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जल शक्ति मंत्रालय
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग
केन्द्रीय जल आयोग

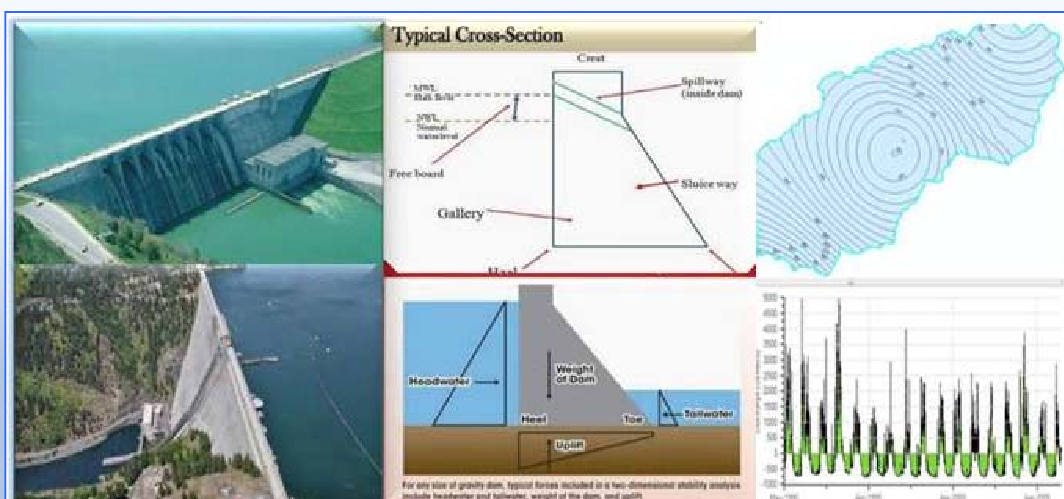
राष्ट्रीय जल अकादमी

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

19 August 2019 – 07 February 2020

डिजाइन और अनुसंधान

Module I : Basic Sciences, Analysis and Design Aspects of Gravity Dams



पुणे
नवंबर 2019

Government of India
Central Water Commission
National Water Academy

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Module-I

BASIC SCIENCES, ANALYSIS AND DESIGN OF GRAVITY DAMS

11 -22 November 2019

Module Co-ordinator

Shri S N Pande, Director (Designs)

Pune
November 2019

MODULE - I

Basic Sciences, Analysis and Design of Gravity Dams

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INDUCTION TRAINING PROGRAM– CWC_NWA

(Prof. Dr.Sukhanand Bhosale)

Introduction to Soil Mechanics

Mr.Karl Terzhaghi spent about 50 years (1913-1963) in doing pioneering research on soil and published his first book titled Erdbaumechanik (means Soil Mechanics) in 1925. This was the first attempt in the world to treat soil mechanics on the basis of the physical properties of the soils, since prior to this, all the designs were used to be based on intuition, experience and empirical formulas, which never provided full confidence about the safety and economy of the design. No wonder then, Mr.Karl Terzhaghi is known as the father of the Soil Mechanics. In 1922 Pavlovsky of Russia solved the complex problem of seepage below the hydraulic structures and enunciated the electrical analogy method for seepage computation.The important **Darcy's law** relating to the flow of water through the soils, and the **Stokes's law** for the settlement of the solid particles in a liquid were enunciated in the year 1856. These laws play very important role even today, in soil engineering.

Soil Mechanics is a discipline of Civil Engineering that concerns the application of the principles of mechanics, hydraulics to engineering problems related to soil. The study of the science of soil mechanics equips a civil engineer with the basic scientific tools needed to understand soil behaviour and its application as an engineering material for construction of earthen dams, bridges and dykes, etc.

Soil consists of a multiphase aggregation of solid particles, water, and air. This fundamental composition gives rise to unique engineering properties, and the description of its mechanical behaviour requires some of the most classic principles of engineering mechanics.

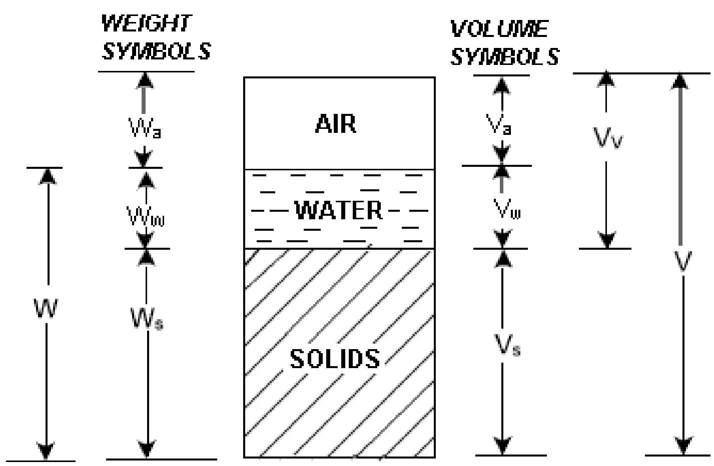
Engineers are concerned with soil's mechanical properties: permeability, seepage, stiffness, and strength. These depend primarily on the nature of the soil grains, the current stress, the water content and unit weight.

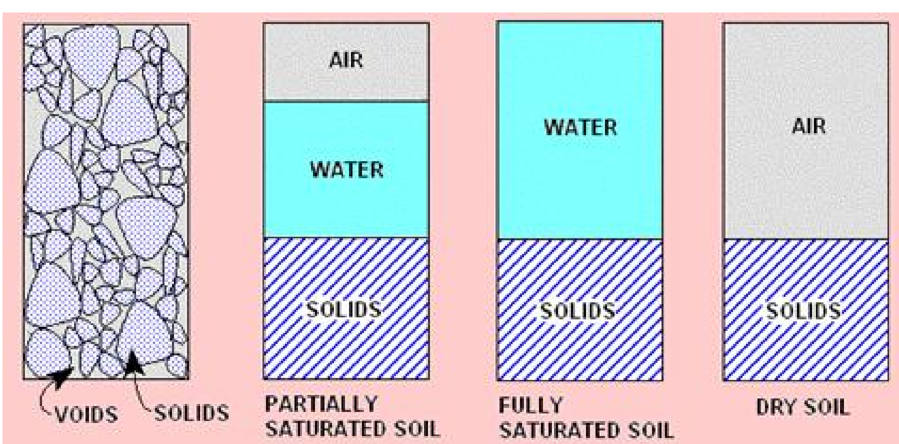
Field of Soil Mechanics is very vast. The civil engineer has many diverse and important encounters with soil. Apart from the testing and classification of various types of soil in order to determine its physical properties, the knowledge of soil mechanics is particularly helpful in the following problems in civil engineering

- Design of earth dams
- Foundation design and construction
- Pavement design
- Design of underground structures and earth retaining structures
- Design of embankments and excavations

Basic characteristics of soil

A soil mass consists of solid soil particles, containing void spaces between them. These voids may be filled either with air, or water, or both. A schematic diagram of the three-phase system is shown in terms of weight and volume symbols respectively for soil solids, water, and air. The weight of air can be neglected. For the purpose of engineering analysis and design, it is necessary to express relations between the weights and the volumes of the three phases.





Volume relation

1. Void ratio (e) is the ratio of the volume of voids (V_v) to the volume of soil solids (V_s), and is expressed as a decimal.

$$e = \frac{V_V}{V_S}$$

2. Porosity (n) is the ratio of the volume of voids to the total volume of soil (V), and is expressed as a percentage.

$$n = \frac{V_V}{V} x\,100$$

Void ratio and porosity are inter-related to each other as follows:

$$e = \frac{n}{1-n} \quad \text{and} \quad n = \frac{e}{(1+e)}$$

3. The volume of water (V_w) in a soil can vary between zero (i.e. a dry soil) and the volume of voids. This can be expressed as the **degree of saturation (S)** in percentage.

$$S = \frac{V_W}{V_V} x\,100$$

For a dry soil, S = 0%, and for a fully saturated soil, S = 100%.

4. Air content (a_c) is the ratio of the volume of air (V_a) to the volume of voids.

$$a_c = \frac{V_a}{V_V}$$

5. Percentage air voids (n_a) is the ratio of the volume of air to the total volume.

$$n_a = \frac{V_a}{V} x\,100 = n\,x\,a_c$$

Weight relation

1. The ratio of the mass of water present to the mass of solid particles is called the **water content (w)**, or sometimes the **moisture content.**

$$w = \frac{W_W}{W_S}$$

Its value is 0% for dry soil and its magnitude can exceed 100%.

2. The mass of solid particles is usually expressed in terms of their **particle unit weight** (γ_s) or **specific gravity** (**G_s**) of the soil grain solids .

$$\gamma_s = \frac{W_s}{V_s} = G_s.\gamma_W$$

where γ_W = Unit weight of water

For most inorganic soils, the value of G_s lies between 2.60 and 2.80. The presence of organic material reduces the value of G_s.

3. Dry unit weight (γ_d) is a measure of the amount of solid particles per unit volume.

$$\gamma_d = \frac{W_s}{V}$$

4. Bulk unit weight $(\gamma_t \text{ or } \gamma)$ is a measure of the amount of solid particles plus water per unit volume.

$$\gamma_t = \gamma = \frac{(W_s + W_W)}{(V_s + V_V)}$$

5. Saturated unit weight (γ_{sat}) is equal to the bulk density when the total voids is filled up with water.

6. Buoyant unit weight (γ') or **submerged unit weight** is the effective mass per unit volume when the soil is submerged below standing water or below the ground water table.

$$\gamma' = \gamma_{sat} - \gamma_W$$

Volume Weight Inter-relation

$$w = \frac{W_W}{W_s} = \frac{\gamma_W.V_W}{G_s.\gamma_W.V_s} = \frac{V_W}{G_s.V_s} = \frac{S.V_v}{G_s.V_s} = \frac{S.e}{G_s}$$

$$\gamma = \frac{(G_s + S.e).\gamma_w}{1+e}$$

$$\gamma = \frac{(1+w).G_s.\gamma_w}{1+e}$$

$$\gamma_d = \frac{G_s.\gamma_w}{1+e}$$

$$\gamma_d = \frac{\gamma}{1+w}$$

$$\gamma' = \frac{[(G_s - 1) + (S-1)e]\ x\ \gamma_w}{1+e}$$

$$\gamma' = \frac{(G_s - 1).\gamma_w}{1+e}$$

Properties of Soil for analysis and design

Volume Weight Characteristics

- Moisture Content
- Density
- Porosity
- Void ratio
- Specific Gravity

Plasticity Characteristics

- Liquid limit
- Plastic limit
- Plasticity index
- Shrinkage limit
- Shrinkage index
- Liquidity index
- Consistency index
- Activity

Gradation Characteristics

- Effective Diameter
- Percent grain size
- Uniformity coefficient
- Coefficient of curvature

- Clay size fraction

Drainage Characteristics

- Coefficient of Permeability
- Capillary head

Consolidation Characteristics

- Coefficient of compressibility
- Coefficient of volume compressibility
- Compression index
- Swell or expansion index
- Coefficient of consolidation
- Coefficient of secondary compression
- Preconsolidation pressure

Strength Characteristics

- Angle of internal friction
- Cohesion intercept
- Unconfined compression strength
- In-situ shear strength
- Bearing capacity factors
- Sensitivity
- Modulus of elasticity
- Lateral earth pressure coefficients

Characteristics of Compacted Soils

- Maximum unit weight
- Optimum moisture content
- Relative density
- California bearing ratio

Soil Classification and Identification

Classification of soil is the separation of soil into classes or groups each having similar characteristics and potentially similar behaviour There can be various criteria for classifying soils such as origin, colour, smell, porosity, water content, sulphate content, strength, permeability, compressibility, size of the particles, plasticity of the soil. The relevant criteria to classify soils are grain size distribution and plasticity of the soil.

Grain size distribution (IS: 2720 (Part 4) – 1985)

In accordance to IS two test methods are available in finding the distribution of grain sizes **Wet sieve analysis** shall be applicable to all soils and **dry sieve analysis** shall be applicable only to soils which do not have an appreciable amount of clay. For the determination of distribution of grain sizes smaller than 75-microns the pipette method is given as the standard method; the hydrometer method is given as a subsidiary method. This method shall not applicable if less than 10 percent of the material passes the 75-micron IS Sieve.

Test Procedure - Dry Sieve Analysis

1. The soil fractions retained on and passing 4’75-mm IS Sieve shall be taken separately for the analysis. The portion of the soil sample retained on 4*75-mm IS Sieve shall be weighed and the mass recorded.
2. The quantity of the soil sample taken shall depend on the maximum particle size contained in the soil.
3. The sample shall be separated into various fractions by sieving through the set of Indian Standard Sieves taken in descending order. While sieving through each sieve, the sieve shall be agitated so that the sample rolls in irregular motion over the Sieve.
4. The material from the sieve may be rubbed, if necessary, with the rubber pestle in the mortar taking care to see that individual soil particles are not broken and re-sieved to make sure that only individual particles are retained.
5. The weight retained in each sieve is measured. The cumulative percentage quantities finer than the sieve sizes (passing each given sieve size) are then determined.
6. The resulting data is presented as a distribution curve with **grain size** along x-axis (log scale) and **percentage passing** along y-axis (arithmetic scale).

Test Procedure - Wet Sieve analysis

1. The portion of the soil passing 4.75mm IS Sieve shall be oven-dried at 105 to 110°C.

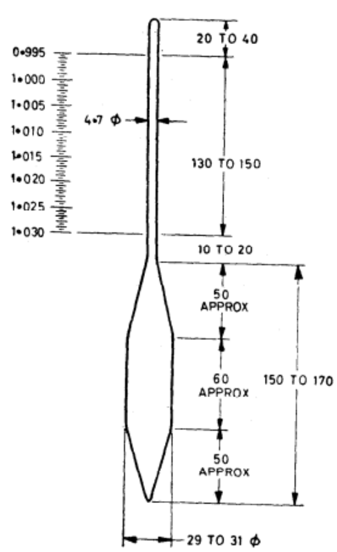
2. The oven-dried material shall then be riffled so that a fraction of convenient mass is obtained. This shall be about 200 g if a substantial proportion of the material only, just passes the 4.75-mm IS Sieve or less if the largest size is smaller.
3. The fraction shall be weighed to 0.1 percent of its total mass and the mass recorded.
4. The riffled and weighed fraction shall be spread out in the large tray or bucket and covered with water.

5. Two grams of sodium hexametaphosphate or one gram of sodium hydroxide and one gram of sodium carbonate perlitre of water used should then be added to the soil. The mix should be thoroughly stirred and left for soaking.
6. The soil soaked specimen should be washed thoroughly over the nest of sieves nested in order of their fineness with the finest sieve (75-micron IS Sieve) at thebottom.
7. Washing shall be continued until the water passing each sieve is substantially clean. Care shall be taken to see that the sieves are not overloaded in the process.
8. The fraction retained on each sieve should be emptied carefully without any loss of material in separate trays. Oven dried at 105 to 110°C and each fraction weighed separately and the masses recorded.

Sedimentation analysis

Sedimentation analysis is used only for the soil fraction finer than 75 microns If the passing from the 75 micron is more than 12 percentages then hydrometer analysis is widely used to estimate the distribution of the particle size from 0.0075 mm to around 0.001 mm.

Soil particles are allowed to settle from a suspension. The decreasing density of the suspension is measured at various time intervals. The procedure is based on the principle that in a suspension, the terminal velocity of a spherical particle is governed by the diameter of the particle and the properties of the suspension.The hydrometer used in this analysis is shown below



All dimensions in millimetres.

In this method, the soil is placed as a suspension in a jar filled with distilled water to which a deflocculating agent is added. The analysis is based on **Stokes's law**, according to which the velocities of free fall of spherical, fine particles, through a liquid are different for different sizes.

The soil particles are then allowed to settle down. The concentration of particles remaining in the suspension at a particular level can be determined by using a hydrometer. Specific gravity readings of the solution at that same level at different time intervals provide information about the size of particles that have settled down and the mass of soil remaining in solution.The results are then plotted between **% finer (passing)** and **log size**.

The diameter of the particle in suspension at any sampling time t shall be calculated from the formula

$$D = \sqrt{\frac{30\,\mu}{980\,(G - G_1)}}\;\sqrt{\frac{H}{t}}$$

where D = diameter of particle in suspension, in mm;

μ = coefficient of viscosity of water in poises at the temperature of suspension at the temperature of suspension at the time of sampling;

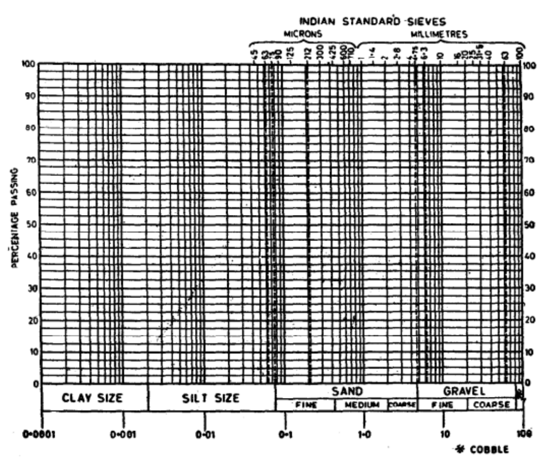
G = specific gravity of the soil fraction used in the sedimentation analysis, in g/cm3;

GI = specific gravity of water, in g/cm';

H = height of fall of the particles or sampling depth, in cm; and

t = time elapsed before sampling, in minutes.

The chart for recording grain size distribution is shown below



Grain size in millimetres.

The sample form is shown below to record the results of grain size distribution for sieve analysis and for hydrometer analysis

PROJECT .... DETAILS OF SOIL SAMPLE.........

SIEVE ANALYSIS OF FRACTION RETAINED ON 4·75-mm IS SIEVE

Weight of total soil sample taken for analysis.....................................

Water content...

IS Sieve Designation	Mass of Soil Retained + Mass Container	Mass of Container	Mass of Soil Retained	Cumulative Mass Retained	Soil Retained as Percentage of Soil Taken	Soil Passing as Percentage of Soil Taken	Combined Percentage Passing as Percentage of Total Soil Sample

Hydrometer Analysis

Hydrometer No. ....

Meniscus correction (C_m)..

Temperature correction (M_t)..

Dispersing agent correction (x)..

Density of soil G...

$$W = \frac{100\ G}{W_b\ (G-1)}(R_h + M_t - x)\ \text{(percent)}$$

Date	Time	Temperature	Elapsed Time	Hydrometer Reading R'_h	Corrected Hydrometer Reading $R_h = R'_h + C_m$	Equipment Diameter, D (mm)	$R_h + M_t - x$	Percentage of Particles Finer than D, W Percentage	Combined Percentage of Finer than D as Percentage of Total Sample

The grading characteristics are then determined as follows:

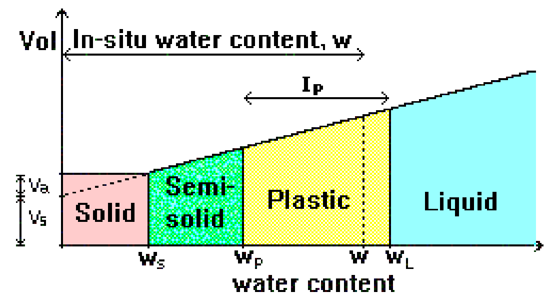
1. Effective size = D_{10}

2. Uniformity coefficient, $C_u = \frac{D_{60}}{D_{10}}$

3. Curvature coefficient, $C_c = \frac{(D_{30})^2}{D_{60} D_{10}}$

Both C_u and C_c will be 1 for a single-sized soil. C_u > **5** indicates a **well-graded soil**, i.e. a soil which has a distribution of particles over a wide size range. C_c **between 1 and 3** also indicates a well-graded soil. $C_u < 3$ indicates a **uniform soil**, i.e. a soil which has a very narrow particle size range.

Consistency of Soilisthe relative ease with which soil can be deformed. This term is mostly used for fine – grained soils for which the consistency is related to a large extent to water content. The liquid and plastic limits of soils are both dependent on the amount and type of clay in a soil and form the basis for the soil classification system for cohesive soils based on the plasticity tests. Besides their use for identification, the plasticity tests give information concerning the cohesion properties of soil and the amount of capillary water which it can hold. They are also used directly in specifications for controlling soil for use in fill. These index properties of soil have also been related to various other properties of the soil.



A gradual increase in water content causes the soil to change from solid to *semi*-solid to plastic to liquid states. The water contents at which the consistency changes from one state to the other are called consistency limits (or Atterberg limits)**.**The three limits are known as the shrinkage limit ($\mathbf{W_S}$), plastic limit ($\mathbf{W_P}$), and liquid limit ($\mathbf{W_L}$)as shown. The values of these limits can be obtained from laboratory tests with reference to IS 2720 (Part V) - 1985. Sample result sheet for liquid and plastic limit is shown below.

	LIQUID LIMIT					PLASTIC LIMIT				
Determination number	1	2	3	4	5	1	2	3	4	5
Number of drops										
Container number										
Weight of container + wet soil, g										
Weight of container + oven dry soil, g										
Weight of water, g										
Weight of container, g										
Weight of oven dry soil, g										
Moisture percent										

Result Summary:

Liquid Limit w_L	Flow Index I_f	Plastic Limit w_p	Plasticity Index I_p	Toughness Index I_T	Liquidity Index I_L	Consistency Index I_c
(1)	(2)	(3)	(4)	(5)	(6)	(7)

Indian Standard Soil classification system **(IS 1498 – 1970)**

The grain-size range is used as the basis for grouping soil particles into boulder, cobble, gravel, sand, silt or clay. Gravel, sand, silt, and clay are represented by group symbols G, S, M, and C respectively.

Very coarse soils	**Boulder size**		**> 300 mm**
	Cobble size		**80 - 300 mm**
Coarse soils	**Gravel size (G)**	***Coarse***	**20 - 80 mm**
		Fine	**4.75 - 20 mm**
	Sand size (S)	***Coarse***	**2 - 4.75 mm**
		Medium	**0.425 - 2 mm**
		Fine	**0.075 - 0.425 mm**
Fine soils	**Silt size (M)**		**0.002 - 0.075 mm**
	Clay size (C)		**< 0.002 mm**

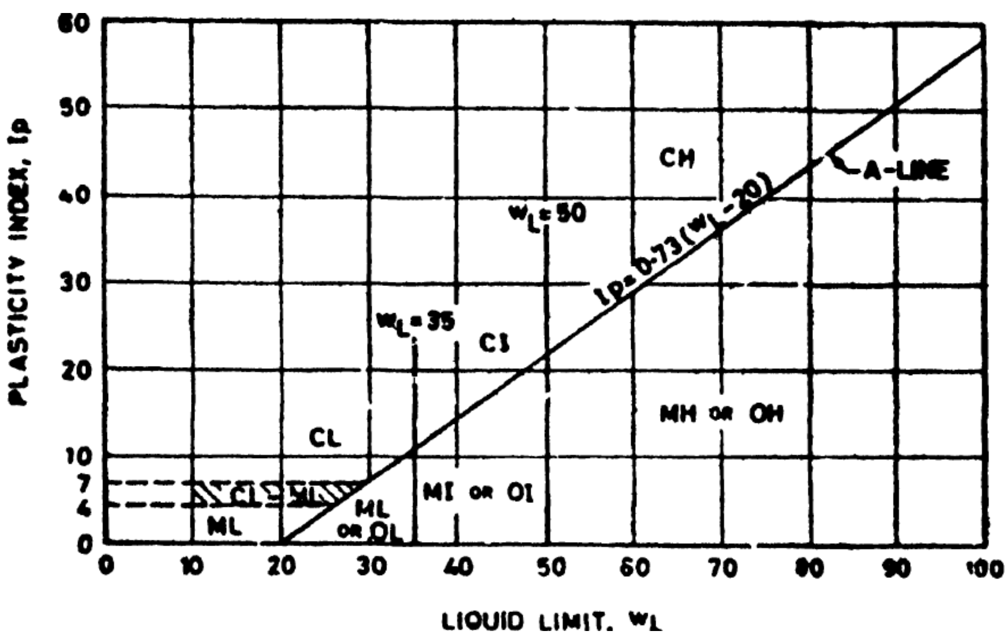


Fig: Plasticity Chart

The laboratory classification criteria for classifying the fine-grained soils are given in the plasticity chart as shown below. The' *A* ' line has the following linear equation between the liquid limit and the plasticity index:

$1p= 0{\cdot}73\ (WL - 20)$

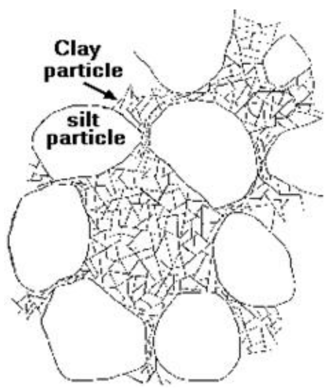
where

Ip - plasticity index, and

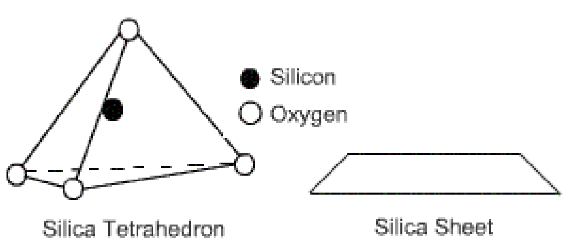
WL -liquid limit.

