power intake, position of river cross sections and base line
6. Layout plan : Dam layout plan showing the chainages along the dam axis for the important structures such as left and right end of the spillway with reference to a baseline connected to the dam axis and location and orientation of the power intake.
7. Spillway section with details such as upstream and downstream crest profiles giving equations and radii of curves, tangent points, slopes and dimensioned details of energy dissipator. cross sections of the non-overflow section of the dam
8. Details of spillway gates and piers in plan and sections including distance of trunnion axis of radial gate with reference to dam axis/crest axis, gate seat elevation, geometric profile of breast wall and details of stoplog groove.
9. Details of power intake including plan and sections of bellmouth entrance, transition, trashrack piers and rib beams, dimensions of gate grooves.

Construction Methodology of Model

Having determined the scale ratio, construction of the model requires following considerations :

- Materials of construction
- Construction accuracy and other requirements
- Extent of river topography to be reproduced in the model including nearby structures

A model need not be made of the same materials as the prototype. If surfaces over which water flows are reproduced in shape and the roughness of the surfaces is approximately to scale (in fact smoother in the model than corresponding to prototype roughness), the model will usually be satisfactory. Generally, the riverbed is made up of smooth cement plaster;

spillway, non-overflow section of the dam etc in masonry with neat plaster, spillway piers in teakwood, radial gates in sheet metal and outlets are fabricated in transparent perspex.

Close tolerances, particularly in critical areas such as spillway crests, tangent points, energy dissipating appurtenances, model dimensions etc are essential. Greatest accuracy should be maintained where there will be rapid changes in direction of flow and very high velocities occur. The profiles of spillways and their allied structures are finished to their final shape with the help of metallic templates fixed in alignment and elevation. Piezometers are generally welded to the templates so that their alignments are secured. The finishing of piezometers in models should be done meticulously to prevent measurement errors that would result from improper installation. Complicated curves for bell mouths of sluice spillways, breastwalls, bends and transitions can be made from Perspex which has been heated in oven and reshaped by pressing between the male and female concrete moulds.

It is not possible to reproduce the entire reservoir upstream of spillway nor it is necessary to do so. For the spillways located in the main river gorge with practically straight river course, reproduction of about 600 to 800 m reach is usually adequate. Where the river has appreciable curvature immediately upstream of dam, or where the spillway is located on a flank, so that obliquity of flow approaching the spillway is likely to occur, special care must be taken to incorporate these features. On the downstream, the river reach to be incorporated would be slightly beyond the section of stage-discharge measurement in the prototype.

Operation of Model

Once the model is ready for experimentation, the operating programme of the model should be carefully planned to evaluate the performance of the proposed design. The operating programme can be divided into two phases:

- Adjustment phase
- Experimental phase

The adjustment phase includes preliminary trials to identify model defects and inadequacies. The need for partial redesign, revision or shifting of measuring instruments is often indicated by these trial runs. The experimental phase includes regular model studies after removing all the defects observed during the adjustment phase. Generally, the following aspects are studied on spillway models:

Approach flow conditions: To observe if the flow approaching the spillway is generally uniform and if not, to find out its effect on functioning of spillway, dam and differential pressures on spillway piers etc.

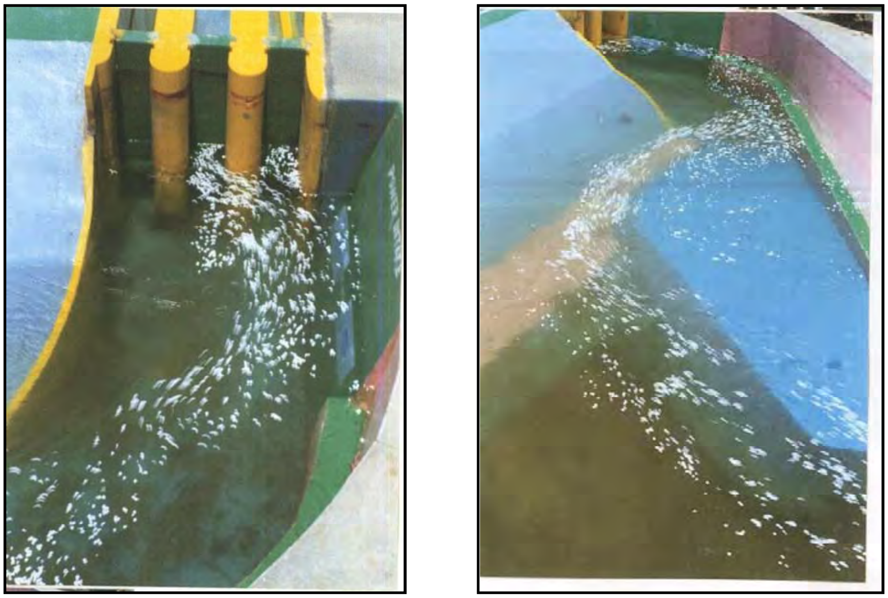

Photo 1: Approach Flow Conditions u/s of spillway

Discharging capacity: Calibration curves for spillway and schedule of operation of gates

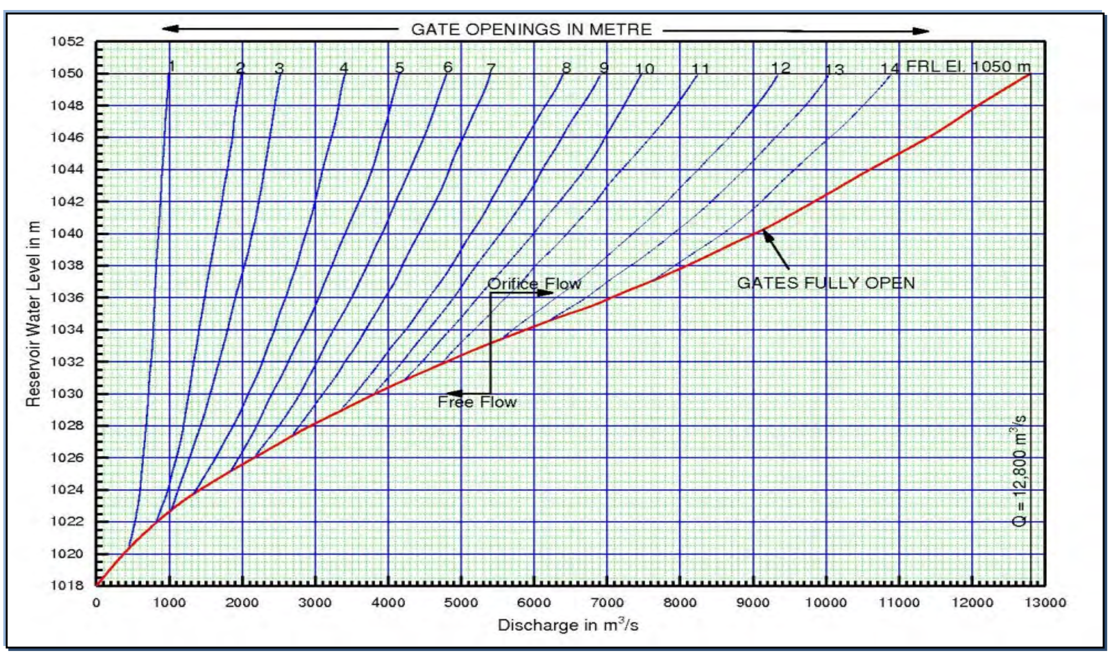

Figure 1: Typical discharging capacity curves

Pressure Distribution: Pressures on crest profile and other appurtenant structures to ascertain that no dangerous sub-atmospheric pressures leading to cavitation damage exist

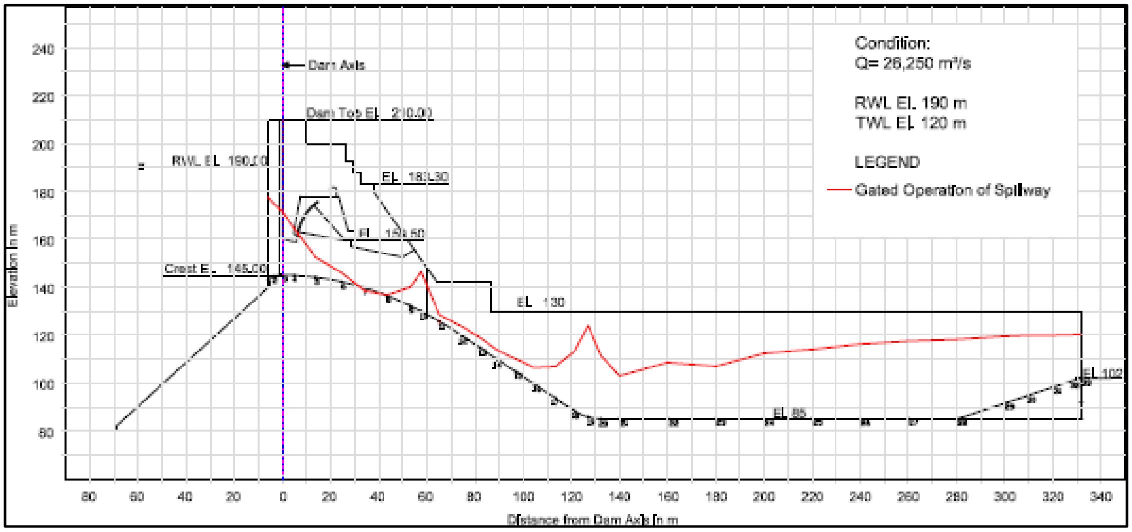

Figure 2: Typical pressure distribution along centre line of spillway with Stilling basin as EDA

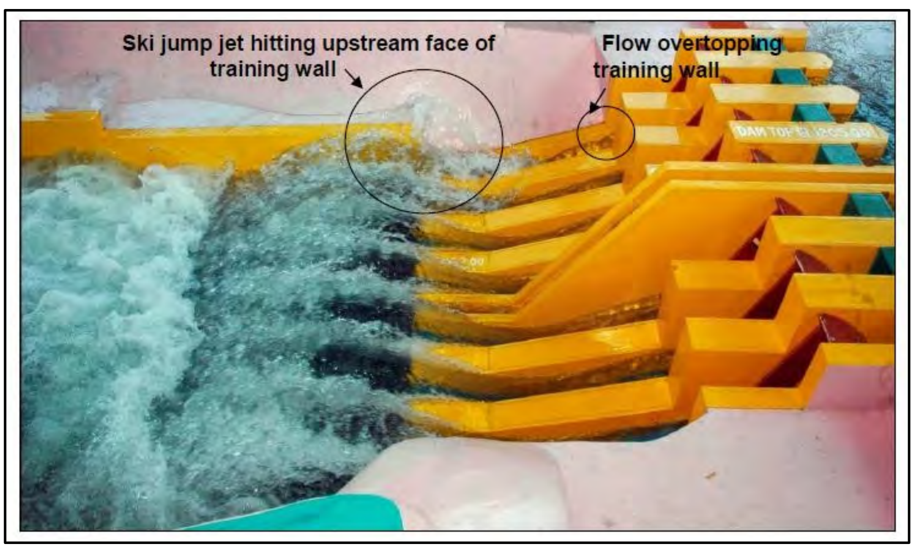

Photo 2: Height of training walls vis-à-vis water surface profiles

Design of training walls and divide walls: Water surface profiles along the training walls and divide walls to finalize their profiles.

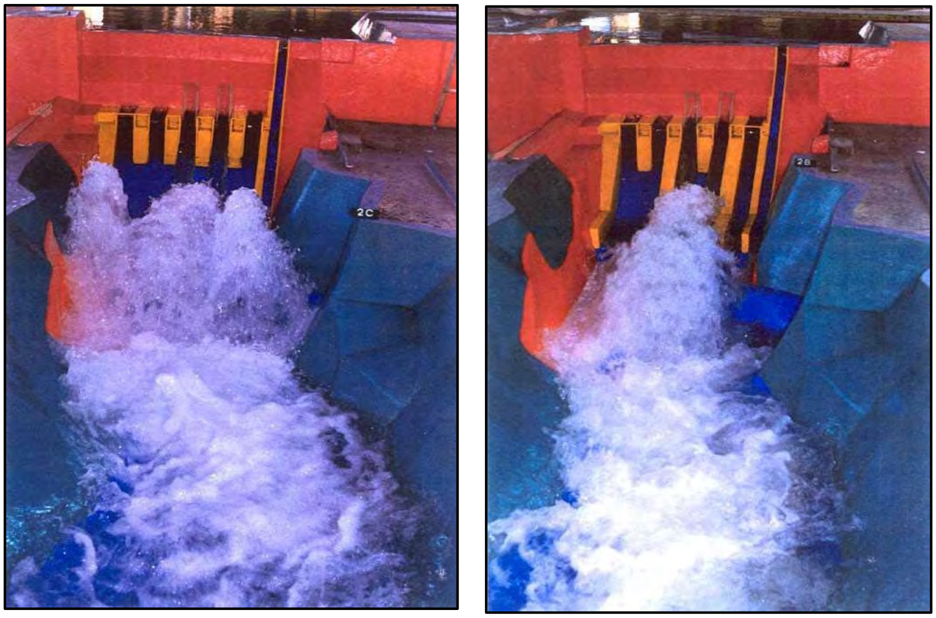

Photo 3: Necessity of divide walls

Operation of Gates: Schedule of operation of gates

Photo 4: Spillway Operation with divide walls

Energy dissipation efficiency: Performance of energy dissipater assessed from visual observation, development of scour, flow concentration, velocities, etc.

Photo 5: Spillway Operation with ski-jump bucket as EDA

Photo 6: Spillway Operation with stilling basin as EDA

Scour d/s of Spillway: The flip bucket type of energy dissipator downstream of spillway is suitable for locations where the tail water depth is low and hard rock is available. It dissipates energy by throwing the jet of water at a sufficient distance away from the spillway bucket. Scour hole is usually formed downstream of the point of impingement of ski-jump jet. The main parameters affecting the performance of the ski-jump bucket are radius and lip angle of the bucket, discharge intensity, hydraulic head acting at the bucket lip and tail water level. Retrogression of the scour hole may endanger stability of the structure. Generally, concrete apron of about 15m to 20m is provided to protect the toe of the dam from undermining due to the flow cascading over the lip of the bucket. Analysis of scouring in rocky bed downstream of spillway with ski-jump bucket is complicated due to the complex hydraulic and geological conditions. An accurate evaluation of the parameters determining the erosion resistance of the rock is difficult. The prediction of the scour depth to ensure the safety of dam is necessary.

Pre-excavated plunge pools are provided d/s of ski jump bucket to minimize the scour damage particularly important for the spillways in Himalayan region where geology is fragile and gorges are narrow. The pre-excavated plunge pool geometry need to be evolved from model studies and has to be based on expected natural scour geometry. The design is based on study of pulsating pressure propagation into slab joints/seals, dynamic propagation underneath concrete slabs due to water hammer effects.

Various Empirical Formulae for Scour Depth

Sr. No.	Researcher	Formula
1	Damle et al.(1966)	$t = A(qH)^{0.5}$ Where, t= scour depth in m A= 0.65 for Ultimate scour A= 0.54 Probable maximum scour A= 0.36 minimum expected scour
2	Chian Min Wu (1973)	$\frac{t}{h} = 2.11\left(\frac{q}{\sqrt{gh^3}}\right) \wedge 0.51$
3	Martins (1975)	$\frac{t}{h} = 2.976\left(\frac{q}{\sqrt{gh^3}}\right) \wedge 0.6$
4	Wang Shixia(1981)	$\frac{t}{h_r} = 2.44\ h_{cr}^{0.69}\ H^{0.11}$
5	Chen (1990)	$t = K_r\ q^{0.5}\ h^{0.25}$
6	Veronese (1937)	$t = 1.90\ q^{0.54}\ h^{0.225}$
7	Mason (1984)	$t = 3.27\frac{q^{0.6} H^{0.05} h_{dr}^{0.15}}{g^{0.3}\ d_m^{0.1}}$
		Where, q = Discharge intensity in m^3/s H = (MWL – Bucket lip) in m h = (MWL – TWL) in m K_r = coefficient depending upon rock strata d_m = Mean diameter of bed material in m h_{ds} = Downstream water depth in m

Photo 7: Scour Pit at Model

Assessment of Discharge for Construction Stages of Spillway

The construction of concrete dam is phased out over several years. During the intervening period floods are required to be passed over the partly constructed dam blocks, because the diversion arrangements provided cater only for normal river flows which are 5 to 10% of 1:100 year outflow flood. The floods are passed over the partly completed spillway blocks having various elevations. Ideally all the blocks should have been constructed to the same level to have favourable flow conditions upstream and downstream of the partly completed spillway blocks. However, it has been observed that the construction of each spillway block proceeds independently and due to many restrictions imposed on the construction for placing of concrete at site, the spillway blocks at the onset of monsoon are at varying levels. The estimation of upstream reservoir water level for the design construction stage flood is very crucial. This is particularly so because the coefficient of discharge that would realize in such a condition would depend on width and length of the blocks, the elevation difference between adjoining blocks, and the approach depth of the blocks.

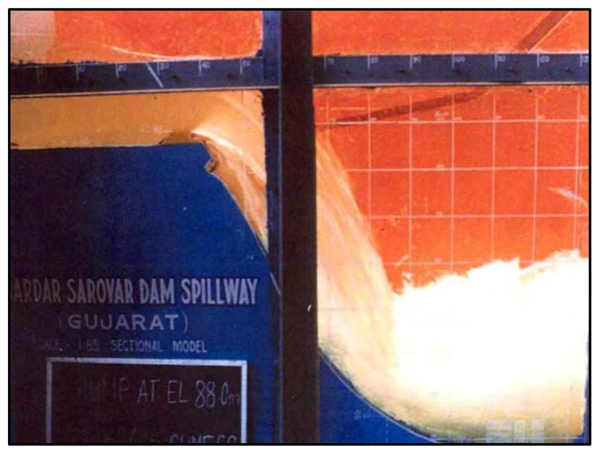

Photo 8: Partly completed Spillway Operation with temporary crest profiles

Flows over a completed profile of a spillway are very different from the flow over a partly completed spillway. The unequal distribution of discharge over the spillway monoliths result in violent flow conditions not envisaged in the design. Hence, the flow conditions over the spillway and downstream are quite different from those expected in the completed stage and can cause serious damage downstream by way of abrasion, erosion, undermining and uplift of panels due to hydrodynamic forces. The flow conditions have the potential to cause large scale damage. The jet of water springs from the downstream edge of the truncated spillway and impinges on the floor of the energy dissipator at the toe of the spillway causing damage. Training the flow over spillway blocks is of paramount importance due to the destructive power of the jet flowing over it. Provision of temporary humps on the spillway monoliths can up to a certain extent help in guiding the flow over the blocks and ensure shear flow. Some arrangement is needed to make the nappe cling to the spillway glacis for as high a discharge as possible. Model studies play an important role in estimating the discharging capacity of the uneven blocks in the direction of flow, the interaction of flow passing over two blocks having different elevations and the head over each block. Hydraulic model studies at CWPRS have demonstrated the usefulness of temporary crests in the form of humps at the downstream edge of the truncated spillway to guide the flow and minimize the damage.

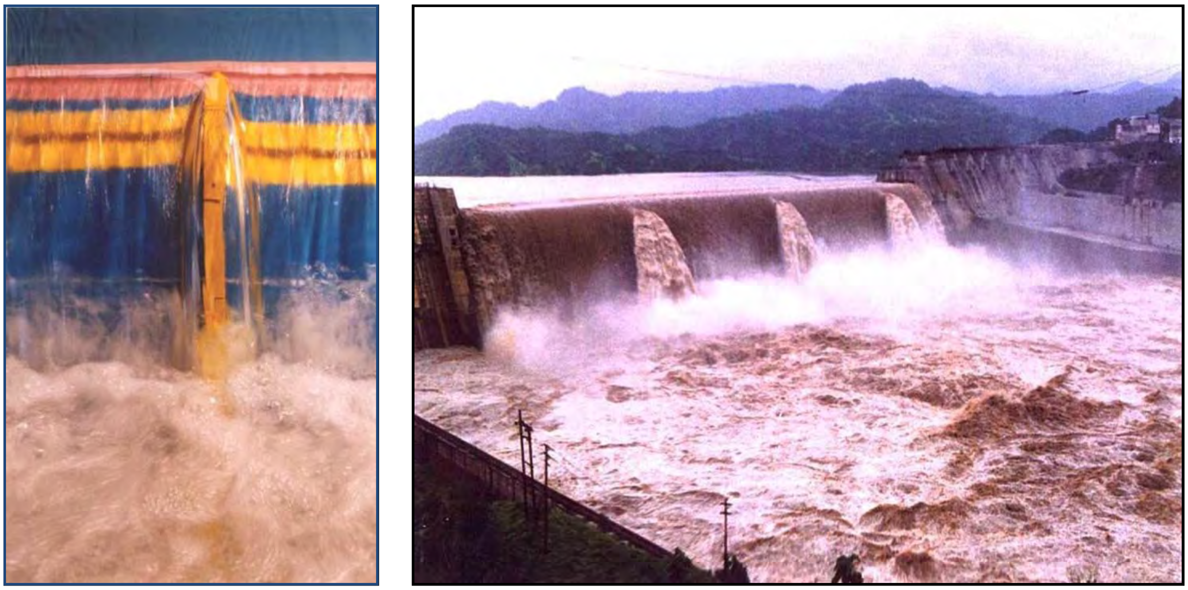

Photo 9: Partly completed Spillway Operation with temporary crest profiles (Model and prototype flow similarity)

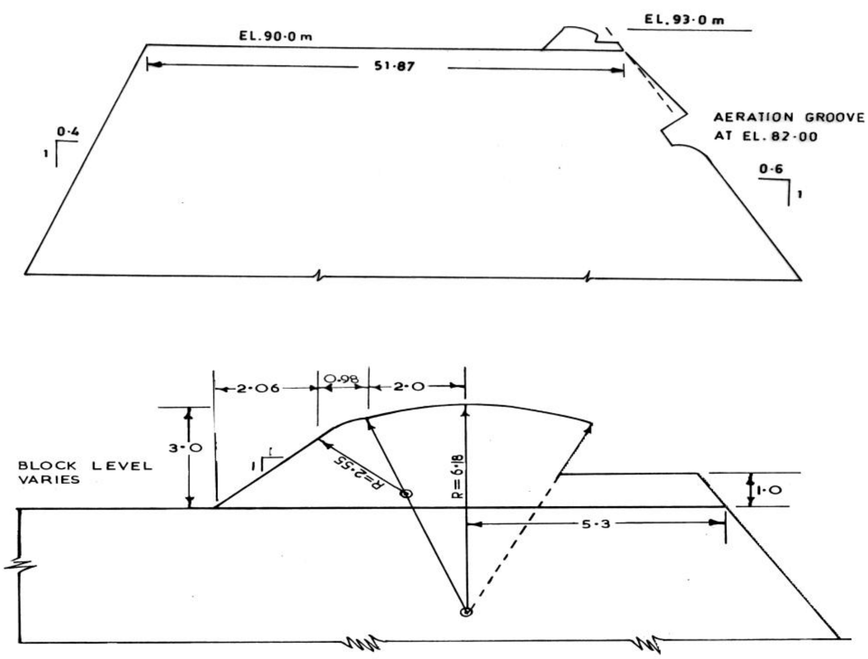

Figure 3: Temporary crest profiles over partly completed spillway blocks

Measurement Techniques

The discharges on the hydraulic models of spillway are measured on the standing wave flume or Rehbock weir using hook gauge of 0.1 mm least count in a stilling well. The accuracy of discharge measurement would be around ±2%. Water levels are measured using pointer gauges fitted with a vernier scale having a least count of 0.1 mm. Reservoir water surface elevations are measured at a location far enough to be free from drawdown and other effects. Tail water levels are measured by a hook gauge having a graduation of 0.1 mm mounted in a stilling well at a distance of about 4 to 5 m downstream of dam axis. Tail water adjustments are made at the downstream end of the model using wooden strips of varying widths or adjustable tailgate. Piezometers (copper tubes) of 3 to 5 mm diameter are provided on the spillway surface along the center of the span for measurement of pressures. Pressures are measured by connecting rubber tubes to the piezometers and to open tube manometers with vertical water columns and could be directly converted to prototype pressure head in meters of water using scaled water manometer board placed by the side of the model.