Structure of clay minerals

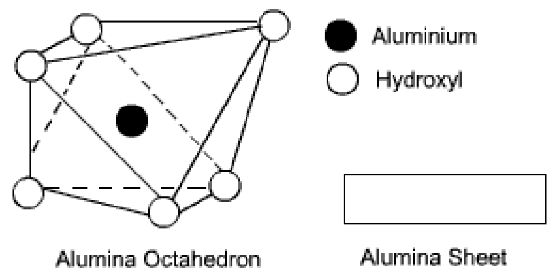
Clay particles are flaky. Their thickness is very small relative to their length & breadth, in some cases as thin as 1/100th of the length. They therefore have high specific surface values. These surfaces carry negative electrical charge, which attracts positive ions present in the pore water. Thus a lot of water may be held as adsorbed water within a clay mass.



A tetrahedral unit consists of a central silicon atom that is surrounded by four oxygen atoms located at the corners of a tetrahedron. A combination of tetrahedrons forms a silica sheet**.**



An octahedral unit consists of a central ion, either aluminium or magnesium, that is surrounded by six hydroxyl ions located at the corners of an octahedron. A combination of aluminium-hydroxyl octahedrons forms a gibbsite sheet, whereas a combination of magnesium-hydroxyl octahedrons forms a brucite sheet.



Field identification of soil

1) Visual Examination

The visual examination of soil is done by naked eye with reference to size, shape (angularity), touch and grading characteristics of soil grains.

2) Wet and manipulated strength test

To perform this test, take a small quantity of the soil in hand, moisten it if needed, and work it with fingers and feel it. If the soil is clayey, a soapy touch would be felt; if the soil is sandy, a feeling of roughness is experienced; and in case of silty soil, moisture will come out if it is squeezed in between the fingers. Also, clay sticks to the fingers and dries slowly, but silt dries fairly quickly, and can be wiped off the fingers easily. This test helps to distinguish the major soil characteristics, that is , whether the soil is clayey, sandy or silty.

3) Thread test

To perform this test, take a small quantity of soil, moisten it if needed, and roll it in between the palms of the hand or on a flat smooth surface into a thread of about 3mm diameter. If crumbling does not occur, fold thee thread and re-roll as before. Repeat the process until the moisture content of the soil has been reduced by drying during manipulation, to the plastic limit, which is indicated by crumbling which occurs as the soil is being rolled. The characteristic of the thread as it approached the plastic limit affords the means of identification of the soil.

4) Dilatancy test - After removing particles retained on IS 425 micron sieve, prepare a pat of moist soil of size of 2cm cube. Add enough water, if necessary to make the soil soft but not sticky. Place the pat in the open palm on one hand and shake horizontally striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a consistency and becomes glossy. When the sample is squeezed between the fingers, the water and the gloss disappear from the surface, the pat stiffens and finally it cracks and crumbles. The rapidity of appearance of water during shaking and its disappearance during squeezing result in identifying the fines in a soil. Very fine clean sand gives quickest and most distinct reaction, whereas, plastic clay has no reaction. Inorganic silt such as typical rock flour, show a moderately quick reaction.

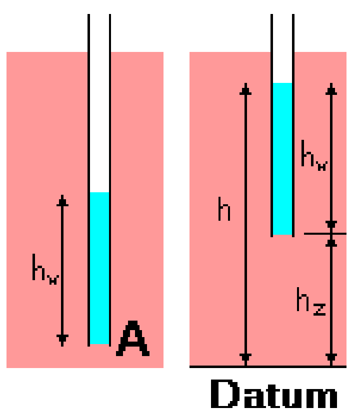
5) Dry strength test

After removing particles retained on IS 425 micron sieve, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength is characteristicfor clays of the CH group. Atypical inorganic silt possesses only very slight dry strength. Silty fine ands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty; whereas typical silt has the smooth feel of flour.

Permeability of soil

Pressure, Elevation and Total heads

In soils, the interconnected pores provide passage for water. A large number of such flow paths act together, and the average rate of flow is termed the coefficient of permeability, or just permeability. It is a measure of the ease that the soil provides to the flow of water through its pores.



At point **A,** the pore water pressure (**u**) can be measured from the height of water in a standpipe located at that point.

The height of the water column is the **pressure head** ($\mathbf{h_w}$).

$\mathbf{h_w = u/\Upsilon_w}$

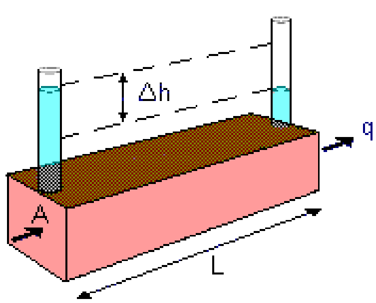
To identify any difference in pore water pressure at different points, it is necessary to eliminate the effect of the points of measurement. With this in view, a datum is required from which locations are measured.

The **elevation head** ($\mathbf{h_z}$) of any point is its height above the datum line. The height of water level in the standpipe above the datum is the **piezometric head** (**h**).

$\mathbf{h = h_z + h_w}$

Total head consists of three components***:*** elevation head, pressure head, and velocity head. As seepage velocity in soils is normally low, velocity head is ignored, and total head becomes equal to the piezometric head. Due to the low seepage velocity and small size of pores, the flow of water in the pores is steady and laminar in most cases. Water flow takes place between two points in soil due to the difference in total heads.

Darcy's law states that there is a linear relationship between flow velocity (**v**) and hydraulic gradient (**i**) for any given saturated soil under steady laminar flow conditions.



If the rate of flow is **q** (volume/time) through cross-sectional area (**A**) of the soil mass, Darcy's Law can be expressed as

$\mathbf{v = q/A = k.i}$

where **k** = permeability of the soil

i = Δh/L

Δ= difference in total heads

L = length of the soil mass

The flow velocity (**v**) is also called the Darcian velocity or the **superficial velocity**. It is different from the actual velocity inside the soil pores, which is known as the **seepage velocity, v_S**. At the particulate level, the water follows a tortuous path through the pores. Seepage velocity is always greater than the superficial velocity, and it is expressed as:

$$v_s = \frac{q}{A_v} = \frac{q}{A_v} \cdot \frac{A}{A} \approx \frac{v}{n}$$

where **A_V** = Area of voids on a cross section normal to the direction of flow

n = porosity of the soil

Permeability of different soil

Permeability (**k**) is an engineering property of soils and is a function of the soil type. Its value depends on the average size of the pores and is related to the distribution of particle sizes, particle shape and soil structure. The ratio of permeabilities of typical sands/gravels to those of typical clays is of the order of **10^6**. A small proportion of fine material in a coarse-grained soil can lead to a significant reduction in permeability.

For different soil types as per grain size, the orders of magnitude for permeability are as follows:

Soil	k (cm/sec)
Gravel	10^0
Coarse sand	10^0 to 10^{-1}
Medium sand	10^{-1} to 10^{-2}
Fine sand	10^{-2} to 10^{-3}
Silty sand	10^{-3} to 10^{-4}
Silt	1×10^{-5}
Clay	10^{-7} to 10^{-9}

Factors affecting Permeability

In soils, the permeant or pore fluid is mostly water whose variation in property is generally very less. Permeability of all soils is strongly influenced by the density of packing of the soil particles, which can be represented by void ratio (**e**) or porosity (**n**).

For Sands

In sands, permeability can be empirically related to the square of some representative grain size from its grain-size distribution. For filter sands, Allen Hazen in 1911 found that $\mathbf{k = 100 (D_{10})^2}$ cm/s where $\mathbf{D_{10}}$= effective grain size in cm.

Different relationships have been attempted relating void ratio and permeability, such as **k µ $\mathbf{e^3/(1+e)}$**, and **k µ $\mathbf{e^2}$**. They have been obtained from the Kozeny-Carman equation for laminar flow in saturated soils.

$$k = \frac{1}{k_0 k_T S_S^2} \cdot \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\eta}$$

where $\mathbf{k_o}$ and $\mathbf{k_T}$ are factors depending on the shape and tortuosity of the pores respectively, $\mathbf{S_S}$ is the surface area of the solid particles per unit volume of solid material, and $\mathbf{g_w}$ and **h** are unit weight and viscosity of the pore water. The equation can be reduced to a simpler form as

$$k = C.\frac{e^3}{1+e} \approx C.e^2$$

For Silts and Clays

For silts and clays, the Kozeny-Carman equation does not work well, and **log k** versus **e** plot has been found to indicate a linear relationship.

For clays, it is typically found that

$$\log_{10} k = \frac{e - e_k}{C_k}$$

where $\mathbf{C_k}$ is the permeability change index and $\mathbf{e_k}$ is a reference void ratio.

Factors affecting the coefficient of permeability

The coefficient of permeability depends on several factors, most of which are listed below.

1. Shape and size of the soil particles.

2. Void ratio. Permeability increases with increase in void ratio.

3. Degree of saturation. Permeability increases with increase in degree of saturation.

4. Composition of soil particles. For sands and silts this is not important; however, for soils with clay minerals this is one of the most important factors. Permeability depends on the thickness of water held to the soil particles, which is a function of the cation exchange capacity, valence of the cations, and so forth. Other factors remaining the same, the coefficient of permeability decreases with increasing thickness of the diffuse double layer.

5. Soil structure. Fine-grained soils with a flocculated structure have a higher coefficient of permeability than those with a dispersed structure.

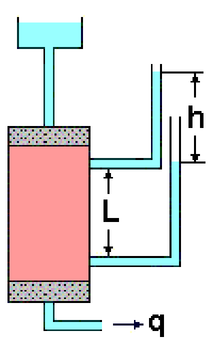
6. Viscosity of the permeant.

7. Density and concentration of the permeant.

Laboratory measurement of Permeability

Constant Head Flow

Constant head permeameter is recommended for coarse-grained soils only since for such soils, flow rate is measurable with adequate precision. As water flows through a sample of cross-section area **A**, steady total head drop **h** is measured across length **L**.

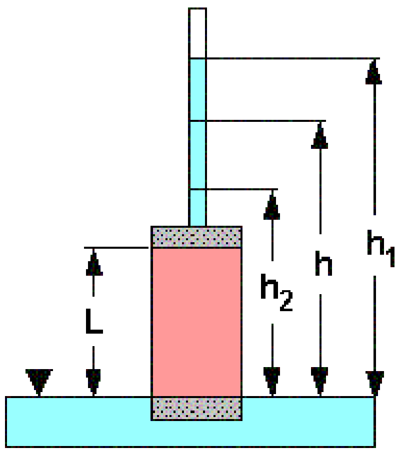


Permeability **k** is obtained from:

$$k = \frac{qL}{Ah}$$

Falling Head Flow

Falling head permeameter is recommended for fine-grained soils.



Total head **h** in standpipe of area **a** is allowed to fall. Hydraulic gradient varies with time.

Heads $\mathbf{h_1}$ and $\mathbf{h_2}$ are measured at times $\mathbf{t_1}$ and $\mathbf{t_2}$. At any time **t**, flow through the soil sample of cross-sectional area **A** is

$$q = k.h.\frac{A}{L} \text{ --------------------- (1)}$$

Flow in unit time through the standpipe of cross-sectional area **a** is

$$= a \times \left(-\frac{dh}{dt} \right) \text{ ----------------- (2)}$$

Equating **(1)** and **(2)** ,

$$-a.\frac{dh}{dt} = k.h\frac{A}{L}$$

or $$-\frac{dh}{h} = \left(\frac{kA}{La} \right) dt$$

Integrating between the limits,

$$\log_e\left(\frac{h_1}{h_2}\right) = \frac{k.A}{L.a}(t_2 - t_1)$$

$$k = \frac{L.a\,\log_e\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

$$= \frac{2.3L.a\log_{10}\left(\frac{h_1}{h_2}\right)}{A(t_2 - t_1)}$$

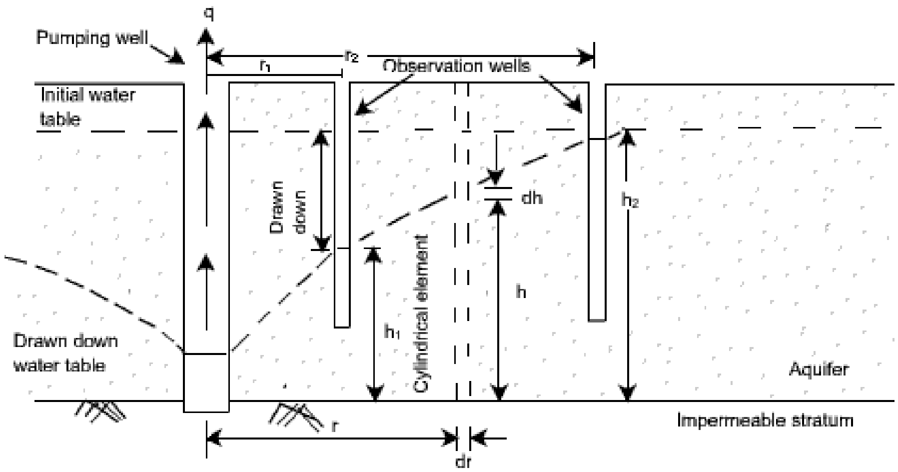
Field test for permeability

Field or *in-situ* measurement of permeability avoids the difficulties involved in obtaining and setting up undisturbed samples in a permeameter. It also provides information about bulk permeability, rather than merely the permeability of a small sample.

A field permeability test consists of pumping out water from a main well and observing the resulting drawdown surface of the original horizontal water table from at least two observation wells. When a steady state of flow is reached, the flow quantity and the levels in the observation wells are noted.

Two important field tests for determining permeability are: Unconfined flow pumping test, and confined flow pumping test.

Unconfined Flow Pumping Test



In this test, the pumping causes a drawdown in an unconfined (i.e. open surface) soil stratum, and generates a radial flow of water towards the pumping well. The steady-state heads $\mathbf{h_1}$ and $\mathbf{h_2}$ in observation wells at radii r_1 and r_2 are monitored till the flow rate **q** becomes steady.

The rate of radial flow through any **cylindrical surface** around the pumping well is equal to the amount of water pumped out. Consider such a surface having radius **r**, thickness **dr** and height **h.** The hydraulic gradient is

$$i = \frac{dh}{dr}$$

Area of flow, $A = 2\pi rh$

From Darcy's Law,

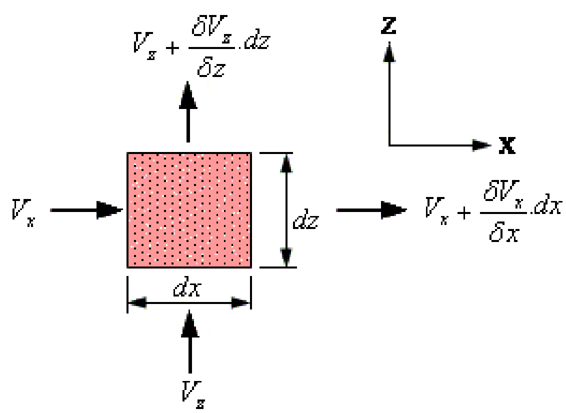
$$q = k.i.A$$
$$= k.\frac{dh}{dr}.2\pi rh$$

Arranging and integrating,

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2r}{q}.K.h.dh$$

$$k = \frac{q.\log_e(\frac{r_2}{r_1})}{r(h_2{}^2 - h_1{}^2)}$$

Seepage in soils



A rectangular soil element is shown with dimensions ***dx*** and ***dz*** in the plane, and thickness ***dy*** perpendicular to this plane. Consider planar flow into the rectangular soil element.

In the **x-direction,** the net amount of the water entering and leaving the element is

$$\frac{\delta V_x}{\delta x}.dx.dy.dz$$

Similarly in the **z-direction,** the difference between the water inflow and outflow is

$$\frac{\delta V_z}{\delta z}.dz.dx.dy$$

For a two-dimensional steady flow of pore water, any imbalance in flows into and out of an element in the z-direction must be compensated by a corresponding opposite imbalance in the x-direction. Combining the above, and dividing by ***dx.dy.dz*** , the **continuity equation** is expressed as

$$\frac{\delta V_x}{\delta x} + \frac{\delta V_z}{\delta z} = 0$$

From Darcy's law, $V_x = k_x \cdot \frac{\delta h}{\delta x}$, $V_z = k_z \cdot \frac{\delta h}{\delta z}$, where **h** is the head causing flow.

When the continuity equation is combined with Darcy's law, the equation for flow is expressed as:

$$k_x \cdot \frac{\delta^2 h}{\delta x^2} + k_z \cdot \frac{\delta^2 h}{\delta z^2} = 0$$

For an isotropic material in which the permeability is the same in all directions (i.e. $k_x = k_z$), the **flow equation** is

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

This is the **Laplace equation** governing two-dimensional steady state flow. It can be solved ***graphically, analytically, numerically, or analogically.***

For the more general situation involving ***three-dimensional*** steady flow, Laplace equation becomes: $\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} + \frac{\delta^2 h}{\delta z^2} = 0$

One dimensional flow

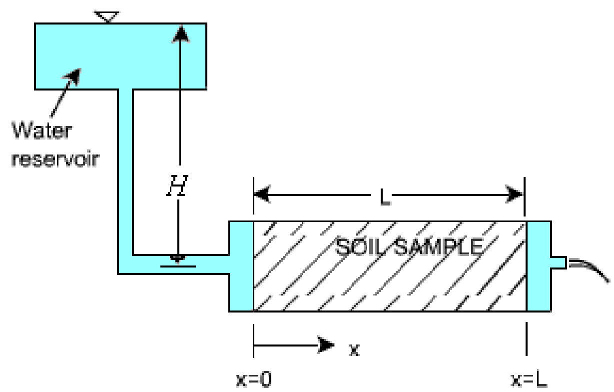
For this, the **Laplace Equation** is $\frac{\delta^2 h}{\delta x^2} = 0$

Integrating twice, a general solution is obtained.

$$\frac{\delta h}{\delta x} = c_1$$

$$h = c_2 + c_1 x$$

The values of constants can be determined from the specific boundary conditions.



As shown, at $x = 0$, $h = H$, and at $x = L$, $h = 0$

Substituting and solving,

$c_2 = H$, $c_1 = -\frac{H}{L}$

The specific solution for flow in the above permeameter is

$$h = H - \frac{H}{L}x$$

which states that head is dissipated in a linearly uniform manner over the entire length of the permeameter.

Procedure for drawing flow nets

At every **point (x,z)** where there is flow, there will be a value of head **h(x,z).** In order to represent these values, contours of equal head are drawn.

A flow net is to be drawn by trial and error. For a given set of boundary conditions, the flow net will remain the same even if the direction of flow is reversed. Flow nets are constructed such that the head lost between successive **equipotential lines** is the same, say **h**. It is useful in visualising the flow in a soil to plot the flow lines, as these are lines that are tangential to the flow at any given point. The steps of construction are:

1. Mark all boundary conditions, and draw the flow cross section to some convenient scale.

2. Draw a coarse net which is consistent with the boundary conditions and which has orthogonal equipotential and flow lines. As it is usually easier to visualise the pattern of flow, start by drawing the flow lines first.

3. Modify the mesh such that it meets the conditions outlined above and the fields between adjacent flow lines and equipotential lines are 'square'.

4. Refine the flow net by repeating **step 3.**

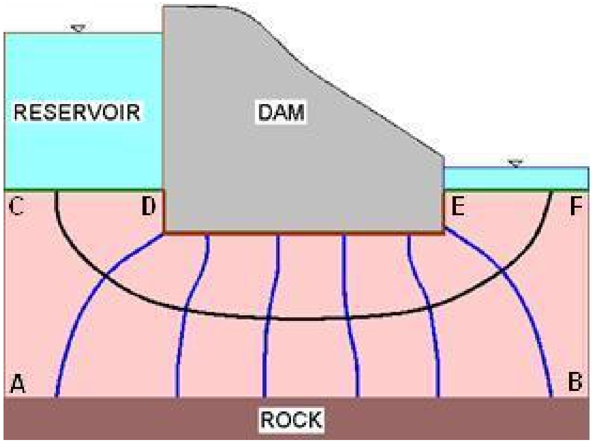
The most common **boundary conditions** are:

(a) A submerged permeable soil boundary is an equipotential line. This could have been determined by considering imaginary standpipes placed at the soil boundary, as for every point the water level in the standpipe would be the same as the water level. (Such a boundary is marked as CD and EF in the following figure.)

(b) The boundary between permeable and impermeable soil materials is a flow line (This is marked as AB in the same figure).

(c) Equipotential lines intersecting a phreatic surface do so at equal vertical intervals.

Uses of flow nets



The graphical properties of a flow net can be used in obtaining solutions for many seepage problems such as:

1. ***Estimation of seepage losses from reservoirs:*** It is possible to use the flow net in the transformed space to calculate the flow underneath the dam.

2. ***Determination of uplift pressures below dams:*** From the flow net, the pressure head at any point at the base of the dam can be determined. The uplift pressure distribution along the base can be drawn and then summed up.

3. ***Checking the possibility of piping beneath dams:*** At the toe of a dam when the upward exit hydraulic gradient approaches unity, boiling condition can occur leading to erosion in soil and consequent piping. Many dams on soil foundations have failed because of a sudden formation of a piped shaped discharge channel. As the stored water rushes out, the channel widens and catastrophic failure results. This is also often referred to as piping failure.

Consolidation of soil

Elastic settlement is on account of change in shape at constant volume, i.e. due to vertical compression and lateral expansion. **Primary consolidation (**or simply **consolidation)** is on account of flow of water from the voids, and is a function of the permeability and compressibility of soil. **Secondary compression** is on account of creep-like behaviour.

Primary consolidation is the major component and it can be reasonably estimated. A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering. For the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain take place in one direction only, as **one-dimensional consolidation** in the vertical direction.

Compaction of soils

Compaction is the application of mechanical energy to a soil so as to rearrange its particles and reduce the void ratio.

It is applied to improve the properties of an existing soil or in the process of placing fill such as in the construction of embankments, road bases, runways, earth dams, and reinforced earth walls. Compaction is also used to prepare a level surface during construction of buildings. There is usually no change in the water content and in the size of the individual soil particles.

The objectives of compaction are:

- To increase soil shear strength and therefore its bearing capacity.
- To reduce subsequent settlement under working loads.
- To reduce soil permeability making it more difficult for water to flow through.

Laboratory Compaction

The variation in compaction with water content and compactive effort is first determined in the laboratory. There are several tests with standard procedures such as:

- Indian Standard Light Compaction Test (similar to Standard Proctor Test)
- Indian Standard Heavy Compaction Test (similar to Modified Proctor Test)

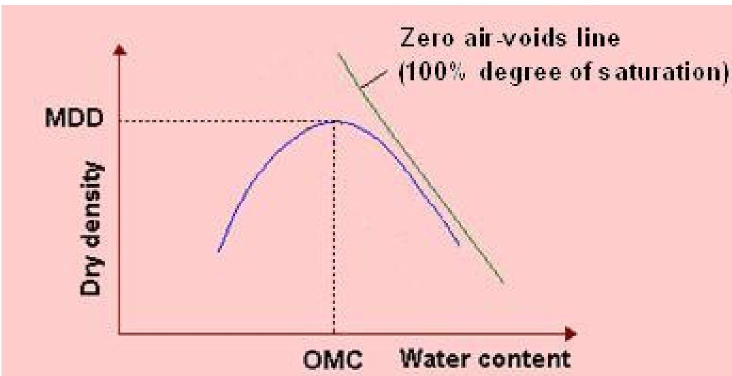
Indian Standard Light Compaction Test

Soil is compacted into a 1000 cm^3 mould in 3 equal layers, each layer receiving 25 blows of a 2.6 kg rammer dropped from a height of 310 mm above the soil. The compaction is repeated at various moisture contents.

Indian Standard Heavy Compaction Test

It was found that the Light Compaction Test (Standard Test) could not reproduce the densities measured in the field under heavier loading conditions, and this led to the development of the Heavy Compaction Test (Modified Test). The equipment and procedure are essentially the same as that used for the Standard Test except that the soil is compacted in 5 layers, each layer also receiving 25 blows. The same mould is also used. To provide the increased compactive effort, a heavier rammer of 4.9 kg and a greater drop height of 450 mm are used.

Dry density and water content relationship

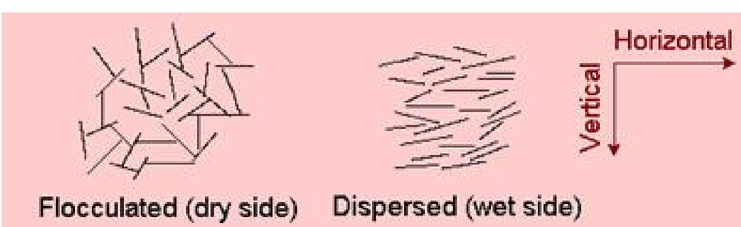


Engineering behaviour of compacted soil

The water content of a compacted soil is expressed with reference to the OMC. Thus, soils are said to be compacted **dry of optimum** or **wet of optimum** (i.e. on **the dry side** or **wet side** of OMC). The structure of a compacted soil is not similar on both sides even when the dry density is the same, and this difference has a strong influence on the engineering characteristics.

SoilStructure

For a given compactive effort, soils have a flocculated structure on the dry side (i.e. soil particles are oriented randomly), whereas they have a dispersed structure on the wet side (i.e. particles are more oriented in a parallel arrangement perpendicular to the direction of applied stress). This is due to the well-developed adsorbed water layer (water film) surrounding each particle on the wet side.



Swelling

Due to a higher water deficiency and partially developed water films in the dry side, when given access to water, the soil will soak in much more water and then swell more.