Sources of Errors and Precautions Needed

Sources of errors in measurements taken during model studies are numerous. Predicting or calculating these errors is most important in establishing the reliability of the studies. A first source of possible error is in the accuracy of model construction. For example, an improperly built spillway crest causes a systematic error in the depths measured for discharges over the crest. An error of plus or minus 1 millimeter in the model construction causes an error in the depth measurement that may be 50 to 100 times larger when converted to a prototype value (upscaling error). A second source of error comes from the instrumentation. Each measurement system-discharge, depth, pressure, velocity, etc. should be evaluated for its probable range of error. Defective zero setting of pointer gauge or hook gauge results in systematic error, which should be checked periodically. The reading error of a pointer gauge is strongly influenced by the distance between the gauge and the observer, the angle which it is read, turbulence of the water and the gradation unit of the gauge.

Accurate and easily understood fabrication drawings for various model components prevent time-consuming errors in construction. Drawings should contain sufficient information to allow the fabrication of a model conforming to the design specifications for the structure. Close tolerances are observed in critical locations such as spillway crests, tangent points and transition curves. Greatest accuracy should be maintained where there will be rapid changes in direction of flow and where high velocities will prevail. In the course of preparing the template assembly, piezometer tubes are soldered or fastened along the profiles of selected templates. If the spillway crest is to be formed with mortar, the pressure taps are closed with lightly fitting plugs before applying the mortar and grout. Upon completion of the model, each tap should be carefully reopened and then finished flush with the surface of the spillway. The finishing of piezometers in models should be done meticulously to prevent measurement errors that would result from improper installation. Piezometers are relatively inexpensive and should be provided in generous numbers to adequately define the critical pressure locations.

Numerical Modelling of Spillway Flows

Traditionally, spillway flows are investigated using physical models. The drawbacks associated with physical model studies of spillways are: cost of construction, delay in time for fabrication and construction of model parts and conducting experiments and the difficulty in changing structural details of various components of the spillway while doing parametric studies. Acquiring necessary insight and understanding the complete hydrodynamics of flow features over spillway by physical models requires sophisticated instrumentation to capture data which is expensive, cumbersome and time consuming. Some scale effects are associated with the physical models for modelling of air water flows. Numerical modelling has been sparsely used in this field due to the complex nature of the flow. The rapid growth in the field of computer technology in terms of computer memory and processing speed has enabled numerical modeling of spillway flows as a viable complementary tool to physical modeling of spillway flows. Simulation of flow over the spillway is possible with advanced CFD software as two phase air-water flow can be modelled.

Numerical Investigations

Spillway flows are essentially rapidly varied flows near crest with pronounced curvature of the streamlines in vertical direction. Two processes simultaneously occurring in the flow down the crest are: formation and gradual thickening of the turbulent boundary layer along the profile and gradual increase in the velocity and decrease in the depth of main flow. Because of the changes of flow boundaries in a short distance, vertical acceleration plays a dominant role in the flow as compared to shear resistance at the solid boundary. The main difficulties while solving the spillway problem numerically are: rapidly varied flow, existence of both subcritical and supercritical flows, development of turbulent boundary layer, unknown free surface and air entrainment. As both subcritical and supercritical flows exist in spillway flow problems, numerical method capable of capturing shock wave should be used. The Navier–Stokes equations can describe virtually any flow problem. However, they are the most difficult to solve. In the last two decades, simulation techniques based on the Navier–Stokes equations have been applied to a large number of flow problems with suitable assumptions and approximations. The rapid development in the computer technology has made the computational fluid dynamics an effective and economical tool for solving various problems in Fluid Mechanics. Computational Fluid Dynamics is a branch of science, which deals with replacing the differential equations governing the fluid flow, into set of algebraic equations. Theses algebraic equations are solved with the help of digital computers. CFD can be a very useful tool to minimize the efforts and expenses of physical modelling as it consumes less time and gives accurate results once the CFD model is validated. It provides good control over all the flow (geometric and dynamic) parameters. It is also cost effective. However, one cannot replace the physical model at this stage, since the validation of the CFD model is done using the results of the physical model.

The basic equations for fluid flow are based on the law of mass, momentum and energy. The equation of conservation of mass or continuity equation and the Navier-Stocks equations form the basis of the continuum model of the fluid flow.

Continuity Equation

The equation for conservation of mass, or continuity equation, can be written as follows:

$$\frac{\partial \rho}{\partial t} + \nabla . (\rho v) = S_m \tag{11}$$

Equation 5 is the general form of the mass conservation equation and is valid for incompressible as well as compressible flows. The source S_m is the mass added to the continuous phase from the dispersed second phase (e.g., due to vaporization of liquid droplets) ρ is the fluid density and v is the fluid velocity.

Momentum Equation

Conservation of momentum in an inertial (non-accelerating) reference frame is described as:

$$\frac{\partial}{\partial t}\left(\rho \vec{v}\right) + \nabla . \left(\rho \vec{v} \vec{v}\right) = -\nabla p + \nabla . \left(\vec{\tau}\right) + \rho \vec{g} + \vec{F} \tag{12}$$

Where p is the static pressure, $\vec{\tau}$ is the stress tensor (described below), and $\rho\vec{g}$ and $\vec{F}$ are the gravitational body force and external body forces (e.g., that arise from interaction with the dispersed phase), respectively. The stress tensor $\vec{\tau}$ is given by

$$\vec{\tau} = \mu\left[\left(\nabla\vec{v} + \nabla\vec{v}^{T}\right) - \frac{2}{3}\nabla.\vec{v}I\right] \tag{13}$$

Where „*μ*' is the molecular viscosity, „*I*' is the unit tensor, and the second term on the right hand side is the effect of volume dilation.

Some of the methods used to calculate the spillway flow found in literatures have adopted some kind of assumptions in order to simplify the computation. Owing to the weak suitability of Finite Difference Method (FDM) to the curvilinear solid geometries, its application to the gravity driven free surface flows with arbitrary curved solid boundaries is severely restricted. Meanwhile, Finite Volume Method (FVM), Finite Element Method (FEM) and Boundary Element Method (BEM) that have excellent suitability to the curved solid boundaries have been widely used. Many popular CFD codes use the finite volume method. While the finite difference and finite element methods start from the differential form of the governing equations, the finite volume method discretizes the Navier-Stokes equations directly in the integral form, ensuring the conservation of mass, momentum, and energy both locally at the discrete cell level and globally over the entire flow domain. Conservation is important for capturing shocks and other flow discontinuities accurately in high-speed compressible flow simulations, and therefore it is the strength of finite volume method.

Free Surface Modeling

In general the motion of the fluid is represented either by Lagrangian or Euler representation. In the Lagrangian calculation, the grid moves with the computed element velocities, while in Eulerian calculation it would be necessary to compute the flow field through the fixed mesh. Free surface flows are more complex than closed conduit flows. The reason is that the free surface is a dependent variable so that various streamline curvatures can create widely variable pressure distributions over the cross section. Rapidly varied flow such as flow over spillway having large streamline curves exerts non-hydrostatic pressure distribution over the section. It is important to track the free surface accurately to solve the flow numerically over the spillway. Tracking involves, locating the surface, defining the surface as a sharp interface between the water and air and applying the boundary condition at the interface. There are different means for tracking the free-surface boundary condition. Volume of Fluid (VOF) is one of them and used in the present study.

Volume of fluid (VOF) Method

The idea for this approach (Hirt and Nichols, 1981) originated as a way to have the powerful volume-tracking feature of the MAC method without its large memory and CPU costs. The VOF formulation relies on the fact that two or more fluids (or phases) are not interpenetrating . For each additional phase that is added to the model, a variable is introduced: the volume fraction of the phase in the computational cell. In each control volume, the volume fractions of all phases sum to unity. The fields for all variables and properties are shared by the phases and represent volume-averaged values, as long as the volume fraction of each of the phases is known at each location. Thus the variables and properties in any given cell are either purely representative of one of the phases, or representative of a mixture of the phases, depending upon the volume fraction values. In other words, if the qth fluid's volume fraction in the cell is denoted as α_q, then the following three conditions are possible:

$\alpha_q = 0$: The cell is empty (of the qth fluid).

$\alpha_q = 1$: The cell is full (of the qth fluid)

$0 < \alpha_q < 1$: The cell contains the interface between the qth fluid and one or more other fluids.

Based on the local value of α_q, the appropriate properties and variables will be assigned to each control volume within the domain. If the amount of fluid in each cell is known, it is possible to locate surfaces, as well as determine surface slopes and curvatures.

Turbulence Modelling

Turbulent flows are characterized by fluctuating velocity fields. These fluctuations mix transported quantities such as momentum, energy, and species concentration, and cause the transported quantities to fluctuate as well. Since these fluctuations can be of small scale and high frequency, they are computationally too expensive to simulate directly in practical engineering calculations. Instead, the instantaneous (exact) governing equations can be time-averaged, ensemble-averaged, or otherwise manipulated to remove the small scales, resulting in a modified set of equations that are computationally less expensive to solve. However, the modified equations contain additional unknown variables, and turbulence models are needed to determine these variables in terms of known quantities. There are three major approaches to predict turbulent flows, viz. Statistical Turbulence Modelling (STM), Large Eddy Simulation (LES) and Direct Numerical Simulation (DNS) (Tannehill *et* *al.*, 1997). Statistical turbulence models based on the Reynolds- Averaged Navier-Stokes (RANS) equations represent transport equations for the mean flow quantities only, with all the scales of the turbulence being modelled.

There are many turbulence models available such as:

- Spalart-Allmaras model
- k-ε models
- Standard k-ε model
- Renormalization-group (RNG) k-ε model
- Realizable k-ε model
- k-ω models
- Standard k-ω model
- Shear-stress transport (SST) k-ω model
- v^2-f model
- Reynolds stress model (RSM)

No single turbulence model is universally accepted as being superior for all classes of problems. The choice of turbulence model will depend on considerations such as the physics encompassed in the flow, the established practice for a specific class of problem, the level of accuracy required, the available computational resources, and the amount of time available for the simulation.

Boundary and Initial Conditions

Boundary conditions are the most important and critical aspects of the numerical modelling. Utmost care has to be taken in the formulation of boundary conditions so that the physical phenomenon could be represented satisfactorily. It is important that the boundary conditions accurately represent what is physically occurring for a given flow condition. Boundary conditions specify the flow variables or their gradients on the boundaries of computational flow domain.

Upstream Boundary Condition

The upstream boundary can be set up at a flow inlet at which the reservoir water level but the incoming discharge and/or velocity are unknown. This section should be far away from the spillway to avoid the reflection effect.

Downstream Boundary Condition

The downstream boundary should be located based on the range of the interested domain. For the study of the spillway crest and the aerator region, the downstream condition will have no effect on the upstream flow since the flow over the downstream slope of the spillway is supercritical. However, the downstream section has to be chosen far downstream of the end of the spillway so that the ski jump jet/hydraulic jump is fully formed.

Solid Boundary Condition

The interface between the fluid and solid boundary is considered as closed boundary. It is also called as wall boundary. There are three kinds of solid boundary conditions:

- Full slip boundary condition
- Partial slip boundary condition and
- No slip boundary condition

The full slip means that tangential velocity at the inner grid is equal to tangential velocity on the solid surface; while no slip means tangential velocity on the solid surface is zero; for partial slip condition, a wall function should be used. There is no flow across solid boundaries. Selection of the three alternatives depends on the nature of governing equations, relative magnitude of the grid size and the boundary layer thickness in the flow domain.