Shrinkage

During drying, soils compacted in the wet side tend to show more shrinkage than those compacted in the dry side. In the wet side, the more orderly orientation of particles allows them to pack more efficiently.

Construction Pore Water Pressure

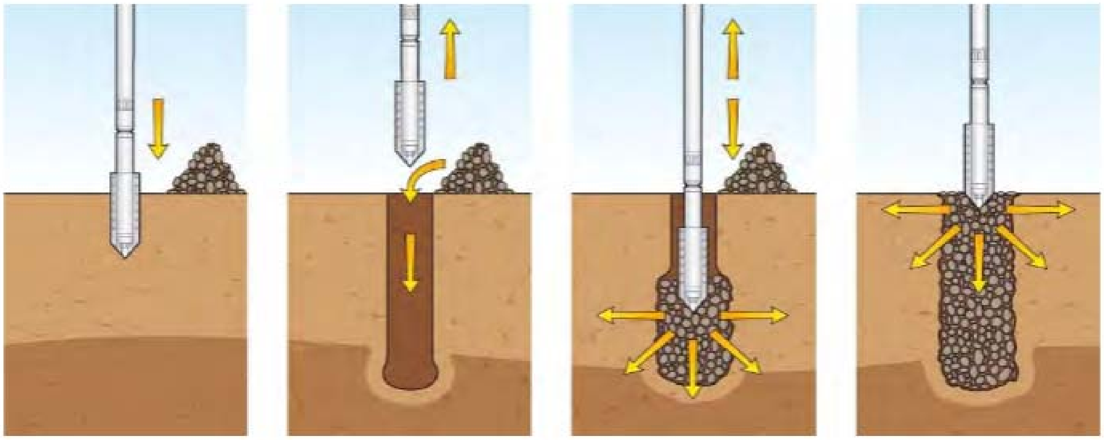
The compaction of man-made deposits proceeds layer by layer, and pore water pressures are induced in the previous layers. Soils compacted wet of optimum will have higher pore water pressures compared to soils compacted dry of optimum, which have initially negative pore water pressure.

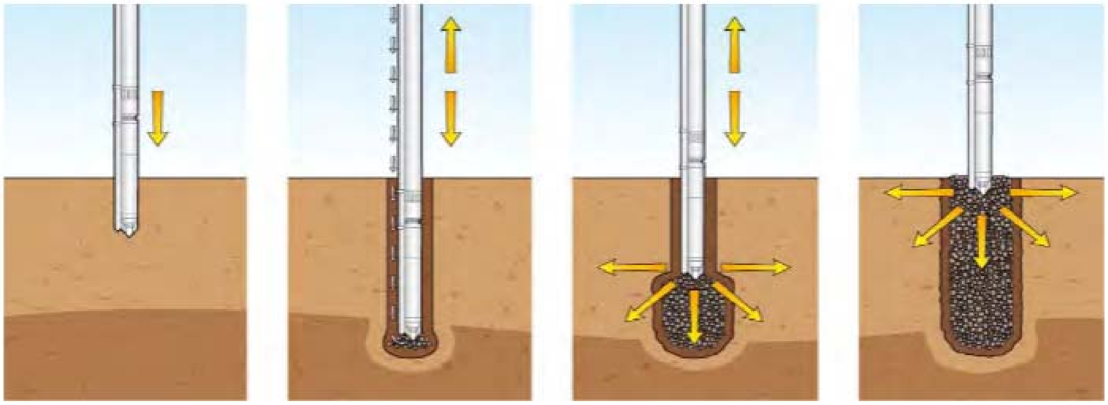
Permeability

The randomly oriented soil in the dry side exhibits the same permeability in all directions, whereas the dispersed soil in the wet side is more permeable along particle orientation than across particle orientation.

Soil Stabilization

Stone columns – Top feeded and bottom feeded procedure are illustrated in the diagram shown below





Other stabilization methods and materials

Lime. Slaked lime is most often used in the **stabilization** of subgrades and road bases, particularly in **soil** that is clay-like or highly plastic. ...

Cement

Bitumen

Chemical Compounds

Geotextiles

Mixing **Materials**

Grouting

Electrical **Stabilization**

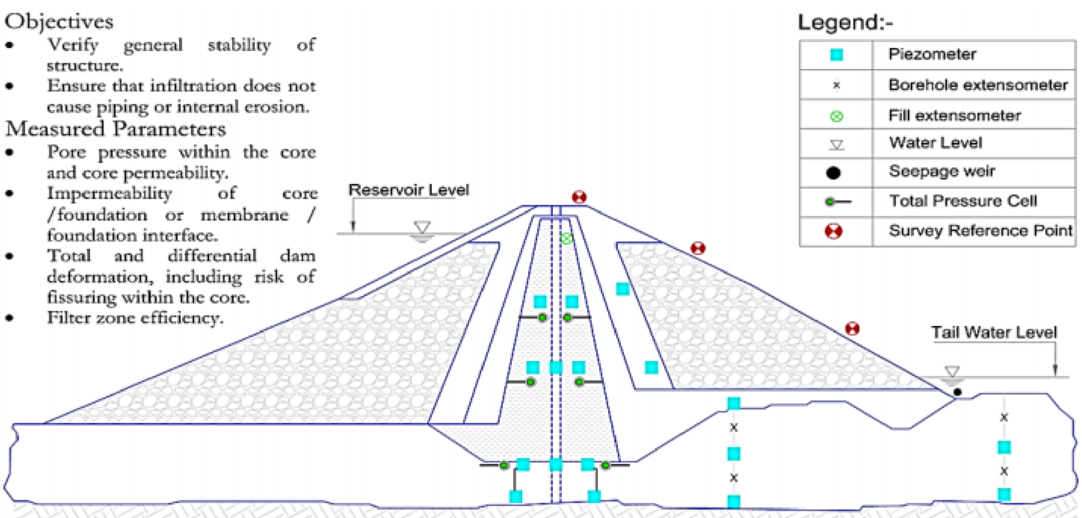
Geotechnical Instrumentation in hydraulic structures

Instrumentation is the use of special devices to obtain critical scientific measurements of engineered behaviour. A typical instrument arrangement consists of one or more of three basic elements: a sensor; a signal conducting media; and a readout/recorder. The data recording may be done by hand, chart recorder, film recorder, digital printout, or magnetic recorder, and in some cases, may be transmitted directly to computer storage. Structural displacements, deformations, settlements, seepages, the piezometric pressure within the structure and its foundation are items that are the focus of a monitoring system

Water Pressure

A certain amount of water seeps through, under, and around all dams. The water moves through the pores in the soil, and through the cracks and joints in the rock. The pressure of the water acts uniformly in all directions; it is termed pore pressure. The upward component of pore pressure, called uplift pressure, has the effect of reducing the effective downward weight of the dam and can decrease the stability of the structure. Devices used to measure pressure include several types of piezometers and total pressure cells.

The below figure illustrates the parameters to be measured at the major cross section of the embankment dam



Piezometers are devices used to measure the water pressure at specific locations in dam bodies, foundations, and abutments. The primary value of water pressure data is to warn of certain conditions or problems that may exist or may be developing. Unusual water pressure data may show that unanticipated movement or seepage is occurring.

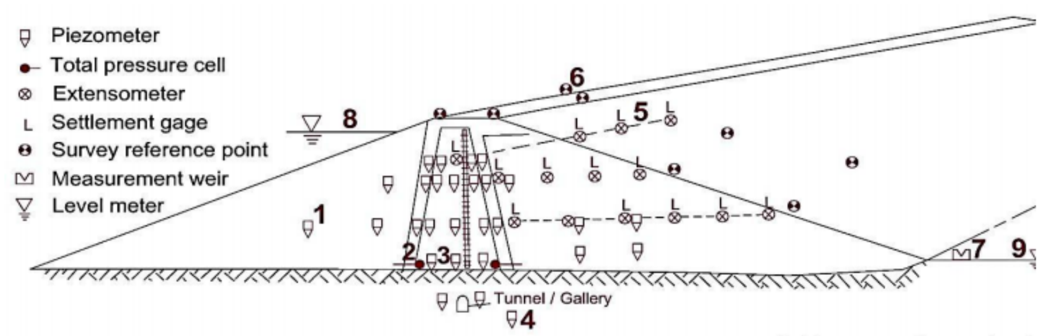
There are two basic types of piezometers: **(1) hydraulic piezometers** in which the pressure is obtained directly by measuring the water level or the pressure into the tube; and **(2) electrical piezometers** in which the pressure is measured with electrical-acoustic or electric resistant pressure gauges (manometers), or with pneumatic sensors.

Stress and Strain - Design stresses may not always occur as expected in a completed dam. For this reason, special total pressure instruments are used to measure the actual stresses at selected locations, such as between a dam and its abutments or foundation, or between certain components of the dam. The purpose of total pressure monitoring is to measure the total pressure (total load) on a contact surface or within the mass of the dam. Several types of devices (cells) are used to measure the static total pressure in a dam. The measured load can be caused by earth, water, or concrete. A dam's principal stresses can be evaluated based on data

Earth pressures within fill and against concrete structures are measured with earth pressure cells, which are also known as total pressure cells. They consist of two flexible diaphragms sealed around the periphery, with a fluid in the annular space between the diaphragms. Pressure is measured by the increase in fluid pressure behind the diaphragm with pneumatic or vibrating wire sensors. Earth pressure cells should have similar stiffness as the surrounding soil to avoid inaccurate measurements of in-situ stress caused by arching.

A variety of mechanical and **electrical strain gauges** are used to measure strain in concrete structures. Some of the instruments are designed to be embedded in the dam during construction, and others are surface mounted following construction. Strain gages are often installed in groups so that the three-dimensional state of strain can be evaluated

The figure below illustrates the overall instrumentation for the core dam.



1. Pore pressure measurement for monitor flow conditions, homogeneity of each zone and efficiency of drains and core permeability. 2. Total pressure measurement to detect arching effects that can cause horizontal fissuring resulting from load transfer and hydraulic fracturing. 3. Permeability monitoring at core-foundation interface.	4. Flow monitoring in foundation - grout curtain efficiency (rock or alluvium) 5. Extensometric and settlement system to monitor internal movements and detect potential fissuring zones, especially in abutment zone. 6. Survey reference points and fissuring bench marks.	7. Measurement of seepage flow through dam. 8. Reservoir level measurement 9. Downstream level measurement.

Temperature measurements of a dam, foundation, ambient conditions or instrumentation are used to reduce data from other instruments, increase precision, or to interpret results. For example, movements of concrete dams and changes in leakage at concrete dams are often related to changes in temperature.

Temperature is also measured in concrete dams under construction to evaluate mix design, placement rates, and block and lift sizes; to time grouting of block joints; and to assess thermal loads. **Resistance thermometers or thermocouples** can measure the temperatures of a dam, its foundation, and other instruments.

References:

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7)”Basic and applied soil mechanics” – Gopal ranjan Rao

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10) Guidelines for instrumentation for large dams – jan 2018 Dam safety rehabilitation directorate

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APPLICATION OF FINITE ELEMENT METHOD IN DESIGN OF GRAVITY DAMS UNDER EARTHQUAKE LOAD COMBINATIONS (DYNAMIC ANALYSIS)

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INTRODUCTION

During 20th century, many large dams have been constructed in all parts of India. These dams have been designed based on experience or simplified methods which often resulted in either over safe design or unsafe design. Due to frequent occurrences of strong earthquakes in northern parts (Himalayan region), eastern parts (Assam and other north-eastern region), western parts (Bhuj earthquake in Gujarat and Koyna and Latur earthquakes in Maharashtra) and central parts (Madhya Pradesh) of India, safety of existing and proposed dams lying in these high seismic zones have become of great concern to project authorities. Moreover, these dams have been designed based on recommendations of old BIS codes and old classified seismic zones of India with less magnitude of seismic coefficients. Since then, BIS codes have been revised several times and seismic zones of India have also been reclassified with higher magnitude of seismic coefficients. As per modern design practices, the dam should be safe against earthquake forces under detailed dynamic response analysis using site-specific seismic parameters. Dynamic analysis of dams is much more complicated since interaction between dam and the foundation rock and hydrodynamic forces need to be modeled in a realistic way. Prior to the invention of fast computers i.e., earlier to 1970, all the dams have been designed based on conventional approaches by considering the dam as a vertical cantilever beam and normal stresses have been evaluated at upstream and downstream faces. With the invent of fast computers, more sophisticated structural analysis methods such as Finite Difference Method, Boundary Integration Method and Finite Element Method have been evolved and state of the art finite element softwares have been developed. At present, more rigorous analysis is carried out generally by Finite Element Method by using these general purpose finite element softwares with automesh generation and with facilities of animation of results during post processing. The methods of analysis need to be applied suitably to get correct estimation of the response, as the results are very sensitive to the approximations and idealizations made. Two dimensional(2D) finite element analysis is generally appropriate for concrete gravity dams. It should be understood that the actual response of the structure is three dimensional(3D) therefore the designer should review the analysis and realistic results to assure that the 2D approximation is acceptable and realistic. For spillway blocks, plane stress approach gives more realistic results as compared to plane strain approach. For long conventional concrete dams, a 2D analysis is reasonably correct. Dams located in narrow valleys between steep abutments and dams with foundation of varying rock Modulii which vary across the valley, dams with openings and weak foundation zones are conditions that necessitate 3D modeling and analysis. Further, the analysis results almost become identical from separate 2D and 3D analysis, if ratio of length to height of the dam is greater than 4. Dynamic analysis results need to be calibrated by monitoring the response to actual earthquakes and forced vibration tests.

FEM Softwares

Many general purpose Finite Element Softwares have been developed and are in use all over the world. A few popularly used, general purpose FEM softwares are as follows:

1. ANSYS	2. LUSAS	3.SOLVIA	4.ADINA
5. NASTRAN	6.IDEA	7.SAP	8.ABAQUS
9. HYPERWORKS	10.SOLIDWORKS		

The basic principle of all the softwares is same. The main difference lies in mesh generation capability and post processing of results. The main components of such programs for structural analysis are as follows:

- Input – Definition of physical model, geometry, material loading and
- Boundary conditions.
- Library of elements – Generation of mathematical models.
- Solution – Assembly and solution of algebraic equations.
- Output – Display of calculated displacements and stresses

REQUIREMENT OF DYNAMIC ANALYSIS

The dynamic analysis using site specific earthquake ground motions becomes necessary if following conditions exist:

i. The dam is 100 feet (30m) or more in height and the peak ground acceleration

(PGA) at the site is greater than 0.2 g for maximum credible earthquake i.e.

Zone III.

ii. The dam is less than 100 feet high and the PGA at the site is greater than 0.4 g for the maximum credible earthquake i.e. Zone IV & V.

iii. There are gated spillway monoliths, wide roadways, intake structures, or other monoliths of unusual shape or geometry.

iv. The dam is in a weakened condition because of accident, aging or deterioration.

The requirements for a dynamic stress analysis in this case should be decided in consultation with experts.

SITE SPECIFIC SEISMIC PARAMETERS

Site specific seismic response spectra and acceleration time history of ground motion is generated by Earthquake experts. At CWPRS, Engineering Seismology division is carrying out such type of studies. In brief, two general approaches for developing Site Specific Response Spectra and Acceleration Time History of ground motion are the deterministic and probabilistic approaches.

a. **Deterministic Approach:** In this approach, often termed a deterministic seismic hazard analysis, or DSHA, site ground motions are deterministically estimated for a specific selected earthquake that is, an earthquake of a certain size on a specific seismic source occurring at a certain distance from the site. The earthquake size may be characterized by magnitude or by epicentral intensity. The earthquake magnitude is typically selected to be the magnitude of the largest earthquake judged to be capable of occurring on the seismic source, i.e., MCE. The selected earthquake is usually assumed to occur on the portion of the seismic source that is closest to the site. After the earthquake magnitude and distance are selected, the site ground motions are then estimated using ground motion attenuation relationship or other techniques.

b. Probabilistic Approach: In the probabilistic approach, often termed a probabilistic seismic hazard analysis, or PSHA, site ground motions are estimated for selected values of the probability of ground motion exceedance in a design time period or for selected values of annual frequency or return period for ground motion exceedance. For example, ground motions could be estimated for a 10 percent probability of exceedance in 100 years or for a return period of 950 years. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquake of different magnitudes on the seismic sources, the uncertainty of the earthquake locations on the sources and the ground motion attenuation including its uncertainty.

Seismic coefficients based on IS: 1893 (Part 1) : 2002 are calculated as follows:

$$A_h = (Z/2) \times (I/R) \times (Sa/g) \qquad (1)$$

where:

A_h= Horizontal seismic coefficient

Z = Zone factor,

I = Importance factor

R = Response reduction factor (I/R=1.0)

Sa / g = Average response acceleration coefficient

The value of seismic coefficients in vertical direction is taken as 1/2 to 2/3 of horizontal seismic coefficient A_h .The value of seismic coefficients is taken as 1.5 times of A_h at top of the dam and reduced linearly to zero at base. Sometimes, site specific seismic coefficients are given by project authority based on seismic studies.

PERMISSIBLE STRESSES

The maximum tensile and compressive stresses developed in the dam body should always satisfy the stability criteria as per IS:6512-1984. The maximum permissible tensile stress in concrete gravity dams under simplified approaches as per BIS is taken as 4% of characteristic cube compressive strength. As per latest design practices in India, the allowable tensile stress using modern rigorous methods of analysis under static and pseudodynamic conditions is taken as 1/10 to 1/8 of characteristic cube compressive strength.

As per International practices, the splitting tensile test or the modulus of rupture test can be used to determine the tensile strength. For initial design investigations, static, pseudostatic, pseudodynamic and linear dynamic analysis, permissible tensile strength of concrete is calculated from the following equation (Raphael 1984):

Direct static tensile strength, $f_{st} = 1.7 f_c^{2/3}$ (2)

The static tensile strength determined by the splitting tensile test may be increased by 1.33 to be comparable to the standard modulus of rupture test. Modulus of rupture i.e. bending tensile strength is obtained as follows:

$$f_{mr} = 1.33 \times f_{st} = 2.3 f_c^{2/3} \quad (3)$$

Dynamic tensile strength of concrete is calculated by increasing 50 percent static tensile strength as follows:

$$f_{dst} = 1.5 \times f_{st} = 1.5 \times 1.7 f_c^{2/3} = 2.6 f_c^{2/3} \quad (4)$$

and

$$f_{dmr} = 1.5 \times f_{mr} = 1.5 \times 2.3 f_c^{2/3} = 3.4 f_c^{2/3} \quad (5)$$

where:

f_{st} = Direct static tensile strength, psi

$f_{mr=}$ Bending Tensile strength or apparent tensile strength, psi

f_c = Uniaxial static compressive strength of the concrete, psi

f_{dst}=Dynamic tensile strength

f_{dmr}= Bending dynamic tensile strength or apparent dynamic tensile strength, psi

DAMPING

The damping plays important role in dynamic analysis. Damping in a structure is due to nonlinearities that cause loss of energy. The dynamic motion of any system is damping dependent. The damping is defined as resistance to motion. Most nonlinear analysis use Rayleigh damping which combines a mass damping equation with a stiffness damping equation to arrive at a percent of critical damping.

$$C = \alpha \times M + \beta \times K \quad (6)$$

Where α and β are damping constants, M and K are mass and stiffness matrices. α and β are calculated as follows:

Time Period, $T = 5.55 \times (H^2 / B) \times \mathrm{SQRT}(\rho / gE)$ (7)

Frequency, $f = 1/T$ (8)

Angular frequency, $\omega 1=2\pi f$ (9)

$\gamma_2/\gamma_1= (\omega 2/\omega 1)^n$, (n=0.5-1.0) (10)

$\alpha = (2\times Y / (\omega 1+\omega 2)) \times (\omega 1\times\omega 2)$ (11)

$\beta = (2\times Y / (\omega 1+\omega 2))$ (12)

where:

H= Height of the dam section, B= Base width of dam section

ρ= Mass density of dam material, E= Young's Modulus of dam material

ω1&ω2 = Angular frequencies, γ =% damping ratios, 5 - 10%

For concrete gravity dams, damping is taken as 5-10% of critical damping. In direct step by step integration method, the Rayleigh damping factors corresponding to damping percentage are computed. In Mode superposition analysis, model damping is defined for each mode varying from 5% to 10%. Also in response spectrum analysis, model damping is defined for each mode.

FOUNDATION – DAM – RESERVOIR INTERACTION EFFECTS

In dynamic analysis, the stresses in dam body are affected due to foundation and reservoir water up to certain extent. Therefore it becomes necessary to consider these interaction effects. The interaction effect of foundation rock strata on dam body is considered by incorporation of certain part of foundation rock strata (2-4 times of height of dam) in the finite element modelling. Further, the effect of reservoir is considered by either lumping a certain mass of reservoir water on upstream nodes as per Westergaard's approach or by taking a portion of reservoir into modelling and by applying dam fluid interaction effects using latest softwares.

LOAD COMBINATIONS

The analysis is generally carried out for the following load combinations based on IS: 6512 – 1984:

(Static Load Combinations)

a) **Load Combination A** (Construction Condition): Dam completed but no water in reservoir and no tailwater.

b) **Load Combination B** (Normal Operating Condition): Full reservoir elevation at FRL normal dry weather, tailwater level and silt load at upstream and downstream faces.

c) **Load Combination C** (Flood Discharge Condition): Reservoir at maximum flood pool elevation at MWL, all gates open, tailwater and silt load at upstream and downstream faces.

f) **Load Combination F:** Combination C, but with extreme uplift (drains inoperative).

(Earthquake Load Combinations)

d) **Load Combination D:** Combination A, with earthquake

e) **Load Combination E:** Combination B, with earthquake

g) **Load Combination G:** Combination E, but with extreme uplift (drains inoperative)

DIFFERENT APPROACHES OF DYNAMIC ANALYSIS

The effect of earthquake loads into two dimensional (2D) and three dimensional (3D) analysis is taken into account by following approaches.

Pseudostatic approach (*Already discussed*)

Pseudodynamic approach (*Already discussed*)

1. Dynamic analysis by Response Spectrum analysis
2. Dynamic analysis by Direct step by step Integration approach
   (Newmark Method, Central Difference Method)
3. Dynamic analysis by Mode Superposition Method
4. Dynamic analysis by frequency substructure approach using EAGD-84 FE software

Each approach is described here by analyzing one sample problem.

Definition of the Sample Problem

The dynamic analysis has been carried out for a proposed 62.5 m high non-overflow concrete gravity dam with a base width of 60.4 m located in the highest seismic zone V as per IS: 1893 (Part I)-2002 as shown in Fig.1. The analysis is based on the assumptions that the material in the dam and the foundation is isotropic and homogeneous and it behaves in linear elastic way, that the dam is in plane strain condition, and that all the displacement components reduces to zero at base of foundation block. The properties of the dam and foundation material adopted in the analysis are as given below*:*

Young's Modulus of Dam Concrete: 2.20×10^5 kg/cm^2

Young's Modulus of Foundation rock: 2.50×10^5 kg/cm^2

Poisson's Ratios of Foundation rock: 0.15

Poisson's Ratio of Dam Concrete: 0.20

Mass Density of Dam Concrete: 2.4×10^{-3} kg/cm^3

Mass Density of Water: 1.0×10^{-3} kg/cm^3

Viscous Damping Ratio for Dam: 5%

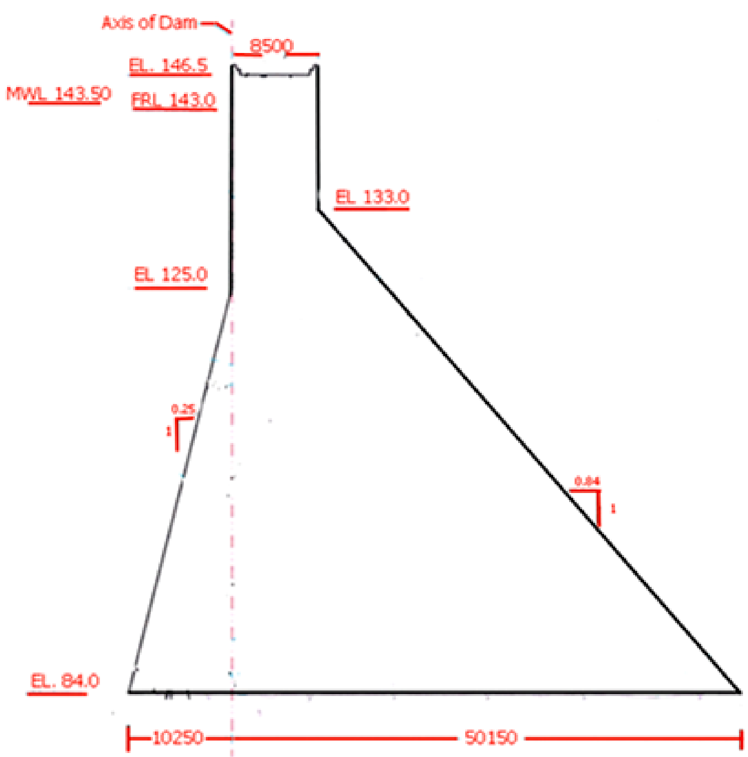


Fig. 1: Cross Section of the Dam

1. DYNAMIC ANALYSIS BY RESPONSE SPECTRUM ANALYSIS

The 2D dynamic analysis has been carried out by modifying the Finite Element Mesh from fine to coarse by utilizing the more integrating points to allow better stress redistribution. Site specific response spectrum generated by CWPRS Pune based on topography, geology and past seismic record has been considered in the analysis. The horizontal and vertical response spectra at different damping are shown in Fig.2.