Initial Condition

Before starting the solution, an initial guess has to be provided for the solution flow field. An accurately assumed velocity and free surface profile will accelerate the convergence of the computations.

Operating Conditions

Operating pressure is defined at the atmospheric pressure. The operating density is specified as 1.223 m^3/s, as air was the primary phase out of the two phases viz. air and water.

Convergence and Stability

Numerical computations often have spurious oscillations in space and time or both. If the computed values within a time step exhibit variations over a distance comparable to cell width or time comparable to the time increment, the accuracy of the computed results is not reliable. It is said that the solution has not converged. To prevent this, certain restrictions in terms of the mesh sizes $(\Delta x, \Delta y)$ or aspect ratio of the cells $\frac{\Delta x}{\Delta y}$, or the time increment Δt and weighted coefficients is required to be made (Courant and Friedrichs, 1948). For accuracy, mesh increments are to be chosen small enough to resolve the spatial variation in all dependent variables. Depending upon the nature of problem, it may be necessary to select the finer grids near the wall boundaries.

Once the mesh is chosen, the choice of the maximum time increment allowable for stability is governed by several restrictions. First the fluid cannot move more than one cell in one time increment. This gives the condition,

$$\Delta t < Min\left(\frac{\Delta x}{|u|}, \frac{\Delta y}{|v|}\right) \tag{14}$$

Δx and Δy are the lengths of any given cell in *x* and *y* directions. *u* and *v* are the velocity components in *x* and *y* directions. The minimum is with respect to every cell in flow domain. The stability of the computations is controlled by the Courant-Friedrichs-Levy condition (Courant et al, 1967):

$$\Delta t \leq \left\{\frac{\forall}{|u_i s_i| + a_0 |s_i|}\right\} \tag{15}$$

Where $\forall$ is the control volume and s_i is the surface area of the control volume.
The maximum allowable time step can be estimated by using the following equation:

$$\Delta t = C_r \left\{\frac{\forall}{(|u_i s_i| + a_0 |s_i|)}\right\} \tag{16}$$

Where, $0 < C_r < 1$

11.12 Air Water Flow Modelling

Hydraulic structures can be operated safely and efficiently if due attention is paid to both water flow as well as the simultaneous movement of air in the system. The difference in the density of air and water is very large viz. water density 1000 kg/m^3 and air density 1.223 kg/m^3. Therefore, they are usually well separated by a sharp interface. However, a number of flow configurations lead to an intense mixing across this surface. This process is called air entrainment. A self-aerating flow configuration continuously produces air bubbles by mechanical action, which is subsequently carried away by the flow if the transport capacity of the water flow is sufficiently high. All these bubbles which are entrained but cannot be transported by the flow will escape through the water surface (detrainment). Thus air entrainment, transport capacity and detrainment are interrelated. The region of detrainment may or may not be near the region of entrainment and it depends on the water flow conditions and their transport capacity. Figure 4 shows the schematic representation of air entrainment, detrainment and transport processes in an open channel flow.

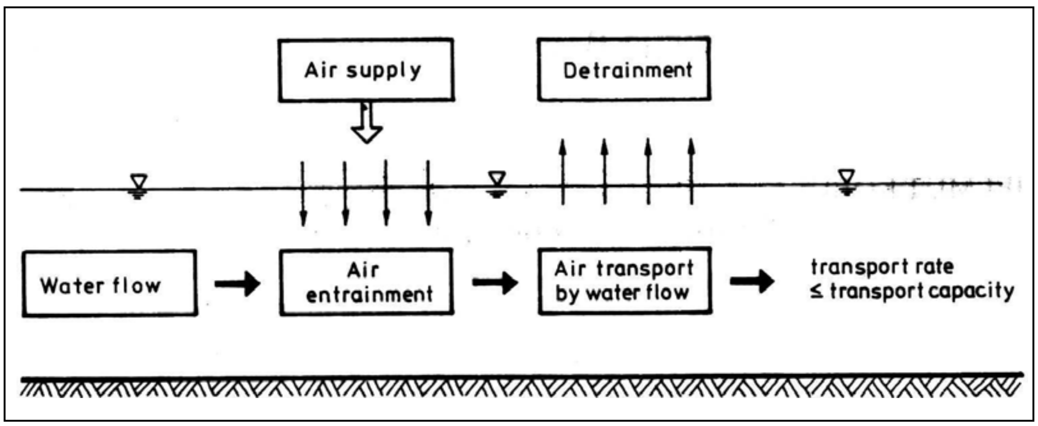

Figure 4: Air Entrainment, Detrainment and Transport Processes in Open Channel, (Kobus H., 1984)

Air Water Flow in Spillways

Normally a region of clear water is observed where the water enters the chute or spillway. Then, at some distance downstream, the water suddenly takes on a milky appearance. The “White Water” begins where the turbulent boundary layer from the floor intersects the water surface. The validity of this assumption has been verified by many researchers. The flow down a long spillway can be divided into a number of distinct regions:

1. A regime of no air concentration where the turbulent boundary layer has not reached the water surface.

2. A regime of developing air entrainment in which the air concentration profiles are not constant with distance.

3. A regime of fully developed air entrainment in which the air concentration profiles are constant with distance.

In the non-aerated region close to the spillway crest the boundary layer grows from the spillway floor. Outside the boundary layer the flow in predominantly irrational and the water surface in smooth and glassy. At the point where the boundary layer reaches the free surface, the surface becomes disturbed and entrainment by multitude of vortices in the turbulent flow commences. This point is called the point of inception. Downstream of the start of air entrainment a layer containing a mixture of both air and water gradually extends through the flowing fluid.

Closure

A lack of understanding of physical processes or complex boundary conditions in many fluid mechanics/ hydraulics problems which are not amenable to numerical or analytical techniques are investigated by physical models. Various factors influencing the selection of scales for Froudian models, construction methodology, measurement techniques adopted and planning of model studies are briefly described in this chapter. Physical model studies have been indispensable tools to optimize various components of spillway structure as they are able to replicate many features of complex flow features of spillways. Various hydraulic design aspects such as discharging capacity, pressures and water surface profiles and energy dissipation arrangement are considered to evolve hydraulically efficient design of spillway.

References

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2. Courant, R. and Friedrichs, K.O. and Lewy, H. (1967). “On the partial differential equations of mathematical physics.” IBM Journal of Research and Development, Vol. 11, pp: 215-234.
3. Hirt, C.W. and Nichols, B.D. (1981). “Volume of Fluid (VOF) method for the dynamics of free boundaries.” Journal of Computational Physics, 39, pp:201-225.

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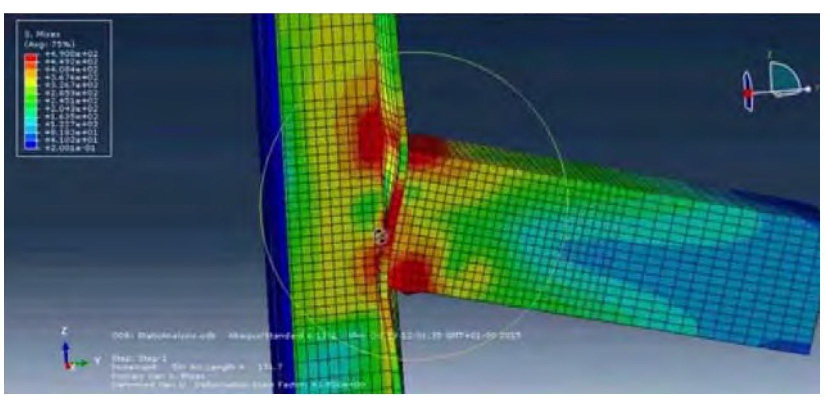
6. Kobus H. (1984). “Local air entrainment and detrainment” Symposium on scale effects in modeling hydraulic structures, Esslingen, Germany, paper 4.10 pp 1-10
7. Tannehill, J. C., Anderson, D. A. and Pletcher, R. H. (1997). “Computational fluid mechanics and heat transfer.” 2nd edition, Hemisphere Publishers
8. WES Hydraulic Design Criteria (1977) Low monolith diversions , Discharge Coefficients -Sheet 711

ABAQUS

-Manish Rathore
Deputy Director

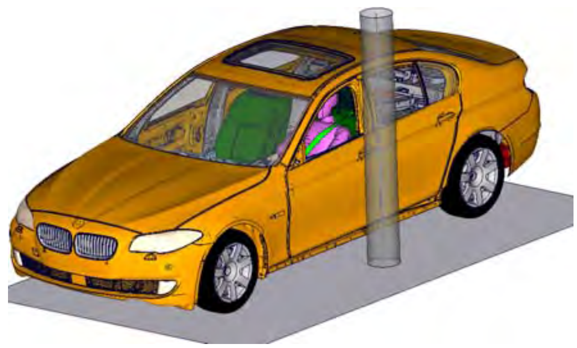
Capabilities

- Static (linear and non-linear)



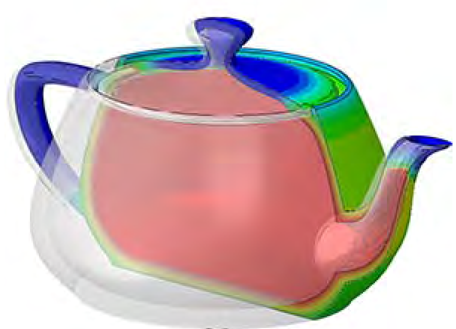
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)



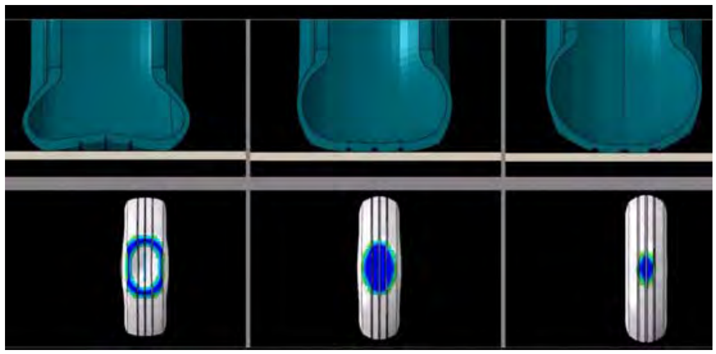
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer



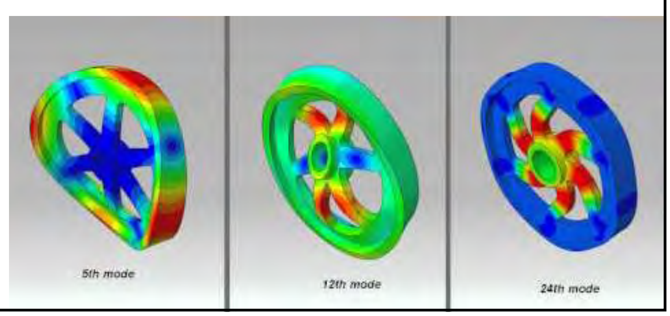
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer
- Contact Analysis



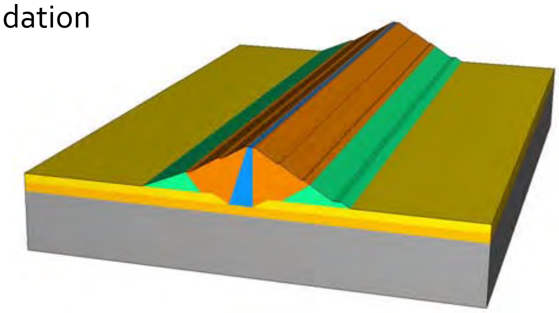
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer
- Contact Analysis
- Vibration, Frequency, Buckling