To take into account the elastic behavior of the foundation rock strata, a section of the foundation part up to a depth equal to more than 4 times the height of the dam and width equal to 4 times of dam base have been included in the analysis. To allow free field ground motion the mass of the foundation is neglected in the analysis. The total 202 numbers of 2-D isoparametric plane strain elements have been generated by defining 891 nodes including generated nodes. The detailed idealization is shown in Fig.3.

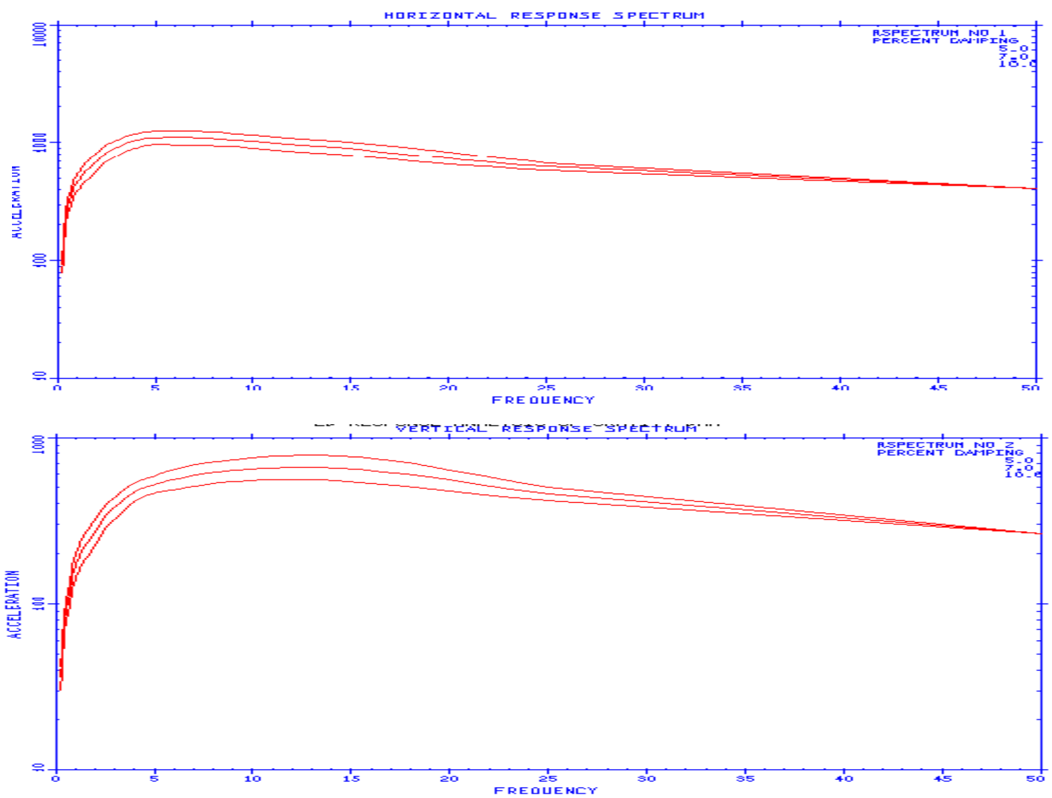


Fig. 2: Horizontal and Vertical Response Spectra

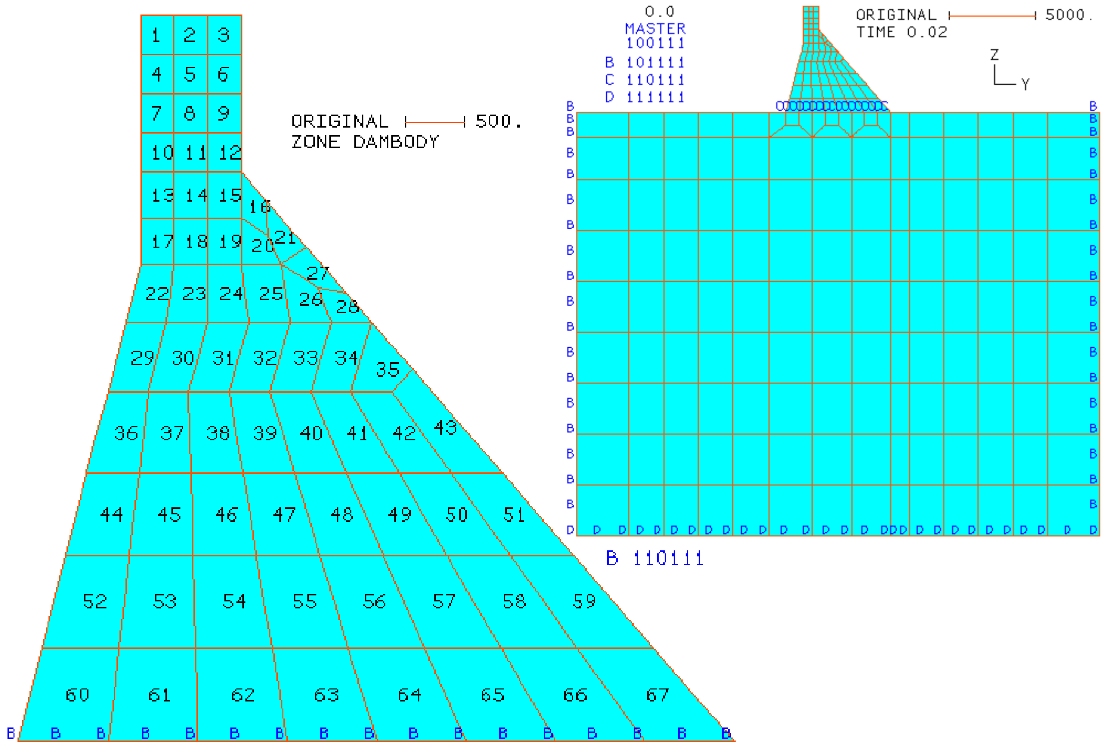


Fig. 3: Finite Element Model of Non Overflow Section for Dynamic Analysis

The boundary conditions are modified under dynamic analysis as compared to other analysis. In foundation, only horizontal movement is allowed at the ends of the foundation block as per recommendation of ICOLD Bulletin 30-Jan 1978. The base of the foundation is assumed fixed i.e. all displacement components are assumed as zero. At the interface of dam body and foundation block, two sets of nodes are defined by giving identical coordinates. The dam body is treated as attached to foundation in vertical direction i.e. dam is not allowed to move horizontally independently and firmly rested on foundation block. At the interface, only vertical movement is allowed in the dam body. At all other places, horizontal and vertical movements are allowed. The detailed boundary conditions are shown in Fig.3. The hydrodynamic effects are modeled as an added mass of water moving with the dam using Westergaard's formula as mentioned above. First, natural frequencies of vibration and corresponding mode shapes for specified modes are computed by applying all static loads as per load combination E. The earthquake loading is computed from earthquake response spectra for each mode of vibration induced by the horizontal and vertical components of response spectra. These modal responses are combined to obtain an estimate of the maximum total response. Stresses are computed by a static analysis of the dam using the earthquake loading as an equivalent static load. The results of static and response analysis are combined by square root of the sum of the squares (SRSS) method for getting complete response. Figs 4-6 show the three mode shapes and fig.7 shows the total response under three modes.

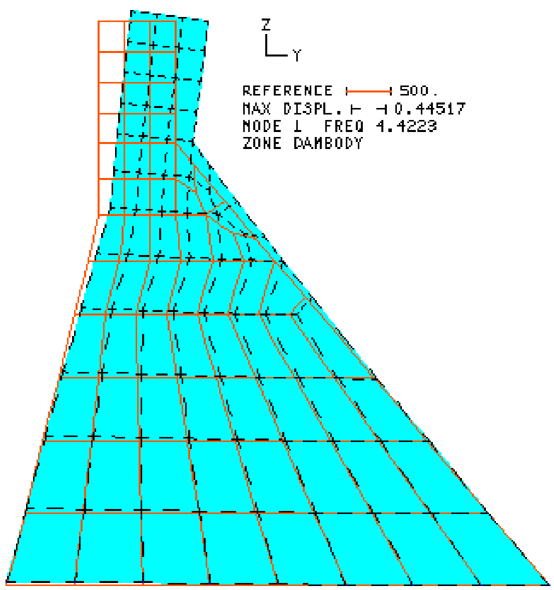


Fig.4: Mode Shape 1

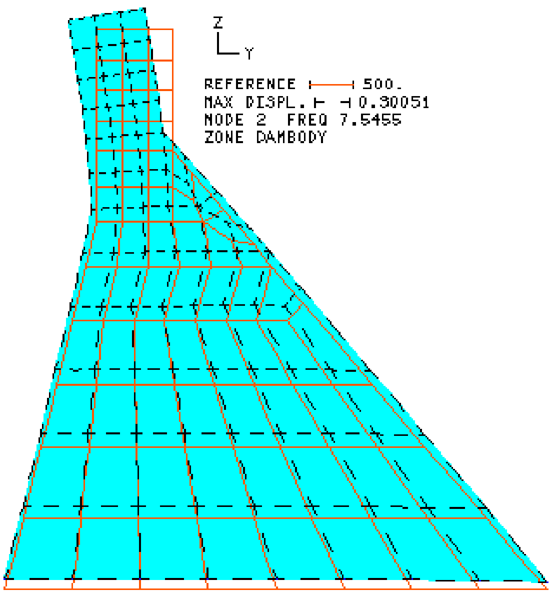


Fig.5: Mode Shape 2

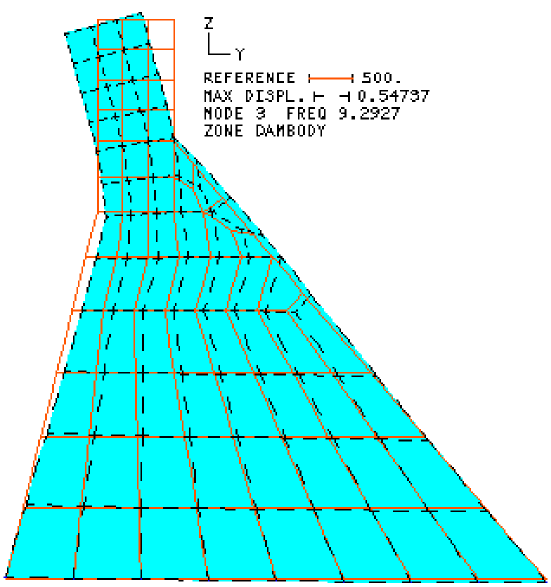


Fig.6: Mode Shape 3

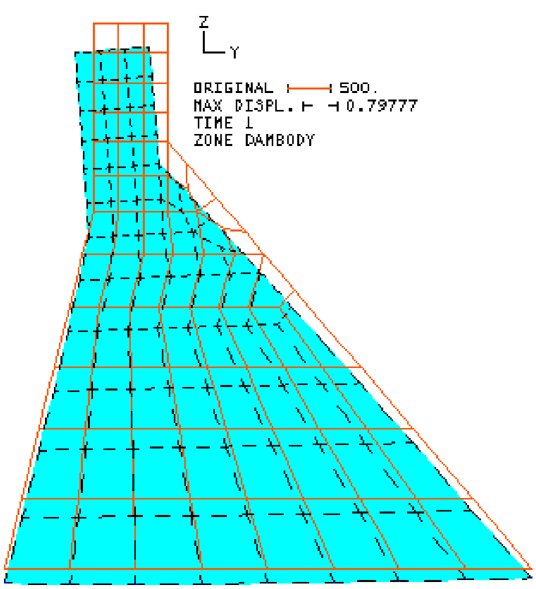


Fig.7: Total Response under 3 Modes

RESULTS

Results are obtained in the form of displacements and principal stresses including static effects. Figs. 8 - 10 show the distribution of horizontal, vertical and resultant displacements in the dam body.

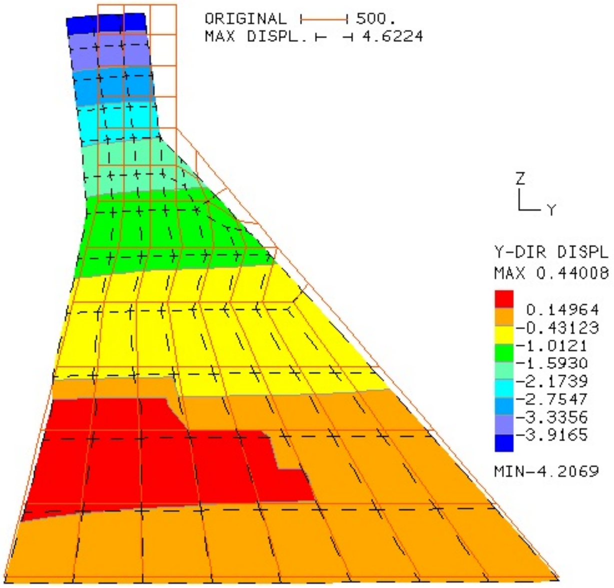


Fig.8: Distribution of Horz. Displacement

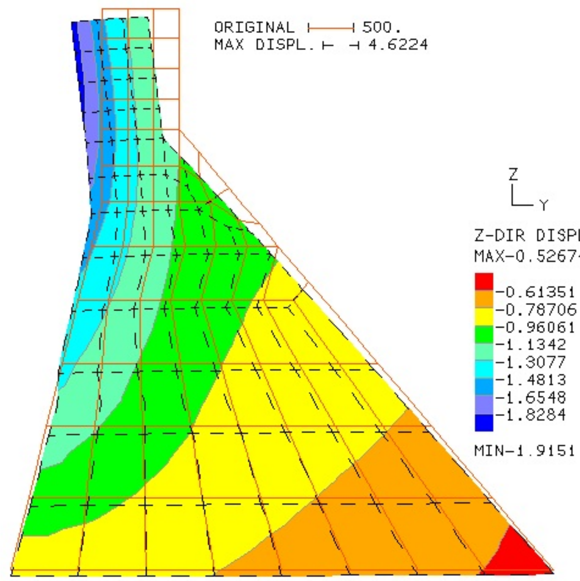


Fig.9: Distribution of Vert. Displacement

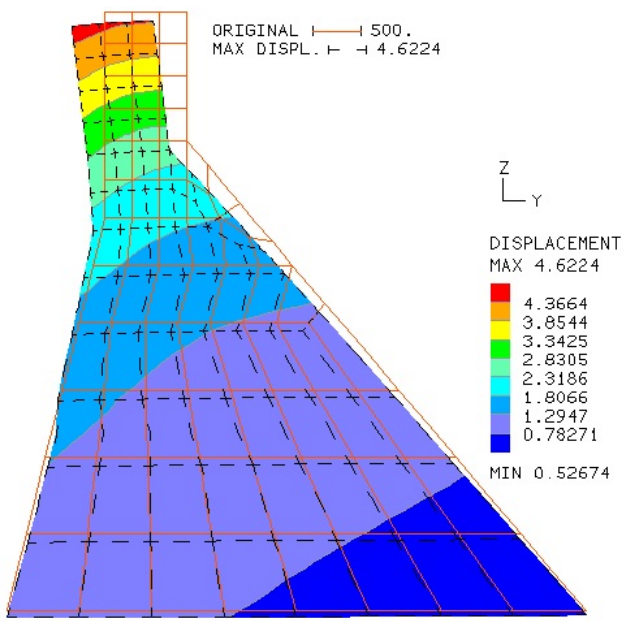


Fig.10: Distribution of Resultant Displacement

Displacements are in cm

As can be seen from above figures the horizontal displacement is towards upstream side which indicates the predominance of earthquake forces. The maximum horizontal displacement under load combination E is found to be 4.20 cm toward upstream side indicating predominance of earthquake forces. Figs.11&12 show the distribution of total maximum and minimum principal stresses including static effect in the dam body. The total maximum principal stress including static effect is found to be 99.528 kg/cm^2 near the downstream slope starting point. The total minimum principal stress is also found to be 108.15 kg/cm^2 near to upstream and downstream slope starting points. The response spectra also gives approximate picture of stresses under earthquake forces and may often result in either underestimation or overestimation of principal stresses.

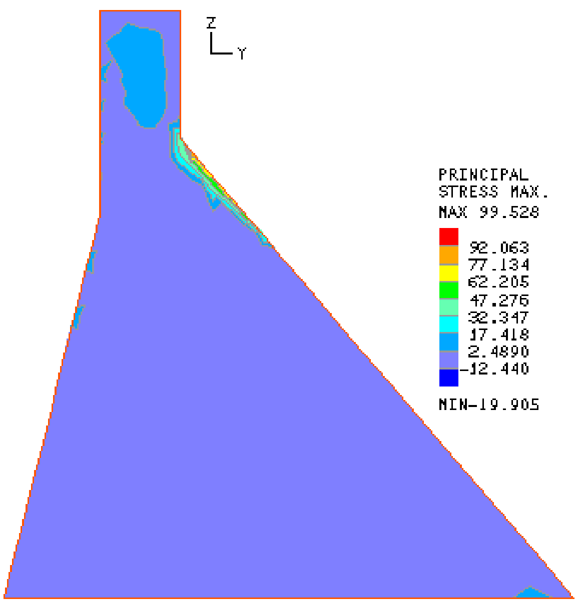


Fig.11: Distribution of Total Maximum Principal Stresses

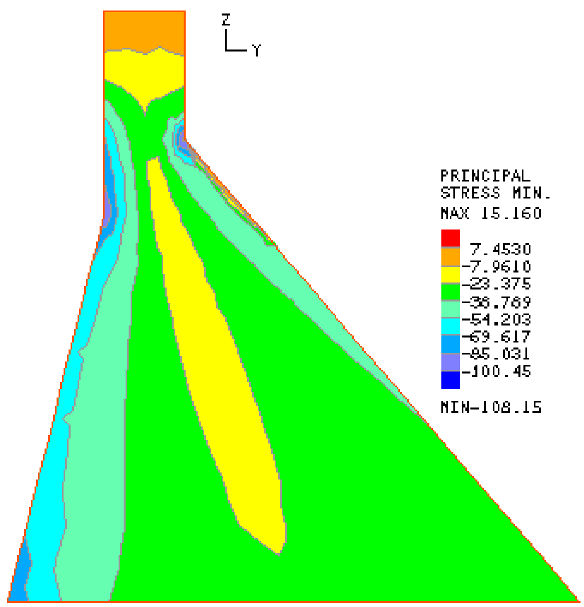


Fig.12: Distribution of Total Minimum Principal Stresses

(+): Tensile, (-): Compressive

2. DYNAMIC ANALYSIS BY DIRECT STEP BY STEP INTEGRATION APPROACH (NEWMAARK METHOD/CENTRAL DIFFERENCE METHOD)

The 2D dynamic analysis has been carried out by using same mathematical model as used for response spectrum analysis shown in fig 3. The dynamic analysis has been performed by taking static as well as dynamic material properties. The dynamic material properties are determined by Resonance Column test (facility available at CWPRS) by taking concrete and rock cores. The Young's Modulus of Elasticity E for old concrete say more than one year is almost same as Static Modulus. Site specific acceleration time histories of horizontal and vertical components of motion generated at CWPRS by taking into account seismotectonics, geology and past earthquake history of the region has been used to define the input excitation. These time-histories are shown plotted in Fig. 14. After generation of mathematical modal based on prepared data as mentioned above, dynamic analysis is performed in two steps. In first step only static analysis is performed and results are saved and plotted for static analysis. In second stage the damping factors are defined and added mass at upstream nodes is added and acceleration time history is defined. The Finite element software considers the static displacements as initial displacements and dynamic analysis by Newmaark method is performed for predefined time steps at the interval of 0.020 seconds. The results are generated at each time step in the form of displacements, direct stresses, principal stresses and effective stresses.

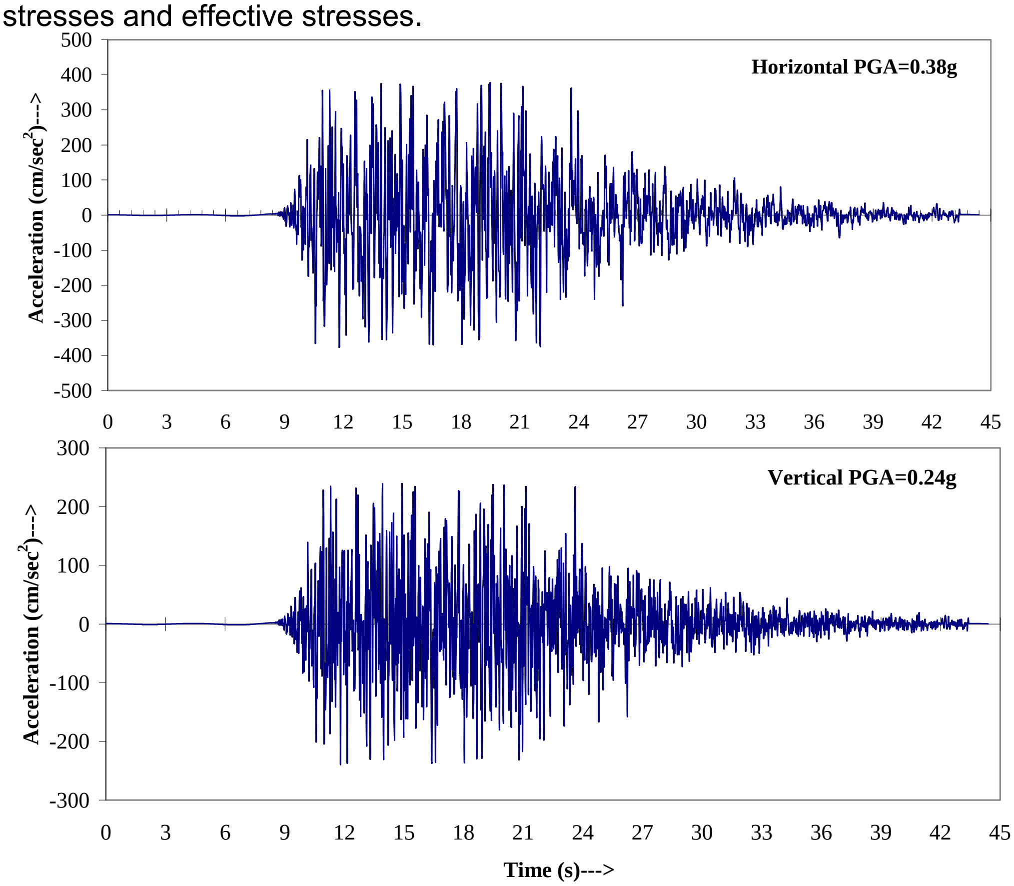


Fig. 14: Acceleration Time History along Horizontal and Vertical Directions

RESULTS

Results are plotted in the form of displacement time history in both directions at selected nodes and element stress history in high stressed zones. The maximum and minimum principal stress envelopes for dambody by taking the principal stresses at the centroid of each element are plotted to know the overall distribution of tensile and compressive stresses. The results are presented for one Earthquake load combination E. Node displacement histories under normal operating condition with earthquake is plotted for node 1 at upstream top point of the dam in both horizontal and vertical directions as shown in Fig.15. The maximum horizontal displacement nearly 2 cm and vertical displacement 1.3 cm are found to have taken place at node 1.

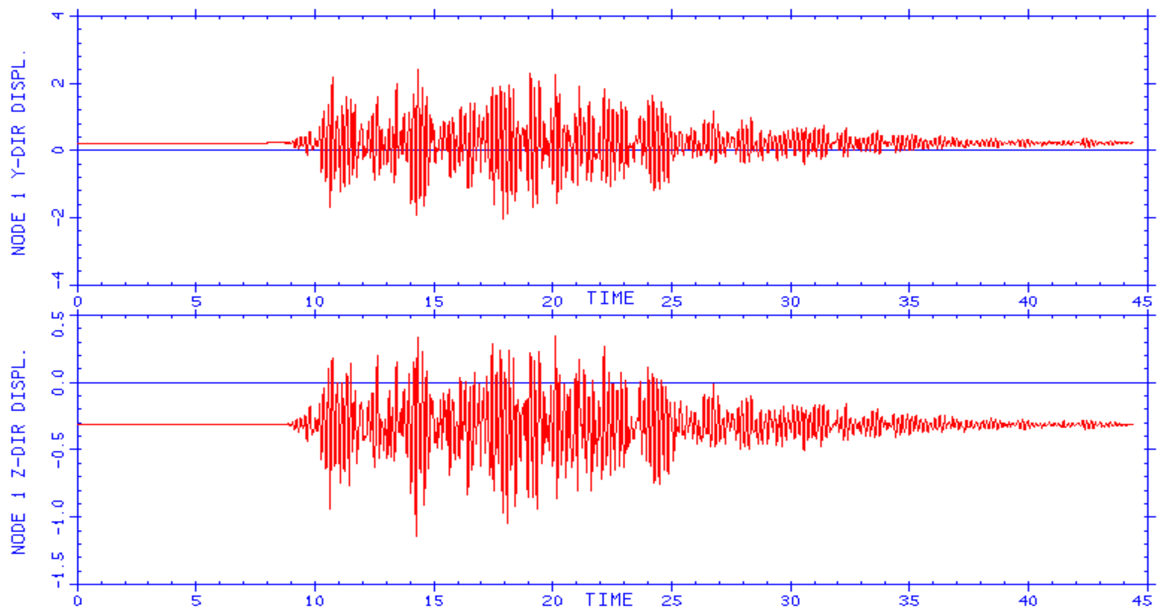


Fig.15: Variation of displacements with time at Node 1

The maximum and minimum principal stress time histories are plotted at the centroid of selected elements to know the stress variation during earthquake. Figs.16-17 show the variation of maximum and minimum principal stress at the centroid of elements 13- 15. The maximum principal stress (tensile) of the order of 40 kg/cm^2 is found to be developed at the centroid of element 15 and minimum principal stress (compression) of the order of 64 kg/cm^2 is also found to be developed at the centroid of element 15.

The envelope of maximum and minimum principal stresses is plotted by taking the principal stresses at centroid of each element in the dambody. The pattern of the tensile and compressive stress distribution is used in assessing the safety of the dam by comparing the calculated stresses with allowable stresses under different loading conditions.

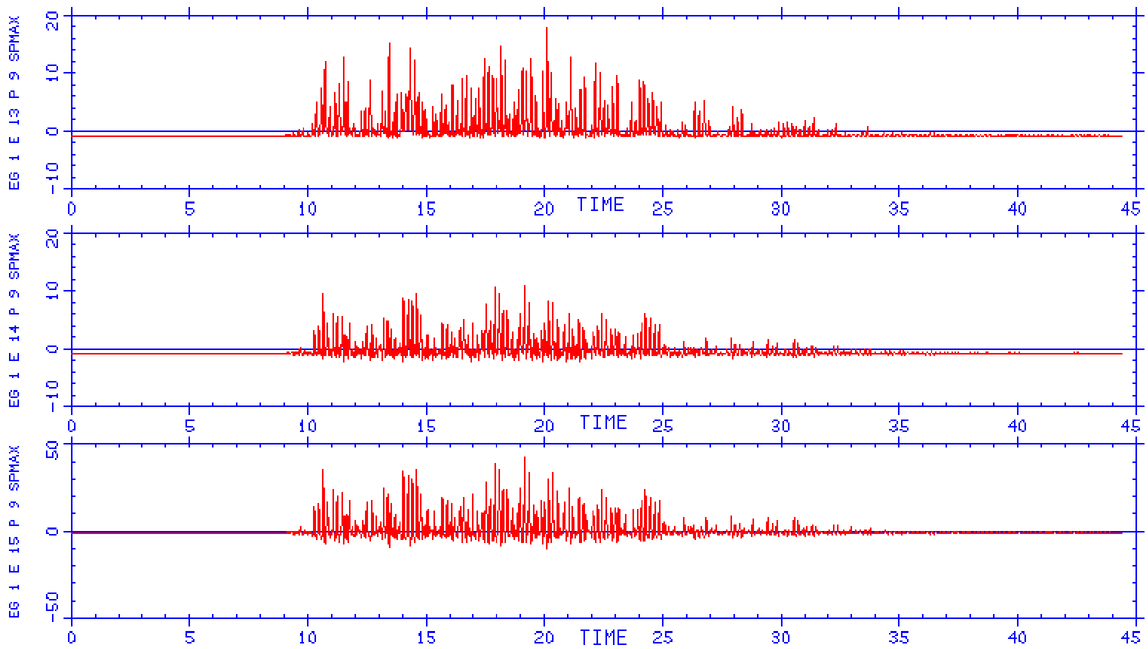


Fig.16: Variation of Maximum Principal Stress with time for selected elements

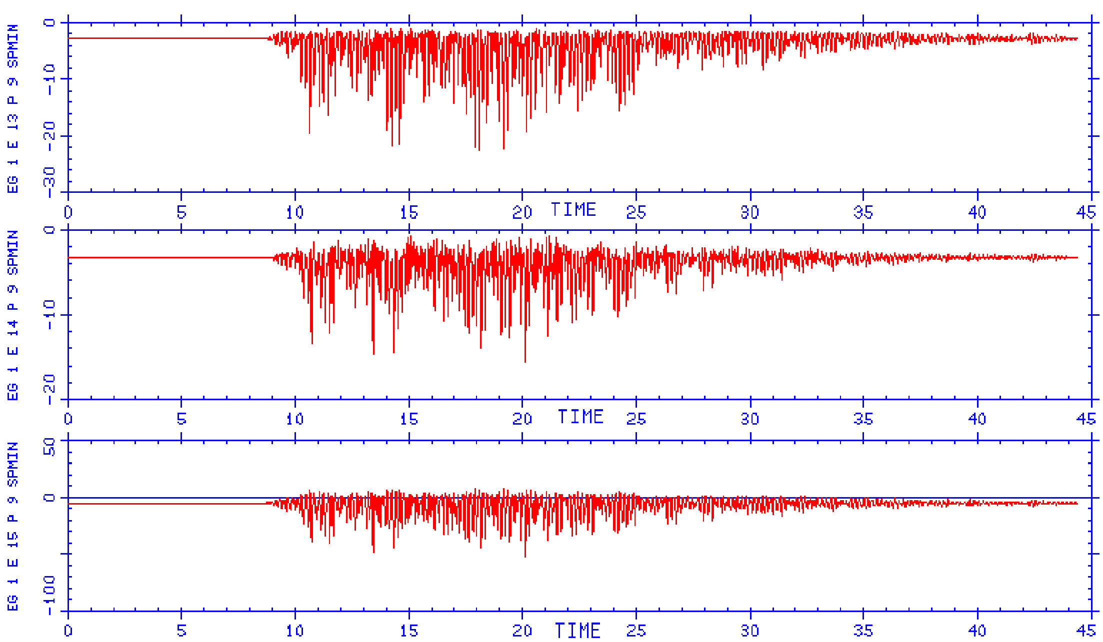


Fig.17: Variation of Minimum Principal Stress with time for selected elements

Fig.18 shows the envelope of maximum and minimum principal stresses. The maximum principal stress (tension) of the order of 40 kg/cm^2 is found to be developed near the slope starting point at downstream face near to top of the dam. The tensile stresses almost cover 80% area of the dam under the complete acceleration time history. The tensile stresses developed under Normal operating condition with earthquake are more than the allowable limits. The maximum compression of the order of 64 kg/cm^2 is also found to develop at the same location. Also the compressive stresses developed almost cover 100% area of the dam under the complete acceleration time history. The maximum compressive stress developed is not excessive and remains within allowable limit. The principal stresses are higher on upstream and downstream faces of the dam body as compared to central region.

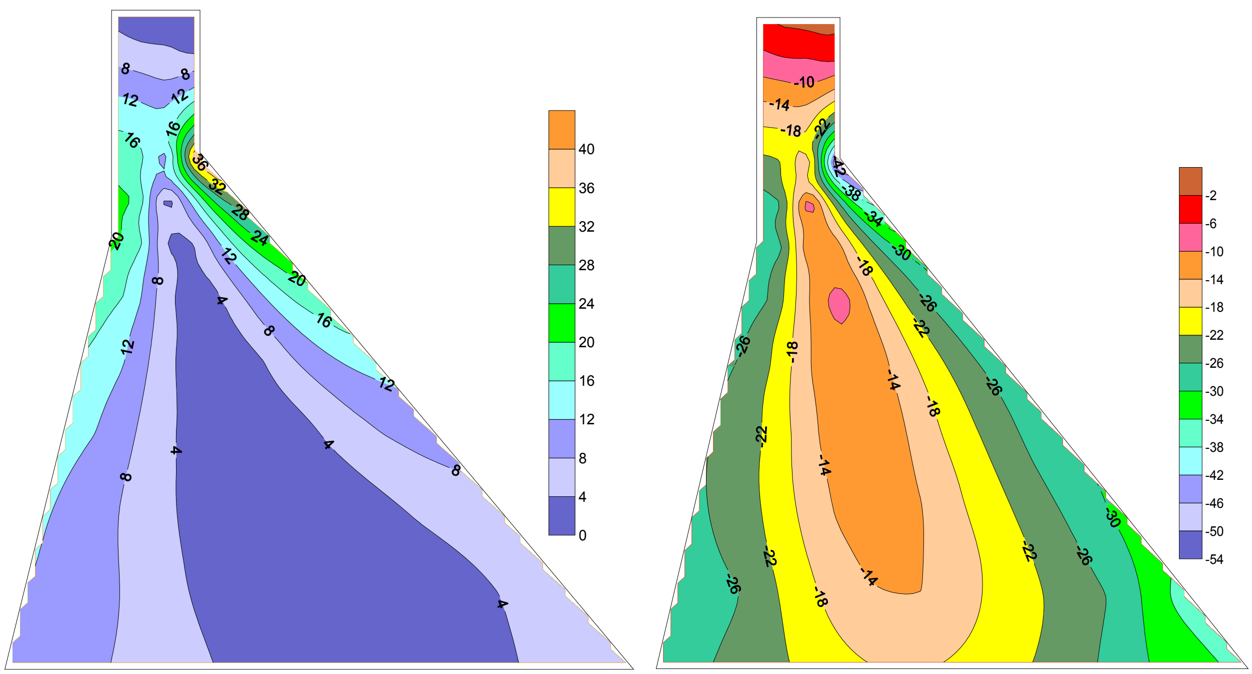


Fig. 18: Envelope of Max. and Min. Principal Stresses under Load combination E

3. DYNAMIC ANALYSIS BY MODE SUPERPOSITION METHOD

The 2D dynamic analysis has been carried out by using same mathematical model (Fig.3) as used for response spectrum analysis and direct time step integration methods. The analysis has been carried out for load combination E in two stages. At first stage, static loads and added mass haven been applied to the model and mode shape alongwith natural frequencies have been calculated. In second stage, analysis is restarted and total response has been obtained by using acceleration time history in both direction by applying trapezoidal rule for the time integration of the modal response. Modal damping varying from 5-10% has been adopted in the analysis for three modes.

RESULTS

Results are plotted in the form of displacement time history in both directions at selected nodes and element stress history in high stressed zones. The maximum and minimum principal stress envelopes for dambody by taking the principal stresses at the centroid of each element are plotted to know the overall distribution of tensile and compressive stresses. The results are presented for one Earthquake load combination E. Node displacement histories under normal operating condition with earthquake is plotted for node 1 in both horizontal and vertical directions as shown in Fig.19. At node 1, the maximum horizontal displacement nearly 3.8 cm and vertical displacement 1.8 cm are found developed.

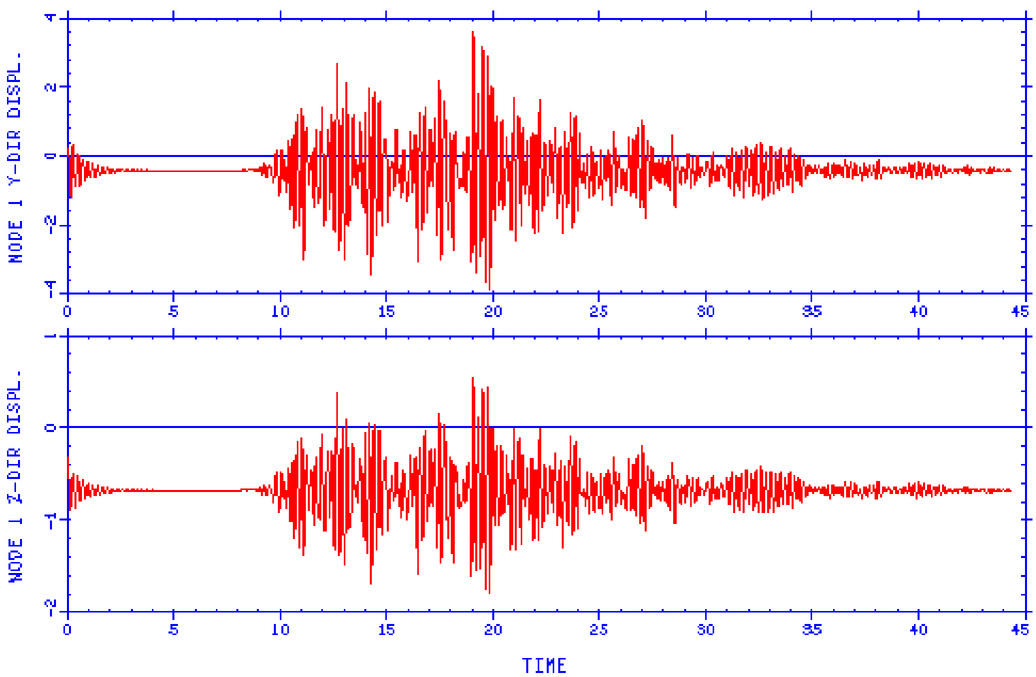


Fig.19: Variation of displacements with time at Node 1

The maximum and minimum principal stress time histories are plotted at the centroid of selected elements to know the stress variation during earthquake. Figs.20 shows the variation of maximum principal stress at the centroid of elements 16- 18.

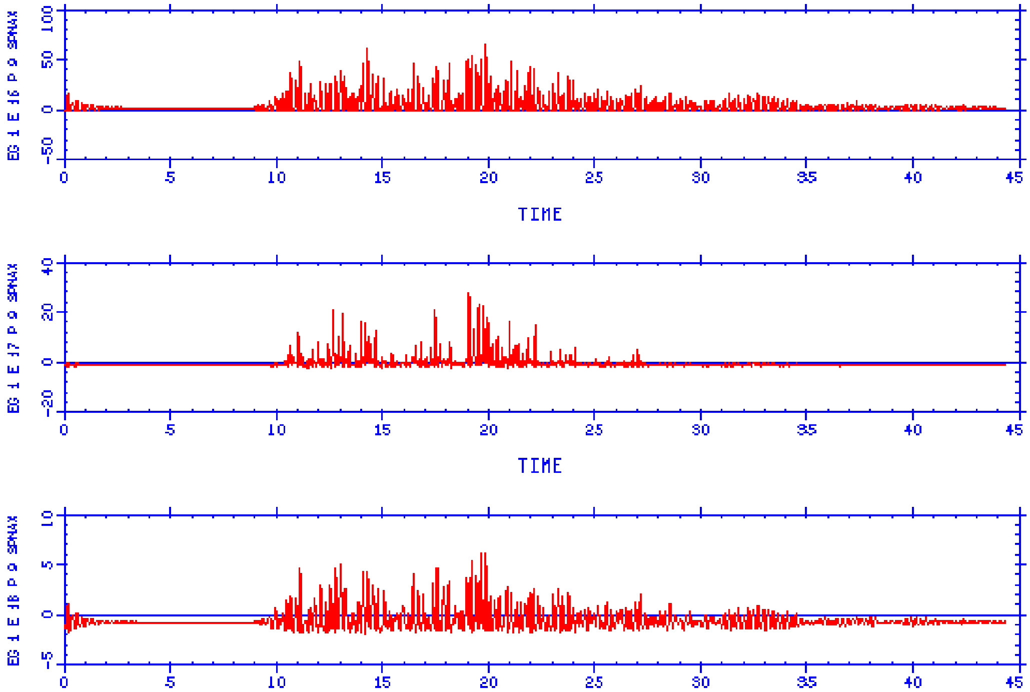


Fig.20: Variation of Maximum Principal Stress with time for selected elements

Figs.21 shows the variation of minimum principal stress at the centroid of elements 22- 24.

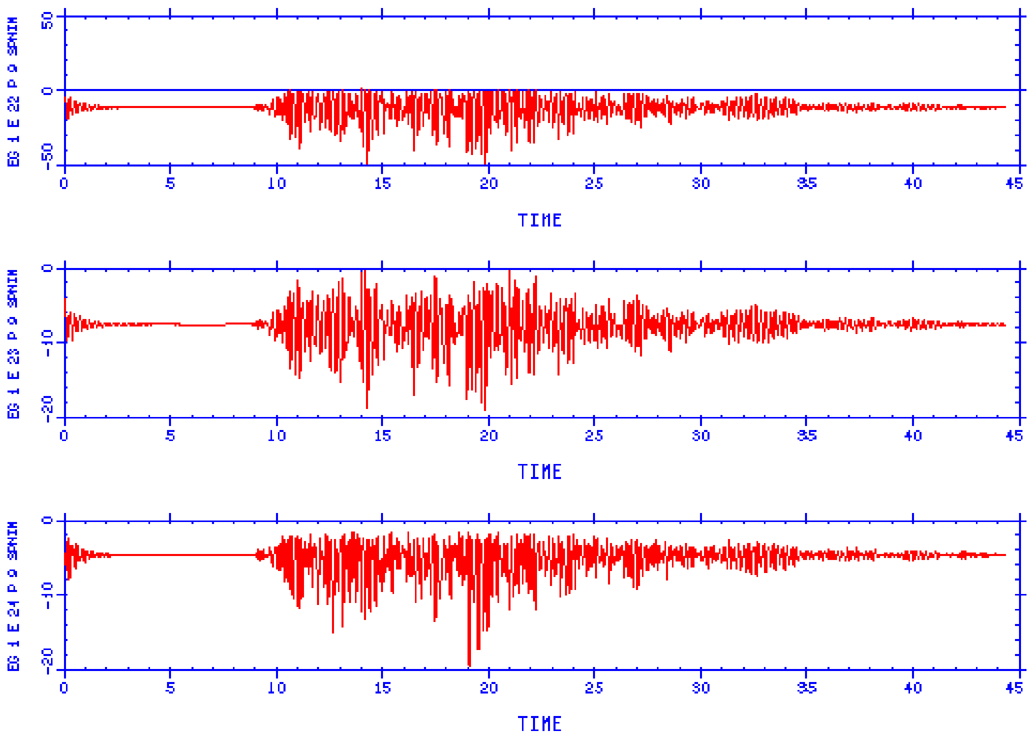


Fig.21: Variation of Minimum Principal Stress with time for selected elements

The maximum principal stress (tensile) of the order of 70 kg/cm^2 is found to be developed at the centroid of element 16 and minimum principal stress (compression) of the order of 65 kg/cm^2 is also found to be developed at the centroid of element 16. The envelope of maximum and minimum principal stresses is plotted by taking the principal stresses at centroid of each element in the dambody. Fig.22 shows the envelope of maximum and minimum principal stresses. The maximum principal stress (tension) of the order of 70 kg/cm^2 is found to be developed near the slope starting point at downstream face near to top of the dam. The tensile stresses almost cover 90% area of the dam under the complete acceleration time history. The maximum compression of the order of 65 kg/cm^2 is also found to be developed at the same location. Also the compressive stresses developed almost cover 100% area of the dam under the complete acceleration time history. The maximum compressive stress developed is not excessive and remains within allowable limit. The principal stresses are higher on upstream and downstream faces of the dam body as compared to central region.

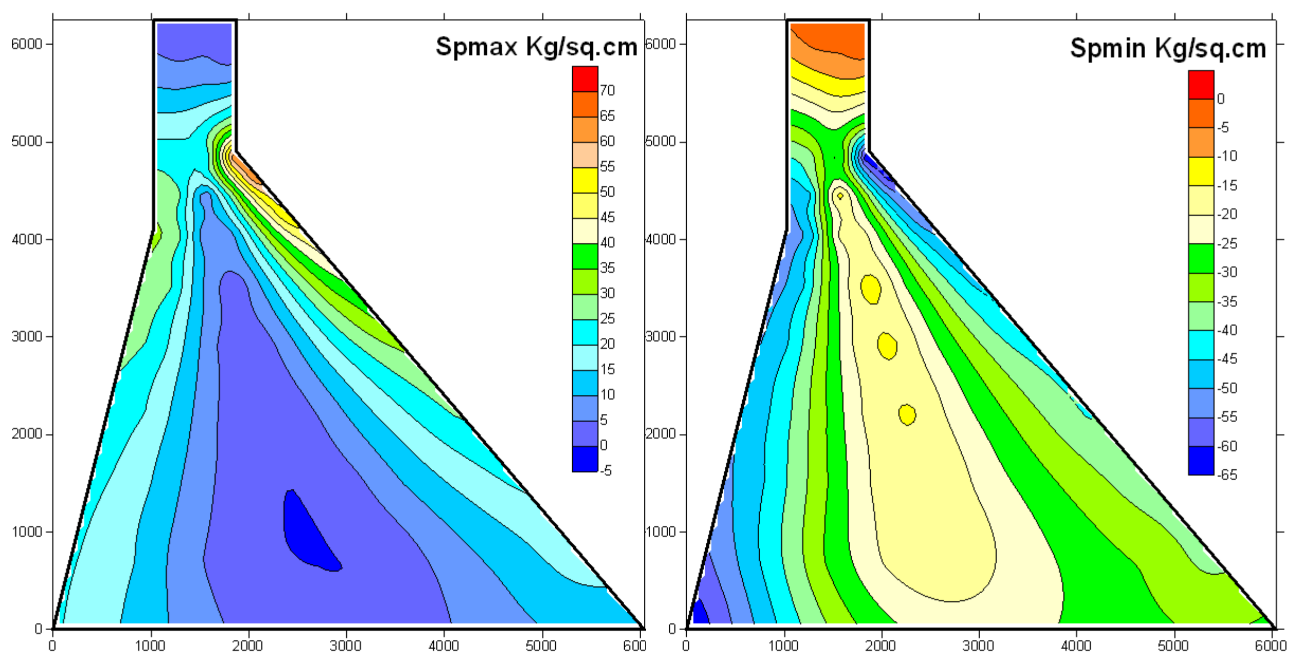


Fig. 22: Envelope of Max. and Min. Principal Stresses under Load combination E

4. DYNAMIC ANALYSIS BY FREQUENCY SUBSTRUCTURE APPROACH USING EAGD-84 FE SOFTWARE

The 2D dynamic analysis has been carried out by substructure approach using EAGD-84 finite element software exclusively developed for 2D dynamic analysis of gravity dams by Dr. AK Chopra, USA and by adopting same mathematical model excluding foundation part as used for earlier analysis and is shown in fig 23.

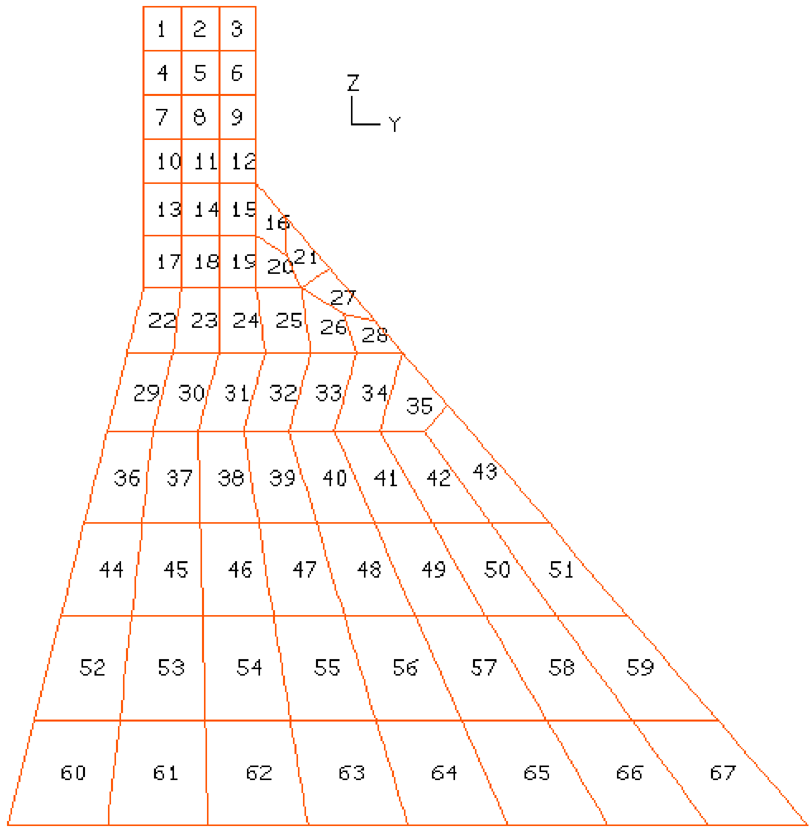
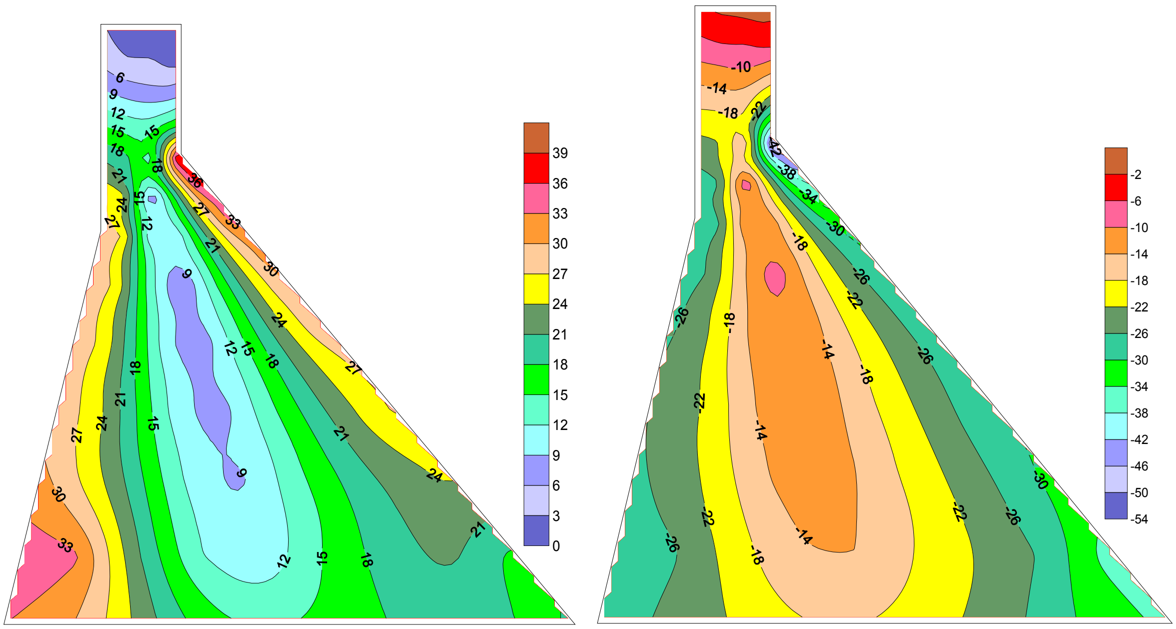


Fig.23: 2D Finite Element Model of Dambody

Same acceleration time history as used in direct time step integration and mode superposition analysis has been adopted in this analysis also. Damping has been considered as 5%. For considering dam- foundation-reservoir interaction effects, substructure approach has been applied and generated foundation frequency compliance data has been taken into analysis. Ten generalized mode shapes have been calculated and total response including static effect has been obtained for load combination E.