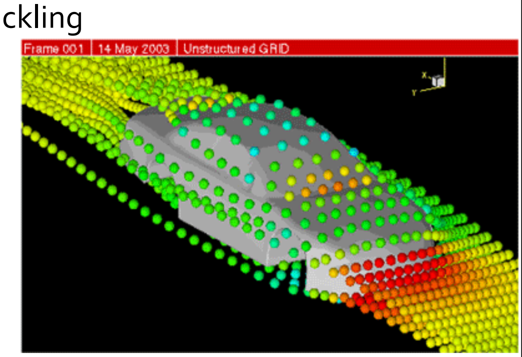
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer
- Contact Analysis
- Vibration, Frequency, Buckling
- Soil Seepage / Consolidation



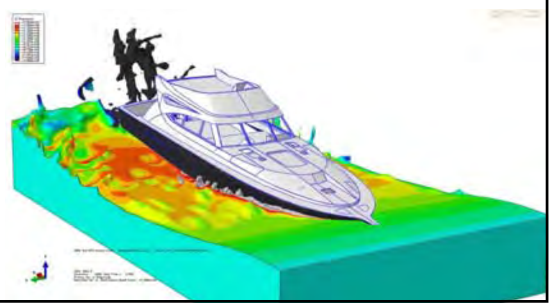
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer
- Contact Analysis
- Vibration, Frequency, Buckling
- Soil Consolidation
- CFD Analysis



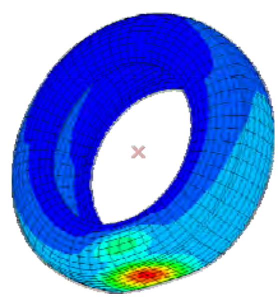
Capabilities

- Static (linear and non-linear)
- Dynamics (Implicit, Explicit)
- Heat Transfer
- Contact Analysis
- Vibration, Frequency, Buckling
- Soil Consolidation
- CFD Analysis
- Multi-physics Analysis

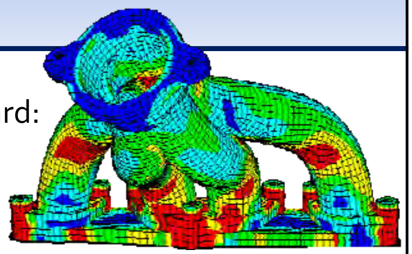


Capabilities

- Linear dynamics:
  - Natural frequency extraction
  - Modal superposition
  - Harmonic loading
  - Response spectrum analysis
  - Random loading
- Nonlinear dynamics:
  - Transient dynamics with implicit integration

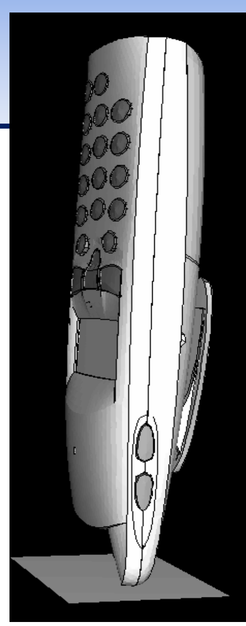


Capabilities



- Multiphysics with Abaqus/Standard:
  - Thermal-mechanical analysis
  - Structural-acoustic analysis
  - Linear piezoelectric analysis
  - Thermal-electrical (Joule heating) analysis
  - Thermal-electrical-structural analysis
  - Fully or partially saturated pore fluid flow-deformation
  - Fluid-structure interaction

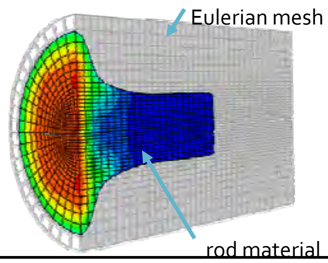
Capabilities



- Abaqus/Explicit
  - Simulates high speed dynamic events such as drop tests.
    - Explicit algorithm for updating the mechanical response.
  - Also a powerful tool for quasi-static metal forming simulations.
    - Annealing is available for multistep forming simulations

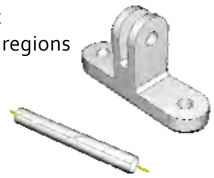
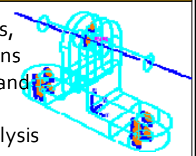
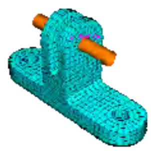
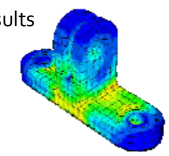
Capabilities

- Special features of Abaqus/Explicit: Coupled Eulerian-Lagrangian (CEL)
  - Define a domain in which material can flow for an Eulerian analysis
    - Flow problems
    - Structural problems with extreme deformation



Database Structure

- Database structure
  - You can have multiple database on one computer
  - One database can have multiple models
  - You can work with only one database at a time
  - If you want to work with multiple models, keep all of them in one database
  - One model can have multiple parts
  - One model can have only one assembly (created by calling parts)
- Model
  - Contains parts, materials, sections, assembly, surfaces, steps, loads, BCs, fields, interactions, meshes.

Part	Property	Assembly
Create the part geometry (and regions for sections, if necessary) 	Define materials Define additional part regions Define and assign sections to parts or regions	Position parts for initial configuration. 
Step	**Interaction**	**Load**
Define analysis steps and output requests	Define contact and other interactions on regions or named sets, and assign them to steps in the analysis history	Apply loads, BCs, and ICs to regions or named sets; and assign them to steps in the analysis history 
Mesh	**Job**	**Visualization**
Split assembly into meshable regions and mesh 	Submit, manage, and monitor analysis jobs	Examine results 

Files Created During Analysis

- Commands issued during an Abaqus/CAE session are saved in journaling files containing Python scripts.

Replay (`.rpy`) file	All commands executed during a session, including any mistakes, are saved in this file.
Journal (`.jnl`) file	All commands necessary to recreate the most currently saved model database (`.cae`) are saved in this file.
Recover (`.rec`) file	All commands necessary to recreate the model database (`.cae`) since it was most recently saved are saved in this file.

  - Journaling files can be modified in any way appropriate for the Python language.

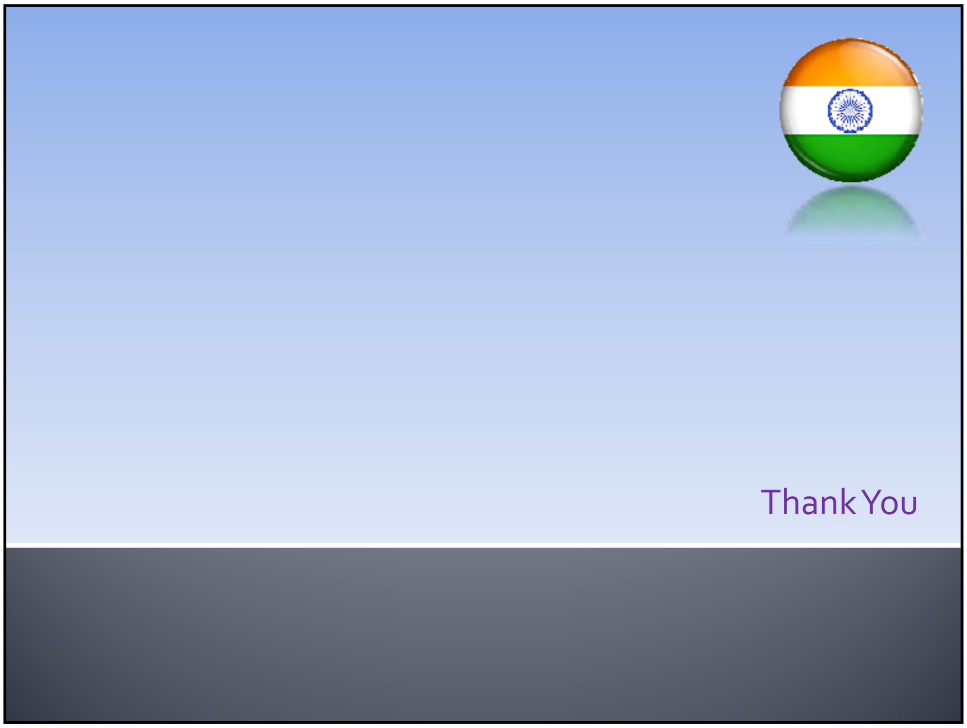
Important Points to Remember

- First thing to do after opening ABAQUS is to set work directory
- Models are feature based and parametric
- Use shortcut keys for view manipulation
- Always look for the dialogue box at the bottom for pending commands
- All inputs and outputs in ABAQUS are entered without units… maintain your units throughout the project

Important Points to Remember

- Since, all inputs and outputs in ABAQUS are entered without units… maintain one of the following system of units throughout the project

Quantity	SI	SI (mm)	US Unit (ft)	US Unit (inch)
Length	m	mm	ft	in
Force	N	N	lbf	lbf
Mass	kg	tonne (10^3 kg)	slug	lbf s^2/in
Time	s	s	s	s
Stress	Pa (N/m^2)	MPa (N/mm^2)	lbf/ft^2	psi (lbf/in^2)
Energy	J	mJ (10^{-3} J)	ft lbf	in lbf
Density	kg/m^3	tonne/mm^3	slug/ft^3	lbf s^2/in^4


Thank You

Introduction to Phase^2

Manish Rathore
Deputy Director
Narmada Control Authority, Indore

Introduction

Phase^2 is a powerful 2D elasto-plastic finite element stress analysis program for underground or surface excavations in rock or soil. It can be used for a wide range of engineering projects and includes support design, finite element slope stability and groundwater seepage analysis. Complex, multi-stage models can be easily created and quickly analyzed, for example: tunnels in weak or jointed rock, underground powerhouse caverns, open pit mines and slopes, embankments, MSE stabilized earth structures, and much more. Progressive failure, support interaction and a variety of other problems can be addressed.

Phase^2 offers a wide range of support modeling options. Liner elements can be applied in the modeling of shotcrete, concrete, steel set systems, retaining walls, piles, multi-layer composite liners, geotextiles and more. New liner design tools include support capacity plots which allow you to determine the safety factor of reinforced liners. Bolt types include end anchored, fully bonded, cable bolts, split sets and grouted tiebacks. One of the major features of Phase^2 is finite element slope stability analysis using the shear strength reduction method.

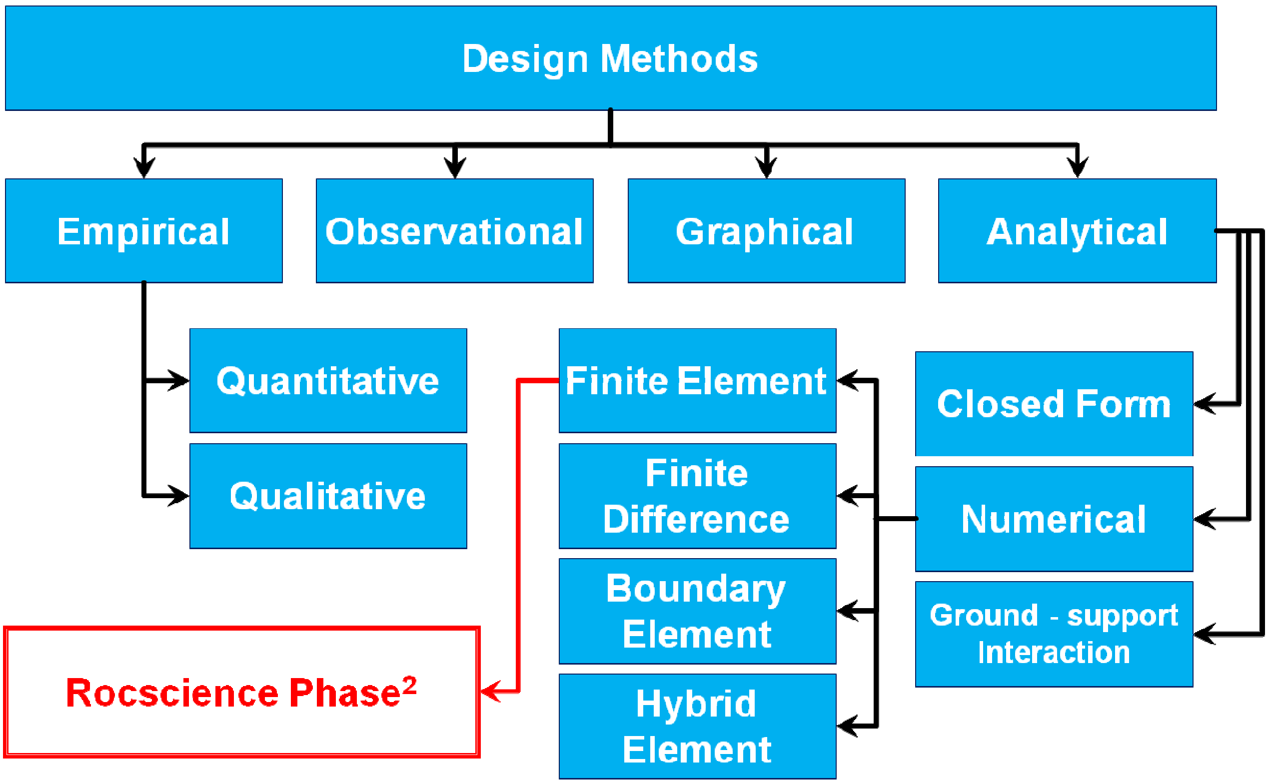
This option is fully automated and can be used with either Mohr-Coulomb or Hoek-Brown strength parameters. Slope models can be imported / exported between Slide and Phase^2 allowing easy comparison of limit equilibrium and finite element results. Phase^2 includes steady state, finite element groundwater seepage analysis built right into the program. There is no need to use a separate groundwater program. Pore pressure is determined as well as flow and gradient, based on user defined hydraulic boundary conditions and material conductivity. Pore pressure results are automatically incorporated into the stress analysis.