Results

Results have been obtained in the form of displacement time history in both directions at selected nodes and element stress history in high stressed zones. The maximum and minimum principal stress envelopes for dam body by taking the principal stresses at the centroid of each element are plotted to know the overall distribution of tensile and compressive stress distribution. The envelope of principal stresses based on stress values at the centroid of each element is shown in fig.24.



Maximum Principal stress **Minimum Principal stress**

Stress in kg/cm^2

Fig. 24: Envelope of Max. and Min. Principal Stresses under Load combination E

The maximum principal stress (tension) of the order of 39 kg/cm^2 is found to be developed near the slope starting point at downstream face near to top of the dam. The tensile stresses almost cover 90% area of the dam under the complete acceleration time history. The maximum compression of the order of 54 kg/cm^2 is also found to be developed at the same location. Also the compressive stresses developed almost cover 100% area of the dam under the complete acceleration time history.

CONCLUSIONS:

- Before taking up rehabilitation of any dam, dynamic analysis should be carried out by suitable methods using numerical modelling.
- For old dams, stability evaluation by static and pseudodynamic stress analysis using in-situ material properties is carried out as per BIS criteria by finite element method for small dams lying in seismic zone II and III.
- The pseudodynamic approach is generally used for small height dam (less than 30m) and for seismic zone II & III. This approach is also applied at the initial

stages of design to finalize the profile of dam section and in 3 Dimensional stress analysis to economies the cost.

- The detailed dynamic analysis should be carried out for dams lying in high seismic zones IV and V irrespective of height.
- A comparison of peak values of principal stresses developed under different approaches is shown in table2.
- The assessment of structural safety under Earthquake loads is very useful in deciding the type of remedial measures required for restoring structural integrity during rehabilitation of dams.

Table 2: Comparison of Principal Stress under Different Approaches

Sr. No.	Name of Approach	Maximum Principal Stress, Spmax		Minimum Principal Stress, Spmin	
		Value Kg/cm^2	Location	Value Kg/cm^2	Location
1	Pseudostatic	41.654	Heel	-35.804	Toe
2	Pseudodynamic	19.065	Starting point of U/S slope	-32.675	Toe
3	Direct Step by Step Integration Method	40	Starting point of D/S slope	-54	Starting point of D/S slope
4	Mode Superposition Method	65.156	Starting point of D/S slope	-65.232	Starting point of U/S slope
5	Response Spectrum Analysis	99.528	Starting point of D/S slope	-108.15	Starting point of U/S & D/S slopes
6	Frequency Sub-Structure EAGD-84	39	Starting point of D/S slope	-54	Starting point of D/S slope

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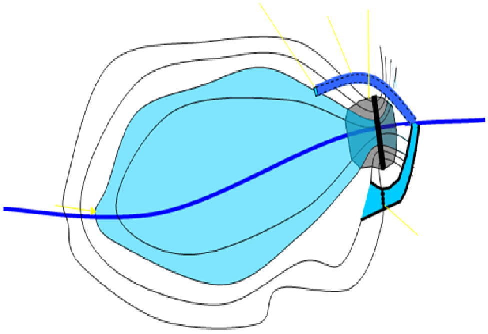
Overview of Dams

Basic Concept

A Dam is a hydraulic structure of impervious material built across a river to create a reservoir on its upstream side for impounding water for various purposes.

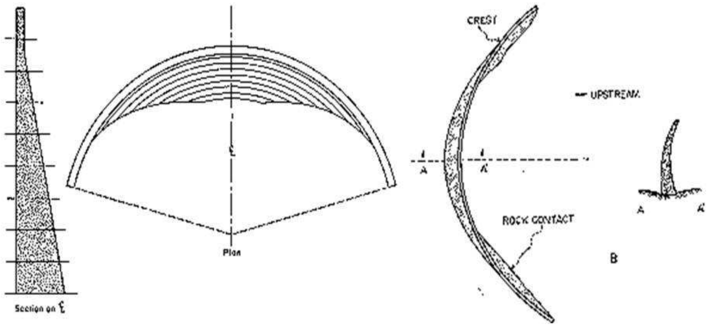


Types of Dams

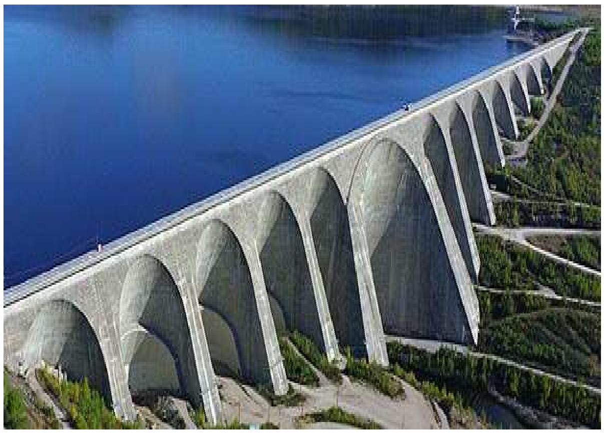
- Gravity Dams
- Arch Dams
- Buttress Dams
- Earthen Dams
- Rock fill Dams
- Steel Dams
- Timber Dams
- Rubber Dams

- **Arch Dams**

- Arch Dams - Single curvature or Double curvature



- Buttress Dams



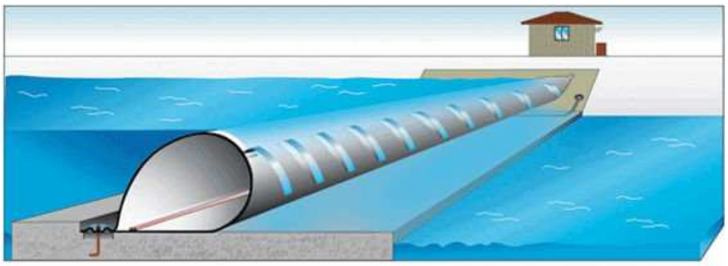
- Earthen Dams

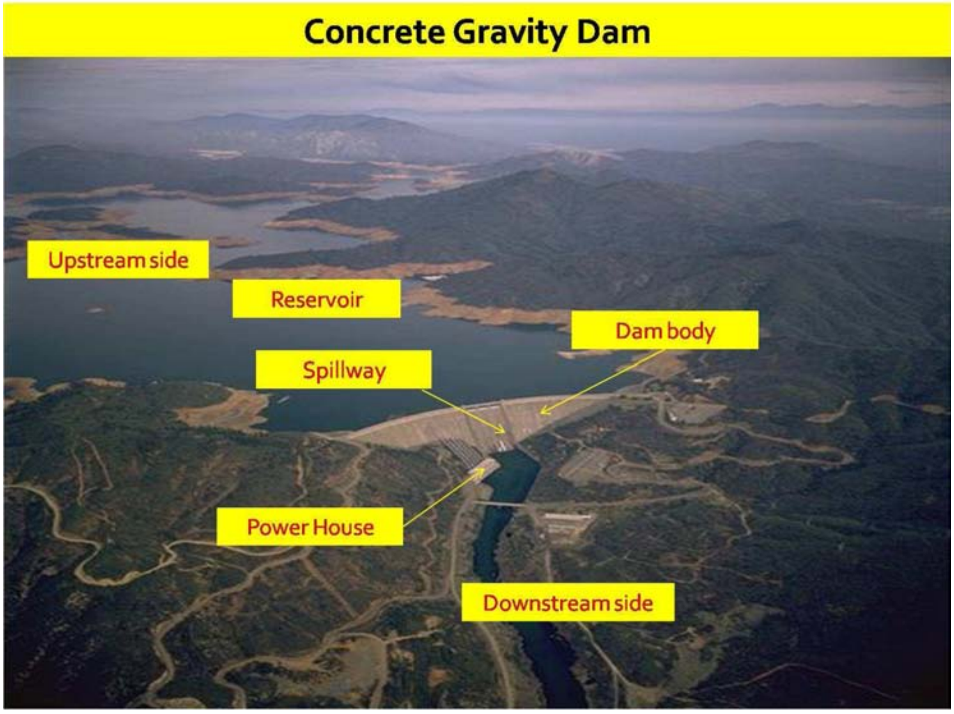
- Rock fill Dams

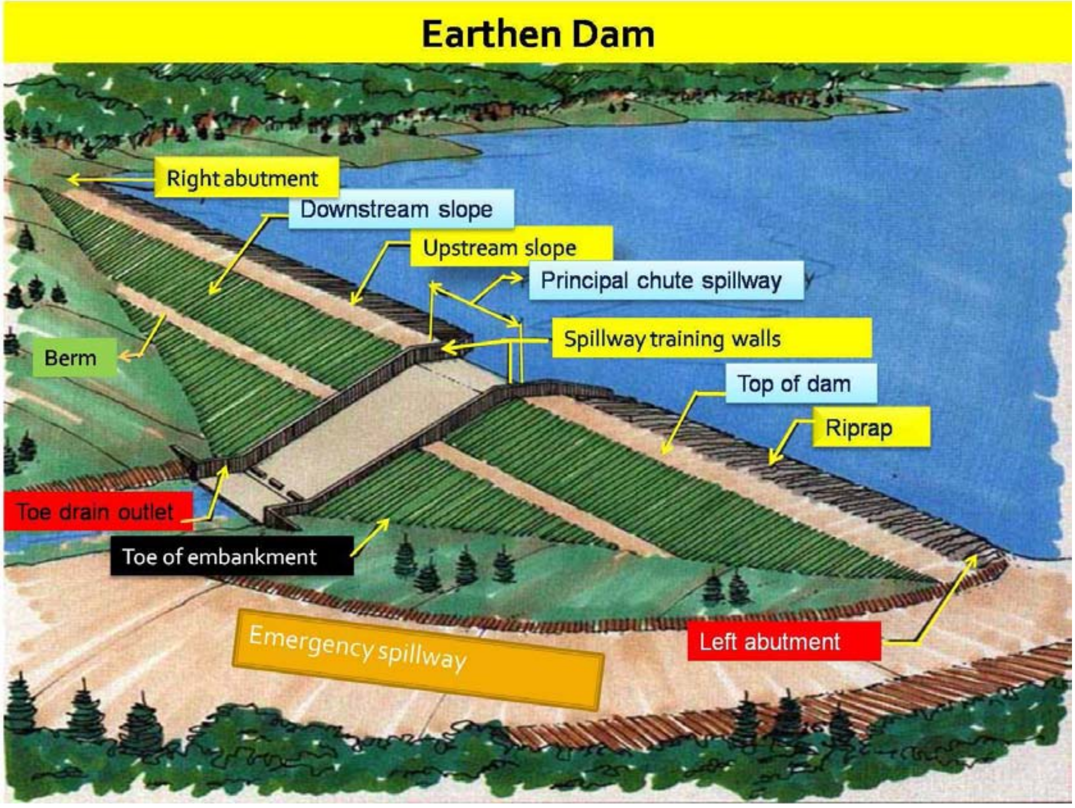
- Steel Dams

- Timber Dams

- Rubber Dams




Concrete Gravity Dam
Upstream side
Reservoir
Dam body
Spillway
Power House
Downstream side



Classification based on Purpose of Dam

- **Storage Dam**

  Storage dams are constructed to create a reservoir to store water during periods when there is huge flow in the river for utilization later during periods of low flow. Water stored in the reservoir can be used for irrigation, power generation, water supply etc. It may also be used as detention dam if regulated accordingly.

- **Detention Dam**

  Detention dam is constructed to temporarily detain all or part of the flood water in a river and to gradually release the stored water later at controlled rates so that the entire region on the downstream side of the dam is protected from possible damage due to floods. It may also be used as a storage dam.

- **Diversion Dam**

  It is constructed to divert part of or all the water from a river into a conduit or a channel. Mostly a diversion weir is constructed across the river for diverting water from a river into an irrigation canal.

- **Coffer Dam** — It is a temporary dam constructed to exclude water from a specific area. It is constructed on the upstream side of the site where a dam is to be constructed so that the site remains dry during construction.
- **Debris Dam** — It is constructed to catch and retain debris flowing in a river.

Storage v/s ROR

Storage Scheme

- Storage dams are constructed to create a reservoir to store water during monsoon periods when there is huge flow in the river
- The stored water is then utilized later on during periods of lean flow.
- Water stored in the reservoir can be used for irrigation, power generation, water supply etc.

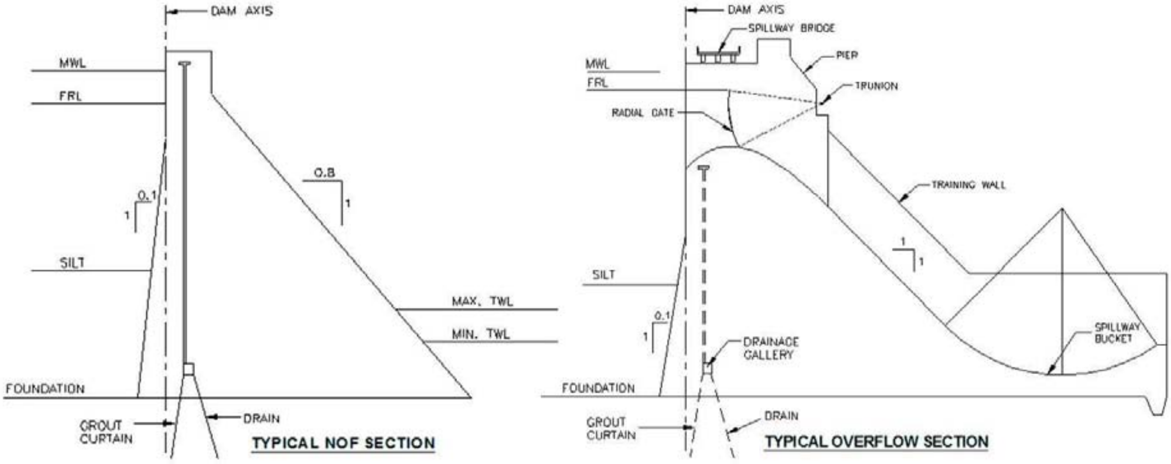
ROR Scheme

- ROR stands for Run-of-River scheme
- In ROR scheme, the water is stored during low power demand, which is utilized during high power demand.
- Run-of-River projects are constructed where no storage or a limited amount of storage is available.

Classification Based on Hydraulic Design

- Non Over Flow Dam
  - The Dam portion which does not allow to escape flood waters are called Non Over Flow Dams.
- Over Flow Dam
  - This is the dam portion which is used for escaping flood waters is called Over Flow Dam.

Overflow / Non-overflow section



Classification Based on Construction Material

- Rigid Dam
  - Made of stone, masonry, concrete, steel, or timber
- Non-rigid Dam
  - Made of earth, rock fill etc.
- Composite Dam
  - Earthen dams are provided with a stone masonry or concrete overflow (spillway) section

Design Aspects – Gravity Dams

Loads on Dams

- Dead loads
- Reservoir and Tail water loads
- Uplift pressures
- Earthquake forces
  - As per IS :1893 -1984
  - Inertial forces of dam body and hydrodynamic forces
- Silt pressure
- Ice pressure (for freezing conditions)
- Wave loads
- Thermal stresses

Dead Loads

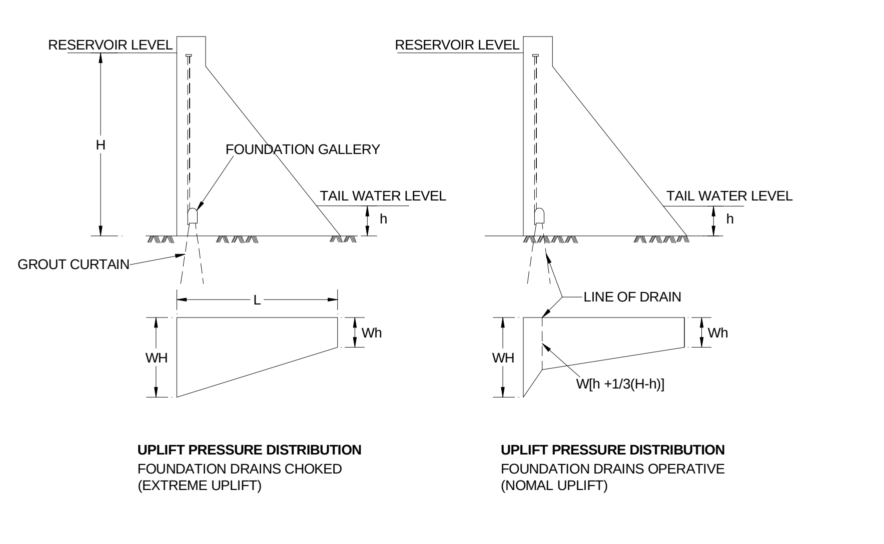
- Self weight
- Weight of other structures on dam body
  - Spillway piers
  - Gates
  - Bridge, etc.
- Unit weigths
  - Plain Concrete : 2400 kg/m^3
  - Reinforced Concrete : 2500 kg/m^3
  - Masonry : 2300 kg/m^3

Water and Silt Loads

- Hydrostatic Water Load
  - Hydrostatic Triangular Distribution taking unit wt. of water as 1000 kg/m^3
  - Weight of flowing water over spillway neglected
- Silt Load
  - Horizontal silt and water pressure is determined as if silt and water have a horizontal unit weight of 1360 kg/m^3
  - Vertical silt and water pressure is determined as if silt and water have a vertical unit weight of 1925 kg/m^3

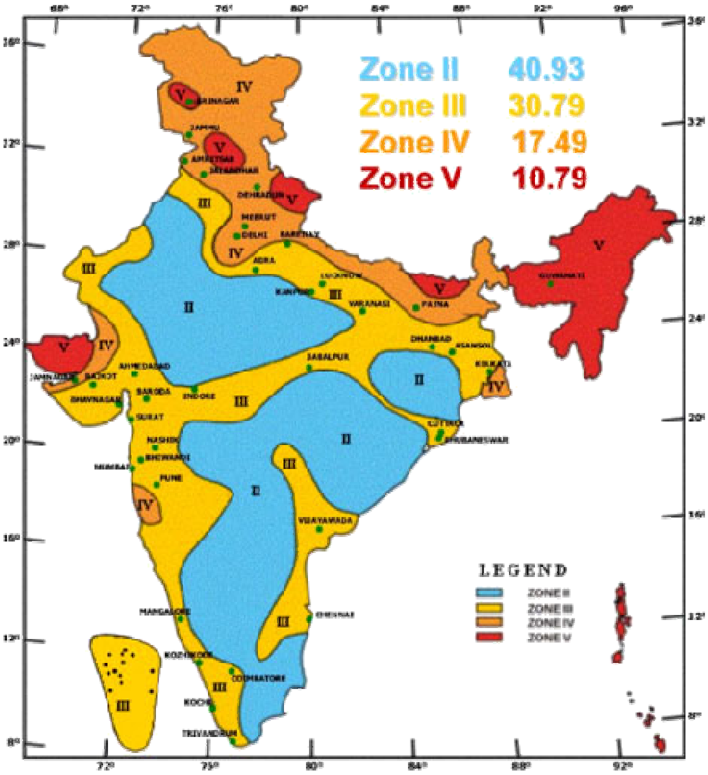
Uplift Loads

- Act over 100% of the area
- Uplift pressures are not affected by earthquakes



Earthquake Loads

- Determined as per IS :1893 -2002
- Design Seismic Coefficient is worked out as per the above code based on the location of the project on the seismic map of India



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Load Combinations

- Combination A (Construction condition)
  - Dam completed but no water in reservoir and no tail water
- Combination B (Normal operating condition)
  - Full reservoir elevation, normal dry weather tail water, normal uplift and silt.
- Combination C (Flood Discharge condition)
  - Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift and silt
- Combination D
  - Combination A with earthquake
- Combination E
  - Combination B with earthquake
- Combination F
  - Combination C but with extreme uplift (Drains inoperative)
- Combination G
  - Combination E but with extreme uplift (Drains inoperative)

Stability Analysis

- Stability analysis is done as per IS: 6512-1984 titled "Criteria for Design of Solid Gravity Dam"
- Dam shall be safe against sliding at any section
  - In the dam
  - At dam foundation interface
  - Within the foundation
- Safe unit stresses in concrete/masonry shall not be exceeded
- Dam shall be safe against overturning

Freeboard

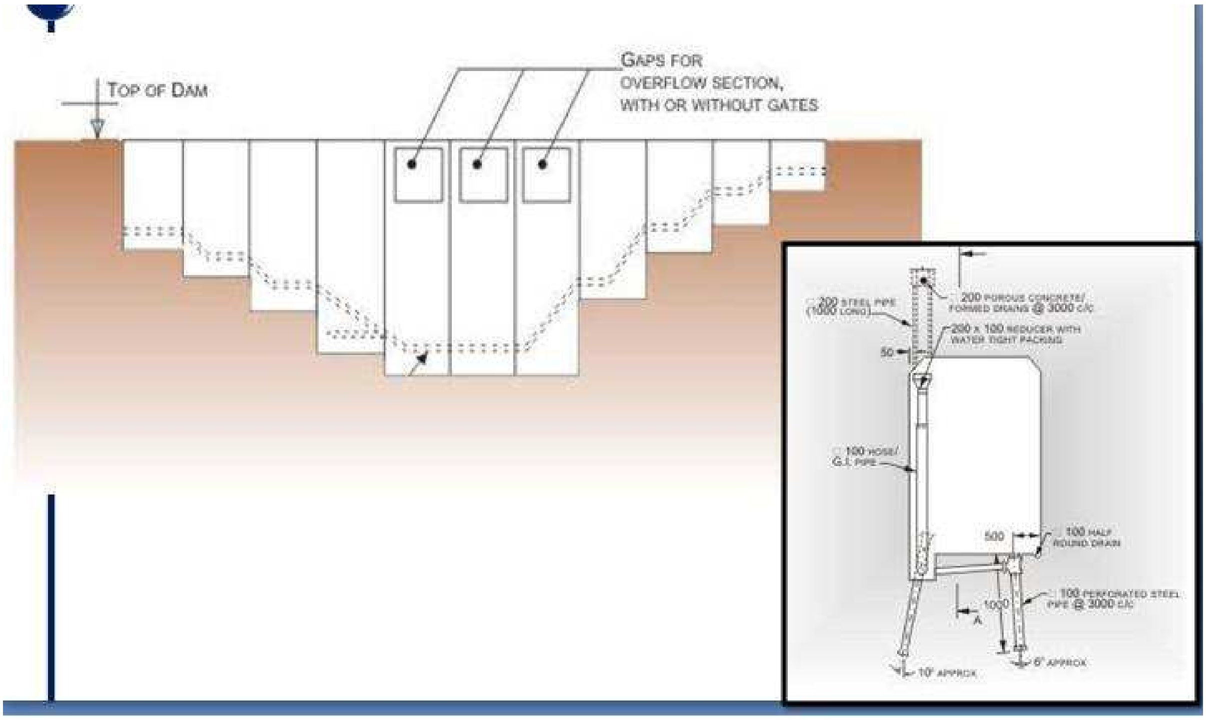
- Free Board is the vertical distance between the top of the dam and the still water level
- Free Board calculations are carried out as per IS: 6512-1984 to fix the top of dam
- Free board shall not be less than 1m above MWL

Galleries

- Provided for various purposes like curtain grouting, drainage, inspection, instrumentation etc.

- Can run in longitudinal or transverse directions
- Additional inspection / drainage galleries are generally provided after every 30 metre height
- General size 1.5m x 2.25m or 2.0m x 2.5m
- Generally provided at a distance equal to 5% of the head or 3m whichever is more from the u/s face of the dam
- Minimum concrete cover between the foundation rock and the gallery is kept about 2 to 3 m
- Sump well and pump chamber arrangements are provided for collecting and removal of seepage water

Galleries



Contraction Joints

- Longitudinal Contraction Joints – not preferred
- Transverse Contraction Joints
  - Spacing 15 to 25 m
  - Grouted / Ungrouted
  - With / without keyways

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- Water stops are installed in the joints near the upstream face to prevent passage of water through contraction joint
- Types of water stops:
  - PVC Water Stops
  - Metal Water Stops
  - Rubber Water Stops

Thermal Studies

- The heat generated due to hydration of cement raises the temperature of concrete far above the placement temperature.
- To restrict the rise in temperature measures like pre-cooling, post-cooling or a combination of both techniques is adopted
- For determining the placement temperature of concrete detailed temperature control studies are carried out
- Necessary guidelines are available in BIS 14591 – 1999 "Guidelines for temperature control of mass concrete for dams".

Temperature Control Measures

- Use of ice or refrigerated water and pre-cooled aggregate for concrete preparation
- Embedding copper pipes for flow of cool water after placing of concrete
- Use of low heat cement, Pozzolana Portland cement, Portland slag cement, etc.
- Controlling the amount of cement used
- Use of shallow lifts (limiting lifts to 1.5m)
- Keeping suitable time interval between successive lifts (normally 3 days)

Design Aspects – Embankment Dams

Types of Embankment Dams

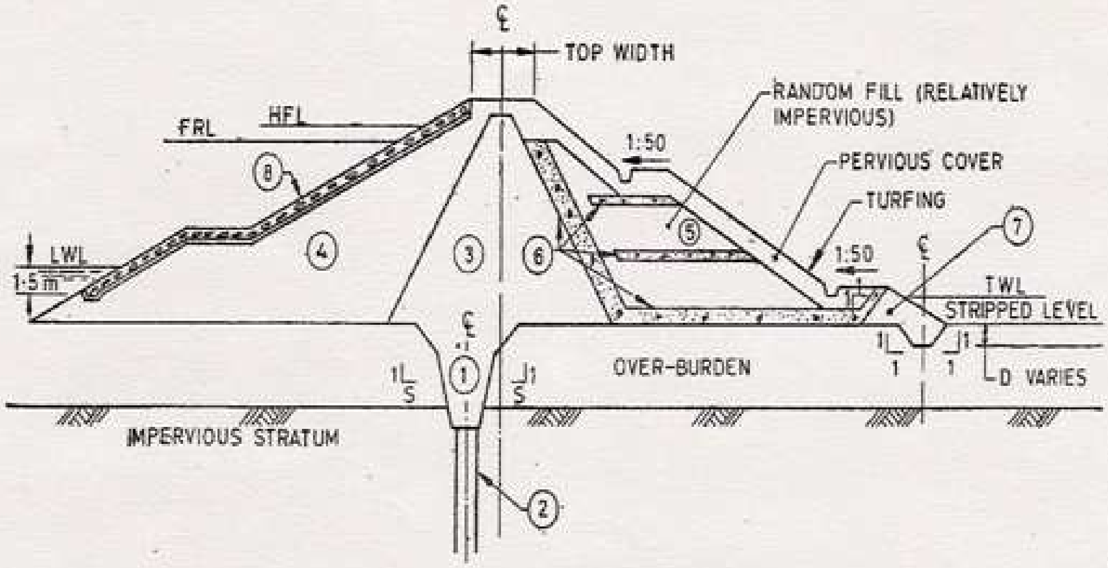
- Homogeneous Embankment
  - ✓ Dam section entirely consists of almost one type of material.
  - ✓ Section is made of low permeability material and requires flatter slopes than a zoned section
- Zoned Embankment
  - ✓ This type of embankment uses two or more types of materials, depending on their availability, utility and costs
  - ✓ There is an impervious zone called the 'core' inside the dam section

- ✓ The outer zones on both sides, called ‘shells’, should preferably be of pervious materials

- Rock-fill Dams with clay cores
  - ✓ Relies on fragmented rock material, either obtained by blasting or available as natural boulder deposits, as a major structural element
  - ✓ Substantial rock fill zones on both sides, with an impervious zone in the middle, and transition zones and /or filters in-between

- Rock-fill Dams with u/s Face Membranes
  - ✓ Relies on fragmented rock material, either obtained by blasting or available as natural boulder deposits, as a major structural element
  - ✓ Substantial rock fill zones on both sides, with an impervious membrane on upstream and downstream face

Embankment Dam- Section



1. Positive cut-off
2. Grout curtain
3. Central impervious core
4. Upstream casing
5. Downstream casing
6. Inclined and horizontal filters
7. Rock toe and toe drain
8. Riprap with filter
9. Catch water drain

NOTE — Horizontal filters at intermediate levels are sometimes also placed in the upstream casing zone where casing material is of impervious nature.

EARTH DAM WITH CENTRAL CORE AND POSITIVE CUT-OFF

Basic Design Requirements

- Safety against overtopping
  - ✓ Sufficient spillway and outlet capacity should be provided
  - ✓ Sufficient freeboard
  - ✓ Exact calculation of settlement of the embankment and of the foundation in order to determine extra freeboard to be provided
- Slope Stability
  - ✓ The slopes shall be stable under all loading conditions
  - ✓ Slopes shall be designed as per IS: 7894-1975
  - ✓ The u/s slope shall be protected against erosion by wave action and d/s slope shall be protected against erosion due to wind and rain
- Safety against internal erosion
  - ✓ The slopes shall be stable under all loading conditions
  - ✓ Slopes shall be designed as per IS: 7894-1975
  - ✓ The u/s slope shall be protected against erosion by wave action and d/s slope shall be protected against erosion due to wind and rain
- Phreatic line within downstream face
  - ✓ This results in "sloughing" or softening of the d/s face and may lead to local toe failure, which may progressively develop upwards
  - ✓ This can be safeguarded by providing a free draining zone on the d/s face or by intercepting the seepage inside the dam section by internal drainage.
- Safety against wave action
  - ✓ There should be no risk of over topping of the dam section
  - ✓ It needs estimation of the design flood and provision of adequate spillway capacity to pass that flood with required net freeboard to protect the dam crest against wave splash.

Causes of Embankment Dam Failure Worldwide

Overtopping	30%
Seepage effect, Piping and Sloughing	25%
Slope Slides	15%
Conduit Leakage	13%
Damage to slope Paving	5%
Miscellaneous	7%
Unknown	5%

List of IS Codes / Technical Literature Being Used for Design of Dams

Concrete/Masonry Dam

SL. NO	CODE NO.	TITTLE
1	4410	Glossary of terms relating to river valley projects: Part 8 Dams and dam section
2	6066	Recommendations for pressure grouting of rock foundations in river valley projects
3	6512	Criteria for design of solid gravity dams
4	8237	Code of Practice for Protection of Slope for Reservoir Embankment
5	8282	Code of practice for installation, maintenance and observation of pore pressure measuring devices in concrete and masonry dams: Part 1 Electrical resistance type cell
6	8605	Code of practice for construction of masonry in dams
7	8605	Code of practice for construction of masonry in dams
8	9296	Guidelines for inspection and maintenance of dam and appurtenant structures
9	9297	Recommendations for lighting, ventilation and other facilities inside the dam
10	10084	Criteria for design of diversion works: Part 1 Coffer dams
11	10084	Design of diversion works - Criteria : Part 2 Diversion channels and open cut or conduit in the body of dam.
12	10135	Code of practice for drainage system for gravity dams, their foundations and abutments
13	10137	Guidelines for selection of spillways and energy dissipators
23	**12966**	Code of practice for galleries and other openings in dams: Part 1 General requirements
24	**12966**	Code of practice for galleries and other openings in dams: Part 2 Structural design
25	**13073**	Installation, Maintenance and Observation of Displacement Measuring Devices in Concrete and Masonry Dams - Code of Practice - Part 1 : Deflection Measurement Using Plumb Lines

26	**13073**	Code of Practice for Installation, Maintenance and Observation of Displacement Measuring Devices for Concrete and Masonry Dams - Part 2 : Geodetic Observation - Crest Collimation
27	**13195**	Preliminary design, operation and maintenance of protection works downstream of spillways - Guidelines
28	**13551**	Criteria for structural design of spillway pier and crest

IS codes related to Earth and Rock Fill Dams

Sl. No	**IS CODE NO**	**TITTLE**
1	4999	Recommendations for grouting of pervious soils
2	6955	Code of practice for subsurface exploration for earth and rockfill dams
3	7894	Code of practice for stability analysis of earth dams
4	10160	Proforma for analysis of unit rate of earthwork used in construction of river valley projects
5	11532	Construction and maintenance of river embankments (levees) - Guidelines
6	11532	Construction and maintenance of river embankments (levees) - Guidelines
7	10751	Planning and Design of Guide Banks for Alluvial Rivers - Guidelines
8	7356	Code of Practice for Installation, Maintenance and Observation of Instruments for Pore Pressure Measurements in Earth Dams and Rockfill Dams - Part 1 : Porous Tube Piezometers
9	7356	Installation, Observation and Maintenance of Instruments for Pore Pressure Measurements in Earth and Rockfill Dams - Code of Practice - Part 2 : Twin Tube Hydraulic Piezometers

IS codes related to Barrages and Canals

SL. NO	CODE NO.	TITTLE
1	IS 4410 (Part 2):1994	Glossary of terms relating to river valley projects: Part 22 Barrages & weirs
2	IS 6966(Part 1):1989	Hydraulic design of barrages and weirs Guidelines: Part 1 Alluvial Reaches (first revision)
3	IS 7720:1991	Criteria for investigation, planning and layout of barrages and weirs (first revision)
4	IS 7349:2012	Barrages and weirs operation and maintenance - Guidelines(second revision
5	IS 11130:1984	Criteria for structural design of barrages and weirs
6	IS 11150:1993	Construction of concrete barrages - Code of practice (first revision)
7	IS 12892:1989	Safety of barrage and weir structures - Guidelines
8	IS 13578:2008	Subsurface exploration for barrrages and weirs - Code of practice
9	IS 14248:1995	Guidelines for instrumentation of barrages & weirs
10	IS 14815:2000	Design flood for river studies of barrages and weirs - Guidelines
11	IS 14955:2001	Guidelines for hydraulic model studies of barrages and weirs

Relevant Technical Literatures and Manuals

SL.NO	CODE NO.	TITTLE
1	SP 16	Design Aid to IS 456-1978
2	SP 22	Explanatory handbook on codes of Earthquake Engineering
3	SP 34	Handbook on Concrete Reinforcement & Detailing.
4	SP 55	Design aid for anchorages for spillways piers, training walls and divide walls.
5	USBR	Design of Small Dams
6	CWC	PENSTOCK MANUAL
7	CBIP	CBIP TUNNEL MANUAL
8	CBI&P	CBIP BARRAGE ON ALLUVIL FOUNDATION

& MANY MORE

Design of Concrete Gravity Dams

1.0 Introduction

A dam is an obstruction or a barrier built across a stream or a river for accumulation of water on its upstream side which is used for different purposes. Dams are constructed for deriving various benefits like irrigation, hydropower generation, flood control, domestic/industrial water supply, recreation etc.

Dams can be classified based on various criteria. As per water resources planning the dams may be classified as storage dams, diversion dams and detention dams. As per hydraulic flow conditions the dams may be classified as overflow dams (spillways) and non-overflow dams. As per materials used they can be classified as earthfill dams, rockfill dams and concrete/masonry dams.

The concrete/masonry dams can be classified further as gravity dams, buttress dams & arch dams based on their structural behavior and as conventional concrete dams & roller compacted concrete dams as per method of construction.

Conventional concrete dams are constructed by dividing the dam length into blocks of 20-25m long. Concrete placement is done by cableways, cranes, trestles etc. in lifts of 1.5-2m. The compaction of concrete is done by vibrators. Roller compacted concrete dams are constructed using same machinery/equipments as that used for embankment dams. Construction is done from abutment to abutment in lifts of 300-600 mm. Compaction of concrete is done with the help of vibratory rollers.

Masonry dams were preferred in our country earlier as they were labour intensive, provided more employment opportunities, consumed less cement and did not involve any temperature control measures. However the quality of workmanship and workers are deteriorating now. There are problems of heavy seepage through many of our existing masonry dams. For seepage control, various remedial measures are being adopted these days, viz. guniting on upstream face, upstream concrete membrane, sandwich concrete membrane, prepacked masonry construction etc. Now-a-days, there is therefore a shift in favour of concrete dams. Further, the construction of concrete dams is faster vis-à-vis masonry dams.

2.0Gravity Dam

A concrete gravity dam is a solid concrete structure so designed and shaped that its weight is sufficient to ensure stability against the effects of all imposed forces. The complete design of a concrete gravity dam includes the determination of the most efficient and economical proportions for the water impounding structure and the determination of the most suitable appurtenant structures for the control and release of the impounded water consistent with the purpose and function of the project.

2.1 General dimensions and definitions

Gravity dams may be straight or curved in plan depending upon the axis alignment. For uniformity, certain general dimensions and definitions have been established and are defined as below:

The *structural height* of a concrete gravity dam is defined as the difference in elevation between the top of the dam and the lowest point in the excavated foundation area.

The *hydraulic height* is the difference in elevation between the lowest point of the original streambed at the axis of the dam and the maximum controllable water surface.

The *length* of the dam is defined as the distance measured along the axis of the dam at the level of the top of the main body of the dam from abutment contact to abutment contact including the length of spillway if it lies wholly within the dam. However, the length of the abutment spillway located in any area especially excavated for the spillway is not included in the length of the dam.

The *volume* of a concrete dam includes the main body of the dam and all mass concrete appurtenances cot separated from the dam by construction or contraction joints.

A *plan* is an orthographic projection on a horizontal plane, showing the main features of the dam and its appurtenant works with respect to the topography. A plan should be oriented so that the direction of stream flow is towards the top or towards the right of the drawing.

A *profile* is a developed elevation of the intersection of a dam with the original ground surface, rock surface or excavation surface along the axis of the dam, the upstream face, the downstream face or other designated location.

The *axis* of the dam is a vertical reference plane usually defined by the upstream edge of the top of the dam.

A *section* is a representation of a dam as it would appear if cut by a vertical plane taken normal to the axis and is usually oriented with the reservoir to the left.

3.0 Design Considerations

3.1 Local Conditions

Collection of data on local conditions will eventually relate to the design, specifications and construction stages of a dam. Local conditions are not only needed to estimate construction costs, but may be of benefit when considering alternative designs and methods of construction. Some of these local conditions will also be used to determine the extent of the project designs, including such items as access roads, bridges and construction camps.

Data required to be collected are:

- i) Approximate distance from the nearest rail road shipping terminal to the structure site
- ii) Local freight or, trucking facilities and rates
- iii) Availability of housing and other facilities in the nearest towns
- iv) Availability or, accessibility of public facilities or, utilities such as water supply, sewage disposal, electric power for construction purposes, telephone services etc.
- v) Local labour pool and general occupational fields existing in the area

3.2 Maps and Photographs

Maps and photographs are of prime importance in the planning and design of a concrete dam and its appurtenant works. From these data an evaluation of alternative layouts can be made preparatory to determining the final location of the dam, the type and location of its appurtenant works and the need for restoration and/or development of the area.

Data to be collected are:

- i) A general map locating the area within the State, together with district and township lines.
- ii) Map showing existing towns, highways, roads, railways and shipping points
- iii) A vicinity map showing the following details:

a) The structure site and alternate sites
b) Public utilities
c) Stream gauging stations
d) Existing man-made works affected by the proposed development
e) Locations of potential construction access roads, sites for Government camp, permanent housing area and sites for Contractor's camp and construction facilities
f) Sources of natural construction materials

iv) Site topography covering the area of dam, spillway, outlet works, diversion works, construction access and other facilities

3.3 Hydrologic Data

In order to determine the potential of a site for storing water, generating power or, other beneficial use, a thorough study of hydrologic conditions is required.

The hydrologic data required include the following:

i) Stream flow records, including daily discharges, monthly volumes and momentary peaks
ii) Stream flow and reservoir yield
iii) Project water requirements, including allowances for irrigation and power, conveyance losses, reuse of return flows, dead storage requirements for power, recreation, fish, wildlife etc.
iv) Flood studies including inflow design floods and construction period floods
v) Sedimentation and water quality studies including sediment measurements, analysis of dissolved solids etc.
vi) Data on ground water tables in the vicinity of the reservoir and dam site.
vii) Water rights, including inter-state and international treaty effects.

3.4 Reservoir Capacity and Operation

Dam designs and reservoir operating criteria are related to reservoir capacity and anticipated reservoir operations. The loads and loading combinations to be applied to the dam are derived from the several standard reservoir water surface elevations. Reservoir capacity and reservoir operations are used to properly size spillway and outlet works.

Reservoir design data required for the design of dam and its appurtenant works are:

1) Area – Capacity curves and/or tables
2) Topographic map of reservoir area
3) Geological information pertinent to reservoir tightness
4) Reservoir storage allocations and corresponding elevation
5) Required outlet capacities of respective reservoir water surfaces and sill elevations etc.
6) Annual reservoir operation tables or charts
7) Method of reservoir operations for flood control, maximum permissible releases consistent with safe channel capacity
8) Anticipated wave action, wind velocity, fetch etc.
9) Anticipated occurrence and amount of ice, floating debris etc.
10) Physical, economic or, legal limitations to maximum reservoir water surface.

3.5 Climate Effects

Climate conditions at a site affect the design and construction of the dam. Measures to be employed during construction to prevent cracking of concrete are related to ambient temperatures at site.

The data on climate conditions considered as part of design data are :

1) Records of mean monthly maximum, mean monthly minimum and mean monthly air temperatures at site
2) Daily maximum and minimum air temperatures
3) Daily maximum and minimum river water temperatures
4) Amount of annual variance in rainfall and snowfall
5) Wind velocities and prevailing direction

3.6 Construction Materials

Construction of a gravity dam requires availability of suitable aggregates in sufficient quantity. Aggregates are usually processed from natural deposits of sand, gravel and cobbles or, may be crushed from suitable rock.

Data required on construction materials are:

1) Sources of aggregate
2) Water for construction purposes
3) Results of sampling, testing and analysis of construction materials
4) Information on potential sources of soils, sand and gravel to be used for backfill, road surface, protection of slope etc.

3.7 Site Selection

The two most important considerations in selecting a dam site are:

1) the site must be adequate to support the dam and appurtenant structures
2) the area upstream of site must be suitable for a reservoir

The following factors should be considered in selecting the best site out of several alternatives:

1)	Topography :	A narrow site to minimize amount of material in the dam, thus reducing its cost
2)	Geology :	Dam foundation should be relatively free of major faults and shears
3)	Appurtenant : Structures	Selecting a site which will better accommodate the appurtenant structures to reduce overall cost
4)	Local conditions	Sites requiring relocation of existing facilities like roads, railway, power lines, canals increase overall cost.
5)	Access:	Difficult access may require construction of expensive roads, thus increasing the cost.

3.8 Configuration of Dam

A gravity dam is a concrete structure designed so that its weight and thickness ensure stability against all the imposed forces.

Non-overflow section

The downstream face is usually a uniform slope, which, if extended, would intersect the vertical upstream face at or near the maximum reservoir level. The upper portion of the dam must be thick enough to resist the shock of floating objects and to provide space for a roadway. The upstream face will normally be vertical. However, the thickness in the lower part may be increased by an upstream batter, if required. The base width (thickness) is an important factor in resisting the sliding and may dictate the d/s slope.

Overflow section

Spillway may be located either in the abutment or in the dam. Section of spillway is similar to NOF section but modified at top to accommodate the crest and at the toe to accommodate the energy dissipater. The elevation of crest and its shape is determined by hydraulic requirements.

3.9 Foundation Investigation

The purpose of a foundation investigation is to provide data necessary to properly evaluate a foundation. Basic data to be obtained during appraisal investigation, with refinement continuing until construction is complete are:

1) Dip, strike, thickness, composition and extent of faults and shears
2) Depth of overburden
3) Depth of weathering
4) Joint orientation and continuity
5) Tests of foundation rock viz.

Physical Properties Tests

- Compressive Strength
- Elastic modulus
- Poisson's ratio
- Bulk specific gravity
- Porosity
- Absorption

Shear Tests

- Direct shear
- Triaxial Shear
- Sliding friction

Other Tests

- Solubility
- Petrographic Analysis

3.10 Construction Aspects

Construction aspects that should be considered in the design stage are:

- Adequacy of area for construction plant and equipment
- Permanent access roads to facilitate construction activities
- Length of construction seasons
- Construction schedule developed by CPM, PERT etc.

4.0 Design Criteria - Stability Analysis

4.1 Requirements for Stability

The following are the basic requirements of stability for a gravity dam:

i) Dam shall be safe against sliding at any section in the dam/dam foundation interface/within the foundation.
ii) Safe unit stresses in concrete/masonry shall not be exceeded.
iii) Dam shall be safe against overturning

4.2 Basic Assumptions

For stability analysis the following assumptions are made:

i) That the dam is composed of individual transverse vertical elements each of which carries its load to the foundation without transfer of load from or to adjacent elements.
ii) That the vertical stress varies linearly from upstream face to downstream face on any horizontal section.

4.3 Load Combinations

The following loading combinations have been prescribed by IS:6512 for stability analysis:

Combination A (Construction condition)	Dam completed but no water in reservoir and no tail water
Combination B (Normal operating condition)	Reservoir at full reservoir elevation, normal dry weather tail water, normal uplift and silt.
Combination C (Flood Discharge condition)	Reservoir at maximum flood pool elevation, all gates open, tail water at flood elevation, normal uplift and silt
Combination D	Combination A with earthquake
Combination E	Combination B with earthquake
Combination F	Combination C with extreme uplift (Drains inoperative)
Combination G	Combination E with extreme uplift (Drains inoperative)

4.4 Forces acting on a gravity dam

The various external forces considered to be acting on a gravity dam are:
1. Dead loads
2. Reservoir and Tail water loads
3. Uplift pressures

4. Earthquake forces
5. Silt pressures
6. Ice pressure
7. Wave pressure
8. Thermal loads, if applicable

Dead Loads

Self weight of dam and weight of appurtenant works such as, spillway piers, gates, hoists, spillway bridge etc. are considered for computing the dead loads. The unit weights adopted in preliminary designs are 2.3 t/m^3 for masonry and 2.4 t/m^3 for concrete.

Reservoir & Tail Water Loads

The load due to reservoir water is calculated using hydrostatic triangular pressure distribution and taking unit weight of water as 1 t/m^3. The weight of flowing water over spillway is neglected. The load due to tail water is calculated by taking tail water pressure corresponding to tail water elevation in case of NOF sections and for a reduced value in case of OF sections depending on the E.D.A.

Silt Pressures

The deposited silt may be taken as equivalent to a fluid exerting a force with a unit weight 0.36 t/m^3 in horizontal direction and 0.925 t/m^3 in vertical direction. Thus the horizontal silt and water pressure is determined as if silt and water have a horizontal unit weight of 1.36 t/m^3 and vertical silt and water pressure is determined as if silt and water have a vertical unit weight of 1.925 t/m^3.

Uplift Pressures

Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through the body of the dam exert an uplift pressure on the base of the dam. It is assumed to act over 100% of the area of base and assumed to vary linearly from upstream to downstream corresponding to water heads. However, in case drainage galleries are provided, there is a relief of uplift pressure at the line of drain equal to two-thirds the difference of the hydrostatic heads at upstream and downstream. It is assumed that uplift pressures are not affected by earthquakes.

Earthquake forces (Ref: IS 1893 – 1984)

Earthquake forces are determined as per IS:1893. Design seismic coefficients in horizontal and vertical direction are worked out as per the above code based on the location of the project on the seismic map of India.

Design horizontal seismic coefficient (α_h)

1. By Seismic Coefficient Method (for dams upto 100 m high)

$$\alpha_h = \beta I \alpha_o$$

where,

α_h	=	Design Horizontal Seismic Coefficient
α_o	=	Basic Horizontal Seismic Coefficient (from Table 2, IS:1893)
I	=	Importance factor of the structure (3.0 for dams)
β	=	Coefficient depending upon soil foundation system (1.0 for dams)

2. By Response Spectrum Method (For dams higher than 100 m)

$$\alpha_h = \beta I F_o S_a/g$$

where,

F_o	=	Seismic Zone factor for average acceleration spectra (from Table 2, IS:1893)
S_a/g	=	Average Acceleration coefficient read from Fig. 2, IS:1893 for appropriate natural period and damping.

Design vertical seismic coefficient

The design vertical seismic coefficient is taken as half the design horizontal seismic coefficient.

Inertia forces on the dam

A triangular distribution of acceleration is prescribed for determining inertia forces on the dam. For horizontal inertia forces 1.5 times the design horizontal seismic coefficient is assumed at the top of the dam varying to zero at the base. For vertical inertia forces also 1.5 times the design vertical seismic coefficient is assumed at the top of the dam varying to zero at the base.

Hydrodynamic Pressure on the dam

The basic work in this regard has been done by Westergaard. Subsequently Zanger in 1952 presented formulas for computing hydrodynamic pressures exerted on vertical and sloping faces by horizontal earthquake effects. Based on Zanger's work, IS:1893 gives the procedure for calculating hydrodynamic pressure on the dam.

Effects of Horizontal Earthquake Acceleration

Due to horizontal acceleration of the foundation and dam there is an instantaneous hydrodynamic pressure (or suction) exerted against the dam in addition to hydrostatic forces. The direction of hydrodynamic force is opposite to the direction of earthquake acceleration. Based on the assumption that water is incompressible, the hydrodynamic pressure at depth y below the reservoir surface shall be determined as follows :

$$p = C_s \alpha_h wh$$

where,

p = hydrodynamic pressure in kg/m² at depth y,
C_s = coefficient which varies with shape and depth
α_h = design horizontal seismic coefficient
w = unit weight of water in kg/m³, and
h = depth of reservoir in m.