Material models for rock and soil include Mohr-Coulomb, Generalized Hoek-Brown and Cam-Clay. Powerful new analysis features for modeling jointed rock allow you to automatically generate discrete joint or fracture networks according to a variety of statistical models.

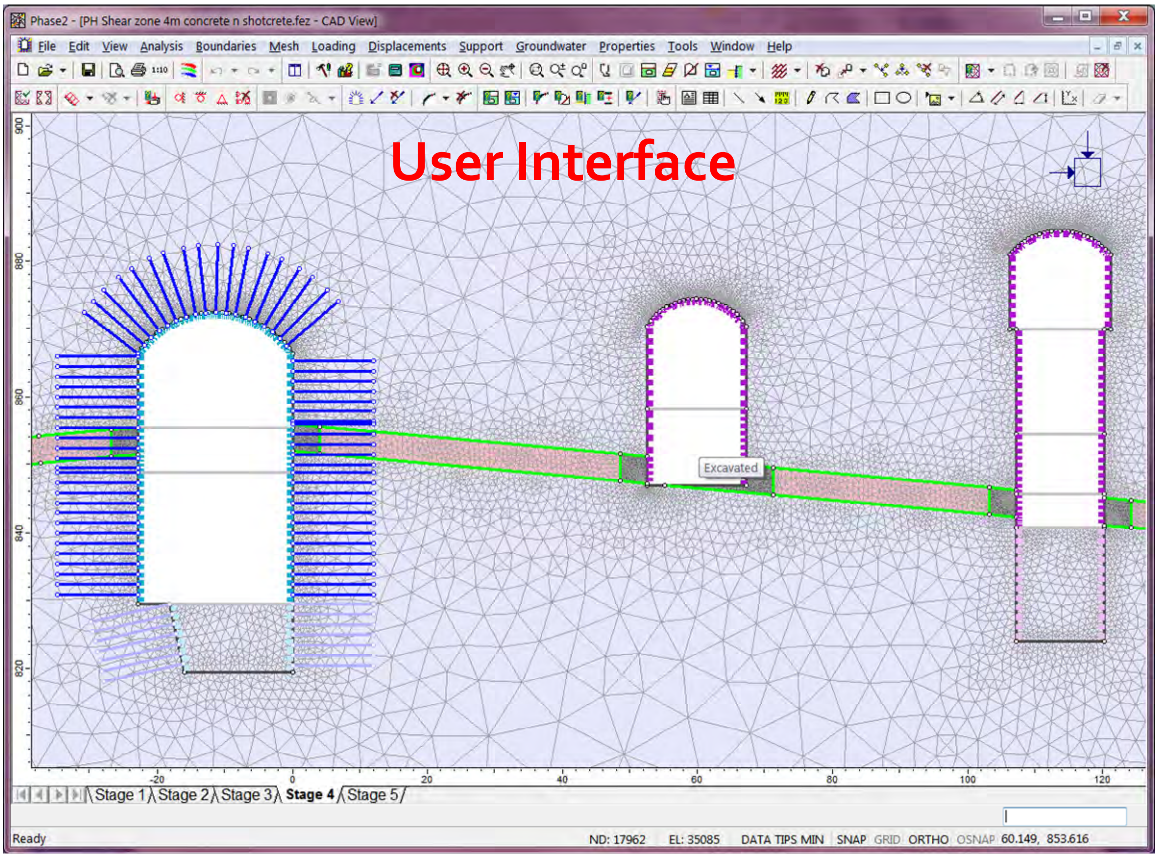
Phase^2 is a 2-dimensional elasto-plastic finite element program for calculating stresses and displacements around underground openings, and can be used to solve a wide range of mining, geotechnical and civil engineering problems, involving:

- Excavations in rock or soil
- Multi-stage excavations (up to 300 stages)
- Elastic or plastic materials
- Multiple materials
- Bolt support
- Liner support (shotcrete / concrete / piles / geosynthetics)
- Constant or gravity field stress
- Jointed rock / construction joints
- Plane strain or axisymmetry
- Groundwater (piezo lines, ru values or finite element seepage analysis)
- Finite element slope stability
- And much more…

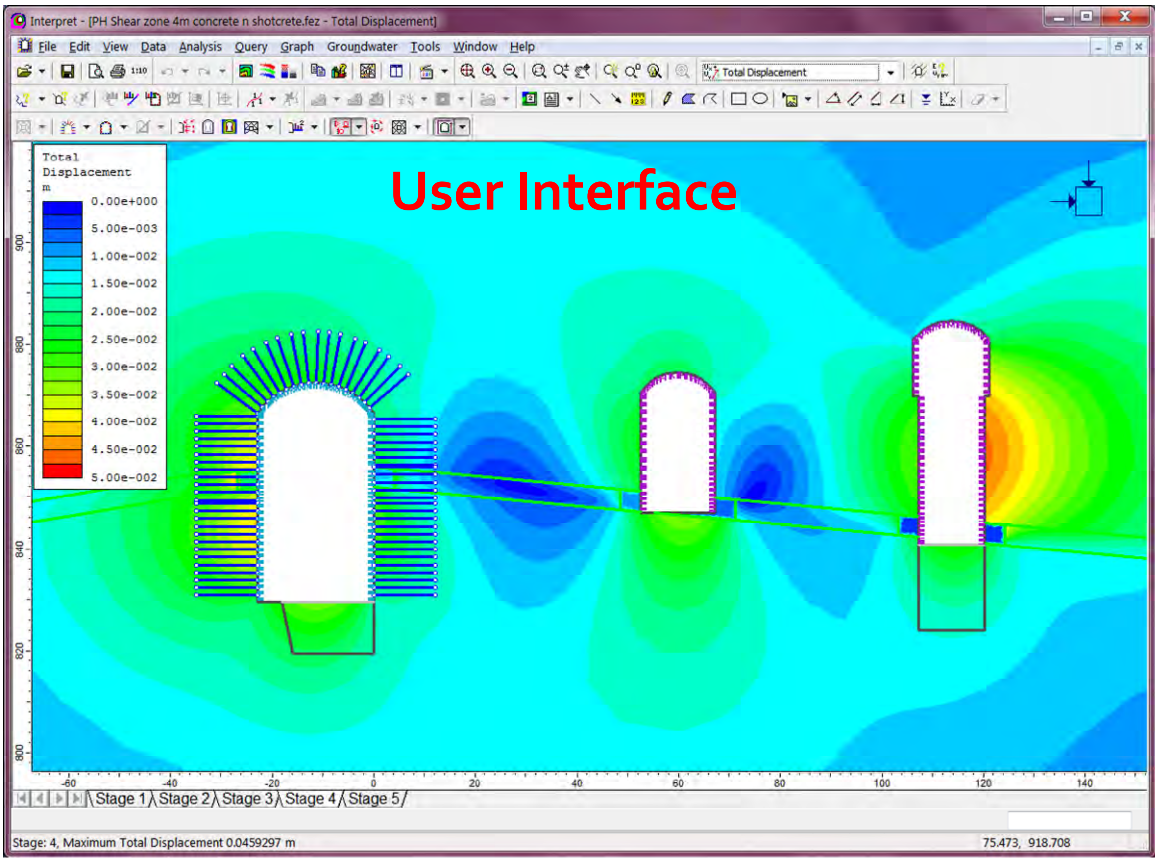
Phase^2 uses Finite Element theory to perform analysis. Following chart represents different methods using which underground excavations/structures can be analyzed.



User Interface of Modeler



User Interface for Interprete



Important Inputs required in different types of analysis

Analysis of Underground Excavation

- Project settings
  - Plain strain / axisymmetric
  - Stages
- Boundaries
  - Openings
  - External
  - Materials
  - Stages
- Mesh
  - Graded
  - Mesh modification
- Boundary conditions
- Loading
  - Field stress
  - External load
- Properties
  - Materials
  - Support system
- Excavate and assign material
- Assign support system
- Analyze
- Interprete

Phreatic Line Estimation

- Project settings
  - Groundwater-FEM
- Boundaries
  - External
  - Materials
- Mesh
  - Uniform
  - Mesh modification
- Boundary conditions
- Hydraulic BC
- Loading
  - Ponded water load
  - External load, if any
- Properties
  - Hydraulic properties
  - Material properties
- Analyze
- Interprete

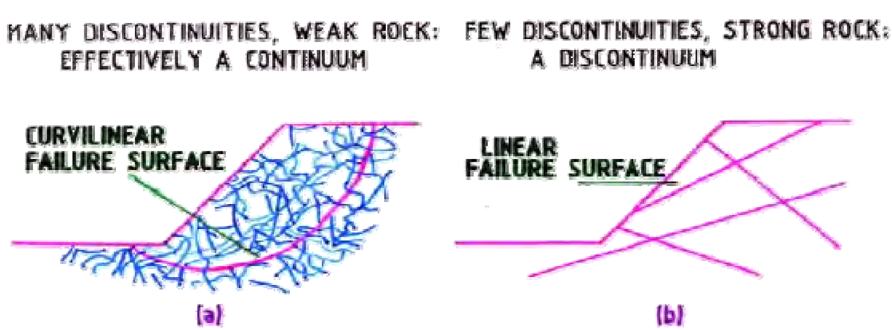
Slope Stability Analysis (SSR)

- Project settings
  - Strength Reduction
- Boundaries
  - External
  - Materials
- Mesh
  - Uniform/Graded
  - Mesh modification
- Boundary conditions
- Loading
  - Gravity
  - External load, if any
- Properties
  - Material properties
  - Material must not be purely elastic
- Define SSR search area, if needed
  - Analysis->SSR
- Analyze
- Interprete

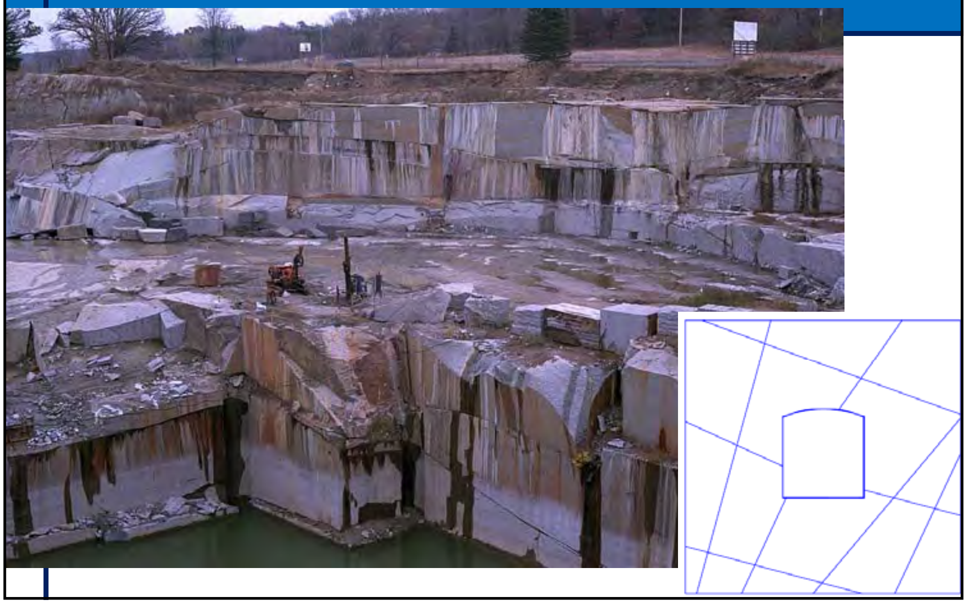
Analysis and Design of Underground Openings & Surface Excavations using Unwedge

-Manish Rathore
Deputy Director

Continuum vs Discontinuum

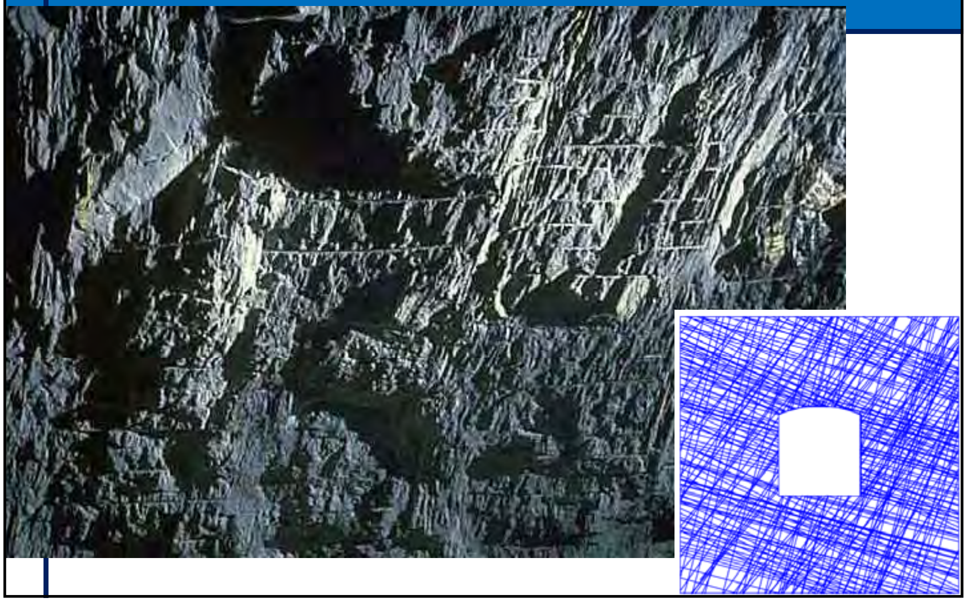


Massive Rock

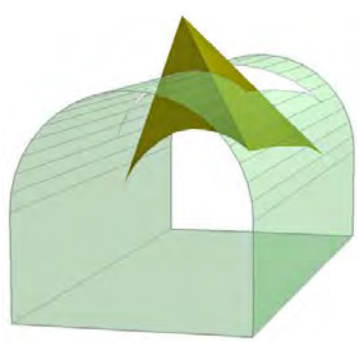


Jointed Rock

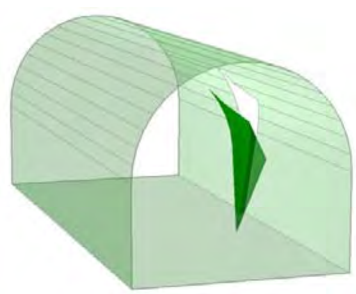
Heavily Jointed Rock



Wedge formation in U/G Openings



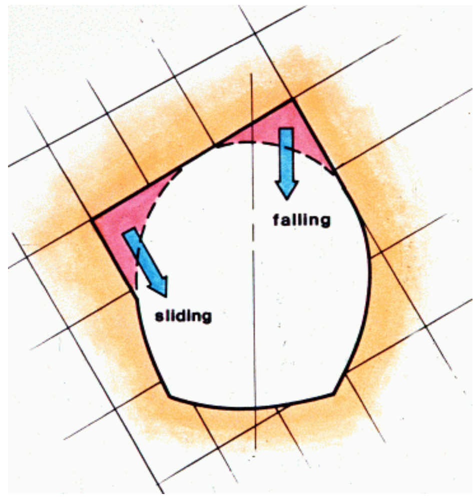
Roof Wedge



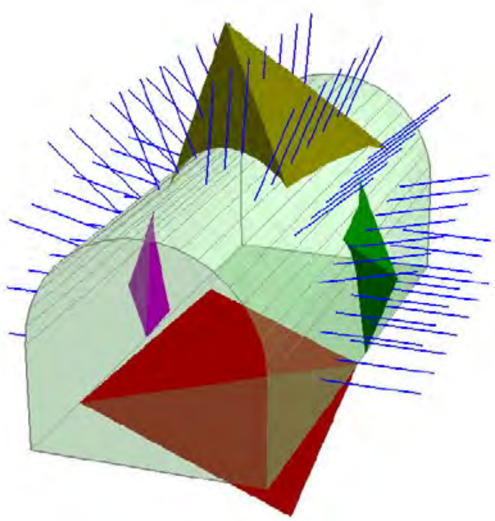
Wall Wedge

Rock Tunnels – Failure Mechanisms

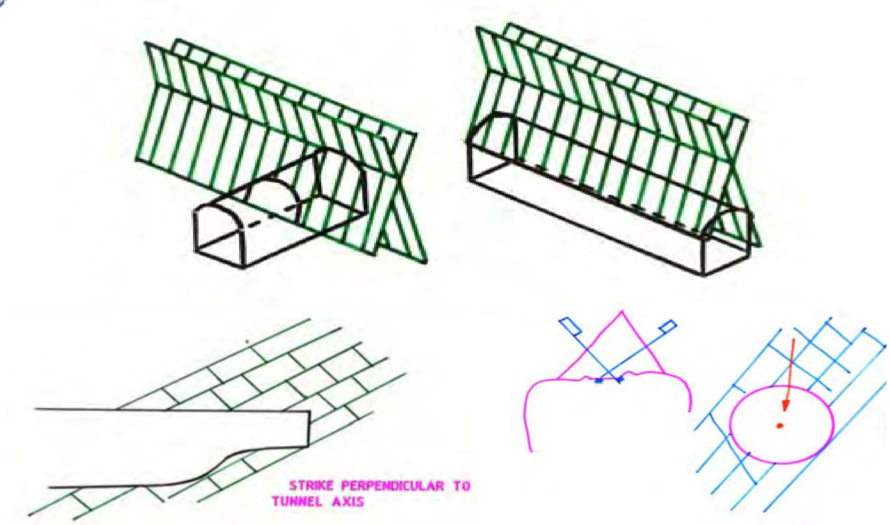
- Falling or sliding of wedges or blocks released by intersecting discontinuities.
- This type of failure is structurally controlled failure, generally occur in hard rock



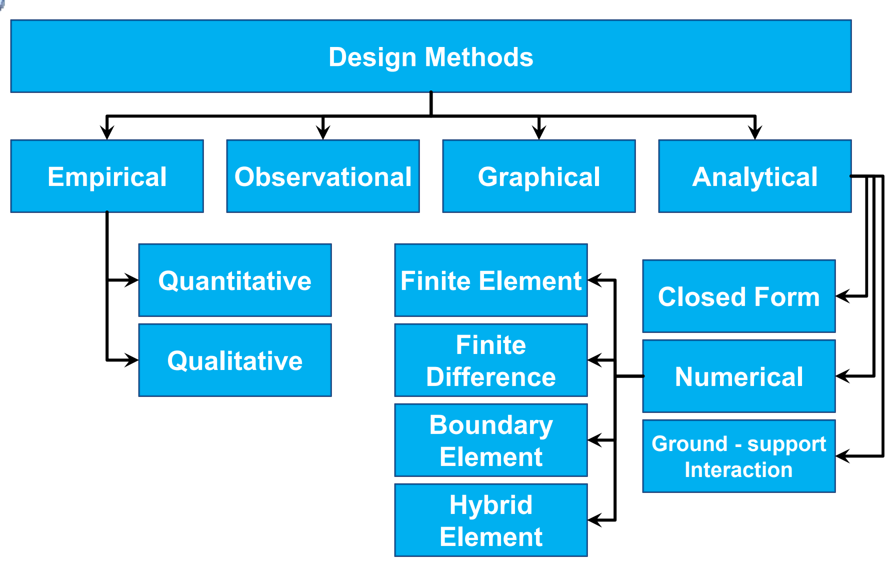
Rock Tunnels – Failure Mechanisms



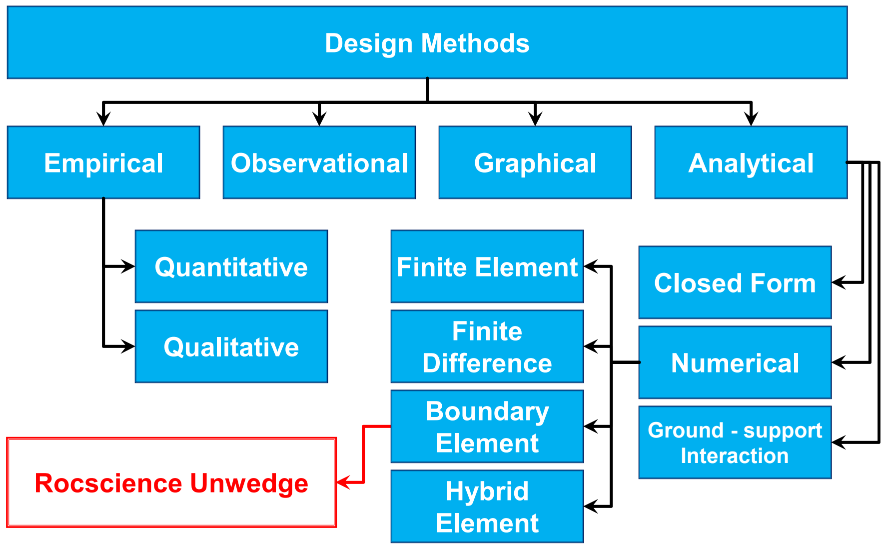
Effect of Orientation



Summary of Design Methods



Summary of Design Methods



What is Unwedge..?

- Unwedge is designed to be a quick, interactive and simple to use tool for the analysis of the geometry and the stability of underground wedges
- These wedges are defined by intersecting structural discontinuities in the rock mass surrounding an underground excavation.

Program Assumptions

- Unwedge should be used to analyze wedge failure around excavations constructed in hard rock, where discontinuities are persistent.
- It is assumed that displacements take place at the discontinuities, and that the wedges move as rigid bodies with no internal deformation or cracking.
- The wedges are tetrahedral in nature, and defined by three intersecting discontinuities. A maximum of three structural planes can be analyzed at one time. If more than three major planes are identified for the analysis of the structural data, then all combinations of these planes should be considered.
- All of the discontinuity surfaces are assumed to be perfectly planar.

Program Assumptions

- Discontinuity surfaces are assumed to be persistent, therefore the discontinuities defining the wedge do not terminate within the region where the wedges are formed. The implication is that no new cracking is required in the analysis of wedge movement.
- The underground excavation is assumed to have a constant cross section along its axis.
- The default analysis is based upon the assumption that the wedges are subjected to gravitational loading only. Effect of field stress is not taken by default.
- Unwedge always initially calculates the *maximum* sized wedges which can form around the excavation.

Analysis Steps

- Project Settings
- Define Opening Section (Add Opening Section / Import DXF)
- Input Data > Tunnel Axis Orientation , Unit Weight
- Input Data > Joint Orientations
- Input Data > Joint Properties
- Assign Joint Properties
- 3D wedge view
- Wedge Information
- Support Design (Bolts / Shotcrete / Pressure)
- Iterate Support Design
- Advanced Features:
  - Combination
  - Field Stress
  - Tunnel Axis Plot

Thank You

Introduction to Water Hammer And Mass Oscillation (WHAMO)

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Introduction

Fluid distribution systems and hydropower plants can be severely damaged by water hammer. Water hammer defined as the forceful slam that occurs in pipes when a sudden change in fluid velocity creates a significant change in fluid pressure. Water hammer can destroy hydraulic machines and cause pipes / penstocks to rupture. Water hammer can be avoided by designing and operating these systems such that unfavorable changes in water velocity are minimized. The WHAMO computer program has been developed to assist engineers in mitigating water hammer by simulating Water Hammer And Mass Oscillation in fluid networks that convey fluids such as water. Some typical applications for WHAMO include analysis of hydropower plants, pumping facilities, jet fueling systems and wastewater collection systems. The program determines time varying flow and head (transients) in a network which may include pipes, valves, pumps, turbines, pump-turbines, surge tanks, and junctions arranged in any reasonable configuration. Such transients are generated due to any variation in the operation of a hydraulic machine or valve within the network, or due to changes in the head or discharge at boundaries of the network. WHAMGR is an associated graphics program used for creating time history plots from WHAMO simulations.

Objective

The objective of the WHAMO program is to perform dynamic simulation of fluid distribution networks comprising of components such as pipes, valves, pumps, turbines, pump-turbines, surge tanks, and junctions. The program calculates time varying flows, pressures, and heads throughout the network.

The WHAMO Modelling System

It is very important to understand some basics about how WHAMO is structured and how it works. The complete WHAMO modeling system includes two separate programs interconnected via data files. These are the main WHAMO simulation program (whamo.exe) and the WHAMGR graphics package (whamgr.exe). The main WHAMO simulation program may receive input from an ASCII text file that has been created with any word processor (like notepad, textpad, etc.). The WHAMGR graphics package reads the output file produced by the WHAMO simulation program and allows the user to print the output and/or display it to the computer screen. A variety of display options are available. These will be described later in the manual.

The Main WHAMO Simulation Program

The main WHAMO simulation program uses an implicit finite difference method for calculating time-varying flows and pressures throughout the network that is being modeled. Numerical techniques used to approximate partial differential equations such as the water hammer equations can be generally classified as implicit or explicit. Implicit methods generally require simultaneous solution of a set of equations while explicit formulations can be solved directly. Explicit methods are constrained according to system geometry to work at computational time steps which are typically very small, while the implicit methods are not so constrained. Implicit methods require greater computational effort per time step because of the necessity of solving simultaneous equations. The amount of effort required for a simulation can be reduced by varying the length of the time step during the simulation (which is allowed in WHAMO). Any initial, high frequency water hammer response in a system should be modeled with short time steps, but in simulations where the water hammer dissipates and mass oscillation becomes predominant, the time step can be greatly increased during the simulation with no significant loss of accuracy.

The following basic elements form the program modules:

1. A set of building block subroutines which represent the one-dimensional continuity and momentum equations for compressible liquid flow in closed conduits.
2. A set of building block routines which represent the head discharge relations across typical elements such as hydraulic machines, valves, and junctions.
3. A set of input routines which accept a description of the preceding elements and their interconnection.
4. A set of organizational routines which restructure the input data into a matrix form representative of the system to be modeled, and a routine for solving the resulting equations.

5. A set of computational routines which automatically generate an initial steady-state condition for the system, consistent with input boundary conditions.
6. A set of output routines which allows the program user to specify the type, amount, and form of computational results to be produced. Output may be printed in either a tabular or graphical form, exported to a spreadsheet, or saved as an ASCII text file. Where simulations are long, time steps short, or the system complex, the program produces a huge amount of output, therefore the user may select the locations and parameters for which an output history is desired.

WHAMO follows certain basic steps during a simulation run. These are:

1. Read and check input data — The user's commands and associated data are read in. The data are processed as necessary and checked for errors.
2. Build system connectivity — The user's specifications of the system structure are reinterpreted to a form compatible with computation. The resulting system is checked to insure that it is physically reasonable.
3. Display input data and system structure — All data input as well as the system structure and other information determined by the program are printed in tabular form. This allows easy verification of the input data by the user.
4. Steady state generation — An initial, steady state condition compatible with the specified boundary and operating conditions is determined for the system.
5. Transient simulation — System response to specified transient machine operation or boundary condition is simulated. Computational difficulties are monitored by the program's self-checking algorithms.
6. Output — The simulation results are printed or stored for later processing according to the user's instructions.

The WHAMO program considers a hydraulic system as constructed of a number of individual components of various types, interconnected in a particular network configuration. This information is supplied to the program by the user through a input data file. The model further requires specification of system boundary conditions and operating conditions for the simulation period. These data are interpreted and processed by the program and computations are performed to simulate the transient state hydraulics of the system.

WHAMO system elements

The basic physical elements considered by WHAMO fall into four main categories:

- Flow elements
- Turbomachines
- Boundary elements
- Junctions.

Altogether, nearly 20 specific element types are available to the user for construction of a system. For each element type, the model employs special forms of mathematical equations to represent the hydraulics of a given component. These equations are utilized in pairs, one based on the continuity principle and one based on the momentum (or energy) conservation principle as applied to a particular element. When the user specifies an element of a particular type, he must designate a name by which it can be referenced and supply data on the properties which are relevant to the hydraulics or mechanics of that element. For example, the length, diameter, roughness, and celerity of a conduit must be specified. Simply stated, enough data must be supplied for an element that continuity and momentum equations can be written under all conditions to be simulated.

Main elements used typically for transient analysis are as follows:

1. Flow elements: Conduits, variable diameter conduits, dummy elements, control valves, etc.
2. Boundary elements: Reservoirs, surge tanks, flow boundaries, etc.
3. Junctions

Why use WHAMO?

WHAMO is used to run transient analysis in water conductor system (WCS). By running transient analysis, we get very accurate results in terms of velocity, discharge, pressure, HGL, etc. at different points in WCS and how these parameters change with respect to time. This data is then used to design the structures like tunnel lining, steel liner, thrust collars, bifurcation pieces. Transient analysis is also performed to work out dimensions of surge tanks.

Installing and running WHAMO

WHAMO is free to download from internet. The software contains one binary executable file. There is no need to install it on pc, and it can be simply run by running the executable file. Input file is programmed in any text editor (like notepad, textpad, notepad++, etc.) and then the path of input file is given when prompted by WHAMO.exe. After running the program, an output file is created in text format.

Programming a system in WHAMO

The input file has to be created in text format. General format of input file looks as follows:

- SYSTEM
  - SYSTEM command is used to define entire water conductor system. It includes creation of nodes (like intermediate nodes, junctions, etc.), creation of ELEMENTS between nodes (like conduits, valves, etc.) and creation of ELEMENTS at nodes (like reservoir, surge tank, turbine, etc.)
  - Second step in SYSTEM is to define ELEMENT properties. All properties of all elements defined in above steps are defined in this step.
- SCHEDULE
  - This command is used to define SCHEDULE properties. This is generally used to define the time variation of flow at boundary conditions (like at turbines)
- HISTORY, PLOT, SNAPSHOT
  - These commands are used to define output request. HISTORY is used to get history of a particular parameter at a particular point for the entire time of simulation in specified time step.
- DTCOMP, DTOUT, TMAX
  - These commands are used to define computational parameters. DTCOMP is the command to specify the time interval for computation purpose. DTOUT is used to specify the time interval for output. DTMAX is used to specify the maximum time for which analysis is to be performed.

Sample WHAMO input file

RESERVOIR DEFENITION

```
SYSTEM
 EL HW AT 1
```

HEAD RACE TUNNEL DEFINITION

```
 EL C1 LINK 1   2
 EL C1 LINK 2   3
 EL C1 LINK 3   4
```

JUNCTION DEFENITION

```
 JUNC AT 4
```

SURGE TANK AND ORIFICE DEFINITION

```
 EL D1 LINK 4   5
 EL ST  AT 5
```

PRESSURE SHAFT

```
 EL C2 LINK 4   6
 EL C3 LINK 6   7
 EL C4 LINK 7   8
```

BOUNDARY CONDITION DEFENITION

```
 EL FB1 AT 8
```

KNOWN NODE ELEVATIONS

```
 NODE   1 ELEV 5614
 NODE   2 ELEV 5614
 NODE   3 ELEV 5614
 NODE   4 ELEV 5614
 NODE   5 ELEV 5614
 NODE   6 ELEV 5614

 NODE   7 ELEV 4993
 NODE   8 ELEV 4993

 FINISH
```

RESERVOIR PARAMETERS

```
RESERVOIR ID HW ELEV 5787 FINISH
```

CONDUIT PARAMETERS

```
CONDUIT ID C1 LENGTH 4702.6  NUMSEG 35 DIAM 34.12 FRICT 0.0100 CELER 4700 FINISH
CONDUIT ID C2 LENGTH 164.00  NUMSEG 10 DIAM 17.06 FRICT 0.0100 CELER 3700 FINISH
CONDUIT ID C3 LENGTH 620.00  NUMSEG 10 DIAM 17.06 FRICT 0.0100 CELER 3700 FINISH
CONDUIT ID C4 LENGTH 392.00  NUMSEG 10 DIAM 17.06 FRICT 0.0100 CELER 3700 FINISH
CONDUIT ID D1 DUMMY DIAM 19.222 ADDEDLOSS CPLUS 1.234 CMINUS 1.234 FINISH

FLOWBC ID FB1 QSCHEDULE 1 FINISH

SURGETANK ID ST DIAM 103.67 ELTOP 5971.0 ELBOTT 5614.0 FRICT 0.0085 CELER 4700
FINISH
```

SURGE TANK PARAMETERS

FLOW BOUNDARY CONDITIONS

```
SCHEDULE
QSCHEDULE 1
 TIME 0.0    Q 14601
 TIME 10.0   Q 0.0
 TIME 2000.0 Q 0.0
FINISH
```

OUTPUT REQUEST

```
PLOT  ELEM  ST ELEV NODE  5  PRESSURE HEAD NODE  3  Q FINISH

HISTORY ELEM  ST ELEV NODE  5  PRESSURE HEAD NODE  3  Q FINISH
```

CONTROL PARAMETERS

```
CONTROL
 DTCOMP 0.2 DTOUT 0.2 TMAX  100.0
 DTCOMP 1.0 DTOUT 1.0 TMAX 2000.0
FINISH

GO
GOODBYE
```

राष्ट्रीय जल अकादमी

पुणे स्थित राष्ट्रीय जल अकादमी, केन्द्रीय जल आयोग की एक विशिष्ट संस्था है। जल संसाधन क्षेत्र से जुडे राज्य तथा केन्द्र सरकार में विविध स्तर पर कार्यरत अभियंताओं के प्रशिक्षण के क्षेत्र में राष्ट्रीय जल अकादमी एक "उत्कृष्ट केन्द्र" के रूप में कार्य कर रही है। राष्ट्रीय जल अकादमी जल संसाधन के विकास एवं प्रबन्धन के क्षेत्र में अल्प एवं मध्यम अवधि के पाठ्यक्रमों के नियमित आयोजन के साथ-साथ केन्द्रीय जल अभियंत्रण (वर्ग 'क') सेवा के अंतर्गत चयनित अधिकारियों के लिए लम्बी अवधि का प्रवेशन कार्यक्रम भी आयोजित करता है।

राष्ट्रीय जल अकादमी की वेबसाइट http://nwa.mah.nic.in से इस संबंध में अधिक जानकारी प्राप्त की जा सकती है ।