The approximate values of Cs for dams with vertical or constant upstream slopes may be obtained as follows :

$$Cs = \frac{Cm}{2}\left\{\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right\}$$

where,

Cm = maximum value of Cs obtained from Fig.10, IS:1893
y = depth below surface in m, and
h = depth of reservoir in m

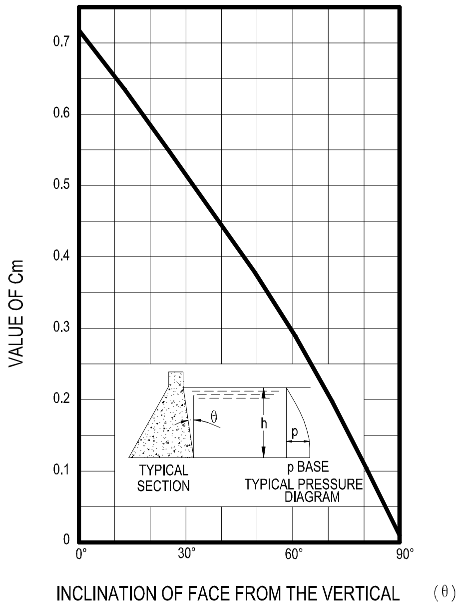


Fig. 1 : Maximum Values of Pressure Coefficient (C_m) for Constant Sloping Faces

The approximate values of total horizontal shear and moment about the center of gravity of a section due to hydrodynamic pressure are given by the relations :

$$V_h = 0.726\,py$$
$$M_h = 0.299\,py^2$$

where

V_h = hydroldynamic shear in kg/m at any depth, and
M_h = moment in kg.m/m due to hydrodynamic force at any depth y.

Inertia forces on the dam

1. Seismic coefficient method (For dams upto 100 m high)

A triangular distribution of acceleration is prescribed for determining inertia forces on the dam. For horizontal inertia forces 1.5 times the design horizontal seismic coefficient is assumed at the top of the dam varying to zero at the base. For vertical inertia forces also 1.5 times the design vertical seismic coefficient is assumed at the top of the dam varying to zero at the base. The design vertical seismic coefficient is taken as half the design horizontal seismic coefficient.

2. Response Spectrum Method (For dams more than 100 m high)

The fundamental period of vibration is calculated as under :

$T = 5.55\, H^2/B\, (W_m/g/E_s)^{0.5}$

where,

T	=	Fundamental period of vibration of the dam in secs.
H	=	Height of the dam in meters
B	=	Base width of the dam in meters
W_m	=	Unit weight of material of dam in kg/m≥
g	=	Acceleration due to gravity in m/sec″
E_s	=	Modulus of Elasticity of material in kg/m″

Damping used for concrete dams = 5%

The design horizontal seismic coefficient is calculated using the above time period and for a damping of 5% from the average acceleration spectra given in IS:1893.

The basic shear and moment due to the horizontal inertia forces is obtained by the formulae given below:

Base shear	=	V_B	=	$0.6\, W.\, \alpha_h$
Base Moment	=	M_B	=	$0.9\, W.h_{CG}\, \alpha_h$

where,

W	=	Self weight of dam in kg
h_{CG}	=	Height of C.G. of dam above base in meters
α_h	=	Design Horizontal Seismic Coefficient

The vertical inertia forces are calculated using the same distribution as outlined in seismic coefficient method but using the seismic coefficient as calculated above.

5.0 Check for permissible stresses

Check for Compressive Stresses

1. Concrete

- Strength of concrete after 1 year should be 4 times the maximum computed stress in the dam or 14 N/mm″ whichever is more.
- Allowable working stress in any part of the structure shall not exceed 7 N/mm″.

2. Masonry

- Strength of masonry after 1 year should be 5 times the maximum computed strength in the dam or 12.5 N/mm″ which is more.
- Compressive strength of masonry can be determined by compressing to failure 75 cm cubes (or 45 cm x 90 cm cylinders) cored out of the structures.

Check for Tensile Stresses

Nominal tensile stresses permitted in concrete/masonry gravity dams (as per is: 6512)

Load Combination	Permissible Tensile Stress	
	Concrete dams	Masonry dams
A	Small Tension	Small Tension
B	No Tension	No Tension
C	0.01 fc	0.005 fc
D	Small Tension	Small Tension
E	0.02 fc	0.01 fc
F	0.02 fc	0.01 fc
G	0.04 fc	0.02 fc

where, fc = Cube Compressive Strength of Concrete/Masonry

6.0 Check for Sliding

The dam should be safe against sliding across any plane/combination of planes passing through:

- The body of the dam
- Dam foundation interface
- Foundation

The partial factors of safety against sliding as per IS:6512 are given below:

Loading Condition	$F\phi$	Fc		
		For dams and the Contact plane with Foundation	For foundation	
			Thoroughly investigated	Others
A,B,C	1.5	3.6	4.0	4.5
D,E	1.2	2.4	2.7	3.0
F,G	1.0	1.2	1.35	1.5

The factor of safety against sliding shall be computed from the following equation and it shall not be less than 1.0.

$$F = \frac{\dfrac{(W-U)\tan\phi}{F\phi} + \dfrac{c.A}{Fc}}{P}$$

Where,

F	=	factor of safety against sliding
W	=	total mass of the dam
U	=	total uplift force
tan ϕ	=	coefficient of internal friction of the material
c	=	cohesion of the material at the plane considered
A	=	area under consideration for cohesion
Fϕ	=	partial factor of safety in respect of friction
Fc	=	partial factor of safety in respect of cohesion, and
P	=	total horizontal force

7.0 Freeboard

Free Board is the vertical distance between the top of the dam and the still water level. Freeboard is computed from the following two considerations:

Wave height considerations

It is equal to wind set up plus 1 1/3 times the wave height above FRL or above MWL (corresponding to design flood) whichever gives higher dam top level. A minimum freeboard of 1m above MWL corresponding to design flood shall be available. If design flood is not equal to PMF then the top of dam should be at least equal to MWL corresponding to PMF. At least 1m high solid parapet is to be provided, not withstanding the above requirements.

Wind velocity generally assumed as below in absence of meteorological data:

For FRL condition	-	120 km/hr
For MWL condition	-	80 km/hr

T. Saville's method as given in IS:6512-1984 is used for calculating the wave height/freeboard.

Operation considerations

IS:11223 specifies the following:

The freeboard as specified in IS: 6512 shall be available at FRL and MWL corresponding to all bays operative condition. For gated spillways a contingency of 10% of gates (min. one gate) being inoperative is considered as an emergency. A reduced freeboard may be acceptable under the emergency condition. The dam shall not be allowed to overtop in any case.

Design of Spillways

1.0 GENERAL

Spillway is a safety valve provided in the dam to dispose of surplus flood waters from a reservoir after it has been filled to its maximum capacity i.e. Full Reservoir Level.

The importance of safe spillways needs no over-emphasis as many failures of dams have been caused by improper design of spillways or spillways of insufficient capacity especially in case of earth and rockfill dams which are susceptible to breaching, if overtopped. Concrete/Masonry dams can withstand moderate overtopping but this should be avoided.

Further, the spillway must be hydraulically and structurally adequate and must be so located that the overflowing discharges do not erode or undermine the downstream toe of the dam.

2.0 SELECTION OF DESIGN FLOOD

The spillway design flood is generally determined by transposing great storms which have been known to occur in the region over the drainage area. The resulting flood hydrographs are then determined by rational methods. In determining the discharge capacity consideration should be given to all possible contingencies, e.g. one or more gates being inoperative.

IS : 11223 – 1985 provides guidelines for fixing spillway capacity. Inflow design flood for the safety of the dam is guided by the following criterion:

The dams may be classified according to size by using the hydraulic head and the gross storage behind the dam as given below. The overall size classification for the dam would be the greater of that indicated by either of the following two parameters:

Classification	**Gross Storage**	**Hydraulic Head**
Small	0.5 to 10 million m^3	7.5 to 12 m
Intermediate	10 to 60 million m^3	12 to 30m
Large	Above 60 million m^3	Above 30 m

The inflow design flood for safety of the dam would be as follows:

Size of dam	**Inflow Design Flood for safety of dam**
Small	100 years flood
Intermediate	Standard Project Flood (SPF)
Large	Probable Maximum Flood (PMF)

Floods of larger or smaller magnitude may be used if the hazard involved in the eventuality of a failure is particularly high or low. The relevant parameters to be considered in judging the hazard in addition to the size would be:

i) Distance and location of human habitations on the downstream after considering the likely future developments

ii) Maximum hydraulic capacity of the downstream channel

For more important projects dam break studies are done as an aid to the judgement in deciding whether PMF needs to be used. Where the studies or judgement indicate an imminent danger to present or future human settlements, the PMF should be used as design flood.

2.1 Standard Project Flood (SPF)

It is the flood that may be expected from the most severe combination of hydrological and meteorological factors that are considered reasonably characteristic of the region and is computed by using the Standard Project Storm (SPS). While transposition of storms from outside the basin is permissible, very rare storms which are not characteristic of the region concerned are excluded in arriving at the SPS rainfall of the basin.

2.2 Probable Maximum Flood (PMF)

It is the flood that may be expected from the most severe combination of critical meteorological and hydrological condition that are reasonably possible in the region and is computed by using the Probable Maximum Storm (PMS) which is an estimate of the physical upper limit to maximum precipitation for the basin. This is obtained from the transposition studies of the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions.

3.0 FLOOD ROUTING

The process of computing the reservoir storage volumes and outflow rates corresponding to a particular hydrograph of inflow is known as flood routing. It is used for arriving at the MWL for the project. The relationship governing the computation is essentially simple – over any interval of time the volume of inflow must equal the volume of outflow plus the change in storage during the period. If the reservoir is rising, there will be increase in storage and change in storage will be positive, if the reservoir is falling, there will be decrease in storage and the change in storage will be negative.

For an interval of time Δt, the relationship can be expressed by the following expression:

$$I.\Delta t - O.\Delta t = \Delta S$$

Where,

I = Average rate of inflow during time equal to Δt

O = Average rate of outflow during time equal to Δt

ΔS = Storage accumulated during time equal to Δt

The following three curves are required for carrying out the computations:

a) The inflow flood hydrograph

b) The reservoir capacity curve

c) The rating curve showing the total rate of outflow through outlets and over the spillway against various reservoir elevations

Flood routing in gated spillways is generally carried out assuming the flood to impinge at FRL assuming inflow equal outflow to at that level. For ungated spillways this would correspond to the spillway crest or a little above this.

The methods generally adopted for flood routing studies are:

i) Trial and Error Method
ii) Modified Puls Method

3.1 Trial and Error Method

This method arranges the basic routing equation as follows:

$$\frac{I_1 + I_2}{2}.\Delta t = \frac{O_1 + O_2}{2}.\Delta t + (S_2 - S_1)$$

The procedure involves assuming a particular level in the reservoir at the time interval Δt, and computing the values on the right side of the above equation. The computed value on the right side of the equation, corresponding to the assumed reservoir level, is compared with the known value on the left side of the equation. If the two values tally, then the assumed reservoir level at the end of the time interval is OK; otherwise a new reservoir level is assumed and the process is repeated till the required matching is obtained.

This method gives quite reliable results, provided the chosen time interval is sufficiently small, so that the mean of the outflow rates at the start and the end of the interval may be taken as the average throughout the interval.

3.2 Modified Puls Method

This method arranges the basic routing equation as below so that the knowns are placed on the left side and unknowns are placed on the right side of the equation.

$$\left(\frac{I_1 + I_2}{2}\right) + \left(\frac{S_1}{\Delta t} - \frac{O_1}{2}\right) = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right)$$

Since this equation contains two unknowns it cannot be solved unless a second independent function is available. In the modified Puls method, a storage-indication curve viz. outflow O versus the quantity $(S/\Delta t+O/2)$ is constructed for the purpose.

In the above equation, it may be noted that subtracting O_2 from $(S_2/\Delta t+O_2/2)$ gives $(S_2/\Delta t-O_2/2)$. This expression is identical to $(S_1/\Delta t-O_1/2)$ on the left side of the equation except for the subscripts. Since the subscript 1 denotes values at the beginning of a time increment and subscript 2 denotes values at the end of a time increment, it is apparent that $(S_2/\Delta t-O_2/2)$ at the end of one time increment is numerically equal to $(S_1/\Delta t-O_1/2)$ for the beginning of the succeeding time increment.

The detailed routing procedure is as follows:

i) Compute the numerical value of left side of the equation for given values of I_1, I_2, S_1 and O_1 for the first time increment.

ii) With this numerical value, which equals ($S_2/\Delta t+O_2/2$), refer storage-indication curve and read outflow O_2 corresponding to this computed value of ($S_2/\Delta t+O_2/2$). The O_2 thus read is the instantaneous outflow at the end of the first time increment.

iii) Subtract this value of O_2 from ($S_2/\Delta t+O_2/2$), which gives the value for ($S_2/\Delta t-O_2/2$). The value of ($S_1/\Delta t-O_1/2$) for the second time increment is equal to ($S_2/\Delta t-O_2/2$) for the first time increment. Consequently the left side of the equation can be computed for the second time increment and the entire procedure is repeated.

4.0 TYPES OF SPILLWAYS

Spillways can be classified as controlled or uncontrolled depending upon whether they are gated or ungated. Further they are also classified based on other prominent features such as control structure, discharge channel or some such other components.

The common types of spillways used are:

i) Overfall or Ogee
ii) Orifice or sluice
iii) Chute or trough
iv) Side channel
v) Tunnel/Shaft or Morning Glory
vi) Siphon

4.1 Overfall or Ogee Spillway

The overfall type is by far the most common and is adapted to masonry dams that have sufficient crest length to provide the desired capacity.

This type comprises a control weir which is ogee or S-shaped. The ogee shape conforms to the profile of aerated lower nappe from a sharp crested weir. The upper curve at the crest may be made either broader or sharper than the nappe. A broader curve will support the sheet and positive hydrostatic pressure will occur along the contact surface. The support sheet thus creates a backwater effect and reduces the coefficient of discharge. The sharper crest on the other hand creates negative pressures, increases the effective head and thereby the discharge.

These spillways are generally provided in Masonry/Concrete dams and also in composite dams as central spillways located in the main river course. Examples are Bhakra dam, Rihand dam, Sriram Sagar dam, Nagarjunasagar dam, Jawahar Sagar dam, Tenughat dam, Srisailam dam, Tawa dam, Ukai dam etc.
A typical ogee spillway is shown in figure below:

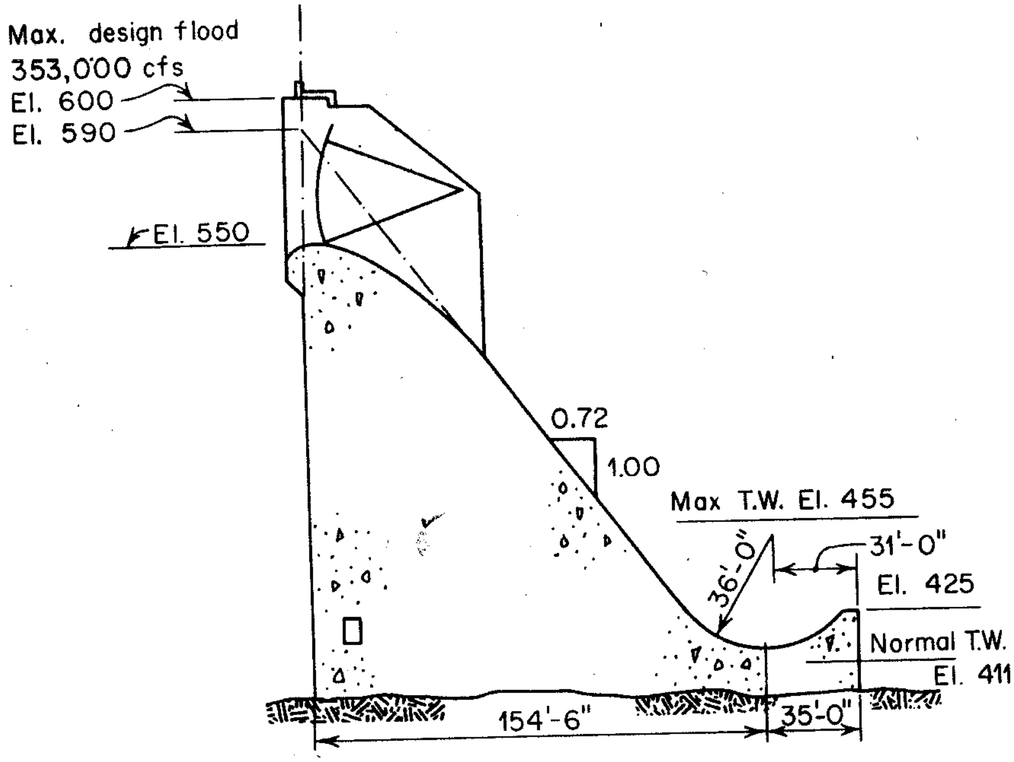


FIG. 37. Guayabo spillway.

4.2 Orifice Spillway

Low crested spillways with either breast wall or sluice type arrangements are now increasingly being provided for flushing out the silt and controlling the silt entry in the power intake which is kept above the spillway crest. These spillways are called orifice or sluice spillways (See figures below).

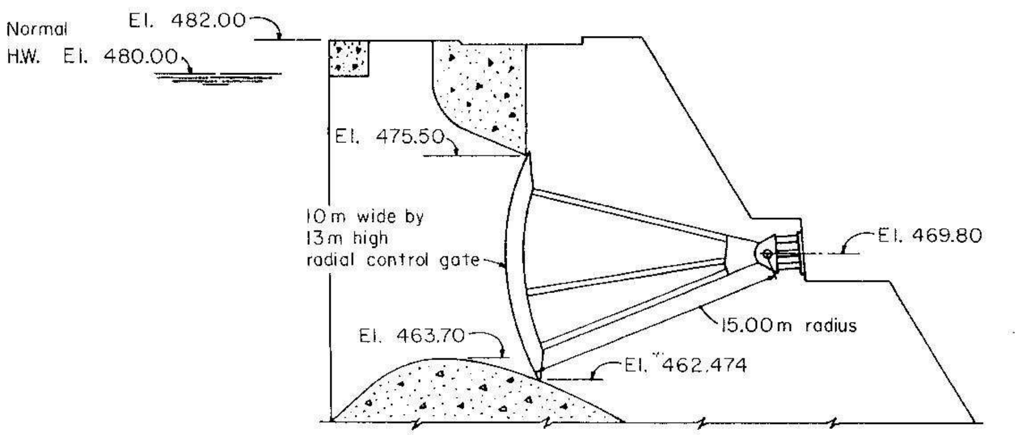


FIG. 17. Spillway, Roseires Dam, Blue Nile River, Republic of Sudan.

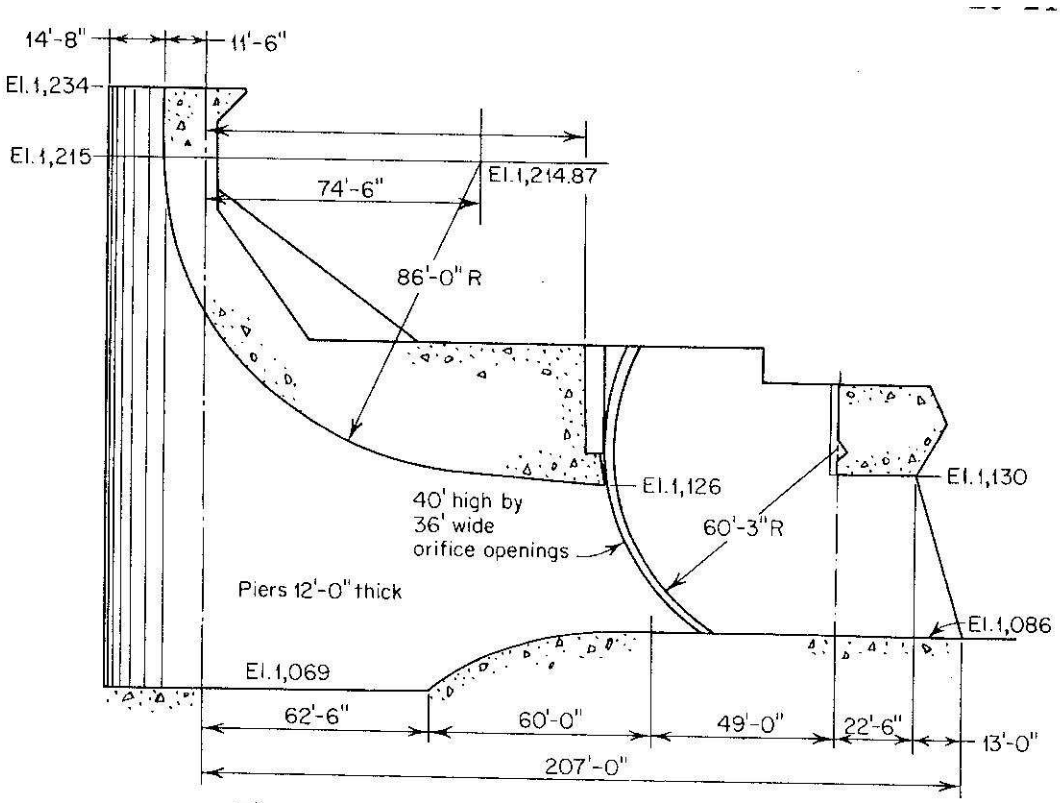


FIG. 19. Mangla Dam spillway, West Pakistan.

The orifice spillways have the advantage of having high discharging capacity due to the high water head. At sites where only a limited area and relatively short length of suitable foundation material are available for the spillway structure, the orifice spillway offers the most economic means of passing the design flood. However, the orifice spillways result in high flow concentration, which increases the size and cost of energy dissipation work below.

Orifice spillways are being provided in many of our diversion dams recently in rivers carrying heavy silt load. The power intake is kept above the spillway crest and as close to the spillway as possible. This kind of spillway arrangements thus performs the dual function of passing the flood and managing the sediment in the reservoir.

Examples of spillways with breast wall type arrangements are in Ranganadi H.E. Project (Arunachal Pradesh), Chamera H.E. Project Stage-I (Himachal Pradesh), Rangit H.E. Project Stage-III (Sikkim) etc. and that of sluice spillways are in Tala H.E. Project (Bhutan), Nathpa Jhakri H.E. Project (Himachal Pradesh), Myntdu H.E. Project Stage-I (Meghalaya) etc.

4.3 Chute Spillway

A spillway where discharge is conveyed from the reservoir to the downstream river through an open channel or chute along a dam abutment or through a saddle is called a chute or trough spillway. The chute is the commonest type of water conductor used for conveying flow between control structures and energy dissipators. Chute can be formed on the downstream face of gravity dams, cut into rock abutments and either concrete-lined or left unlined and built as free-standing structures on foundations of rock or soil.

These are mostly used with earth/rockfill dams and have the following main advantages:

i) Simplicity of design
ii) Adaptability to all types of foundations and
iii) Overall economy by using large amount of spillway excavation in dam construction

Examples of chute spillways are Beas Dam, Ram Ganga Dam, Kolar Dam, Tehri Dam etc.

A typical chute spillway is shown in figure below:

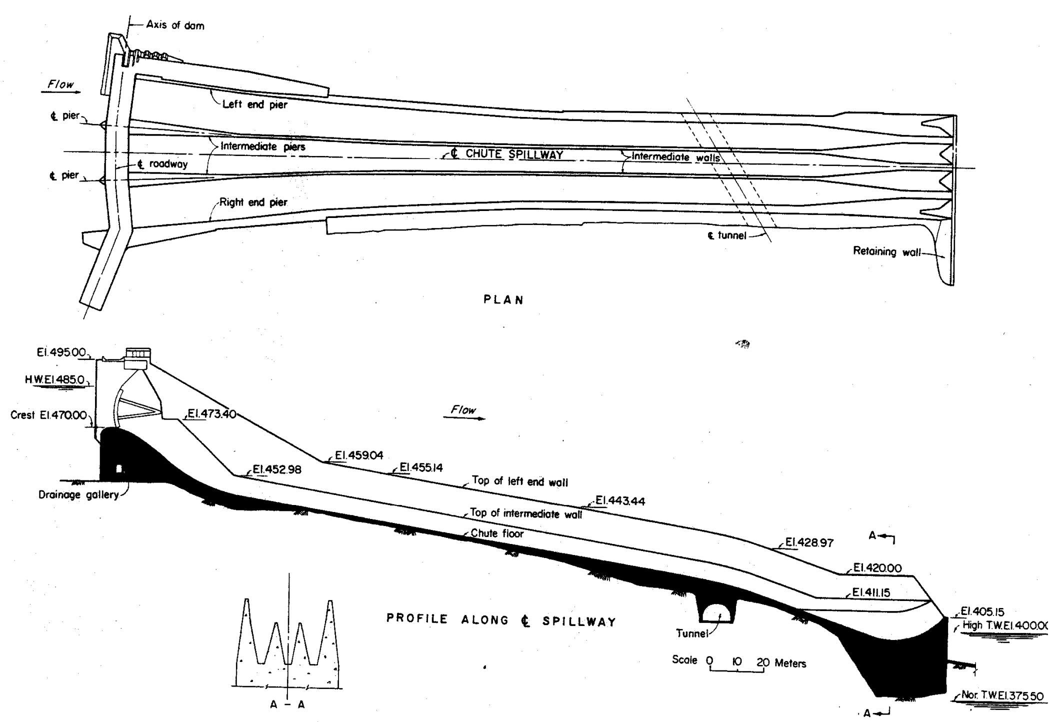


FIG. 20. Derbendi Khan Dam.

4.4 Side Channel Spillway

The distinctive feature of side channel spillway which distinguishes it from chute spillway is that whereas in the chute spillway the water flows at right angle to the axis of the dam, in the side channel spillway, the flow is initially in a channel parallel to the axis of the dam and thereafter it flows in a discharge channel at approximately right angle to the dam axis.

This type of spillway is suited to narrow canyons with steep sides which rise to a considerable height above the dam. This type of spillway is also provided at sites where the overfall type is ruled out for some reason and where saddle of sufficient width is not available to accommodate a trough (chute) type spillway. It is assumed that all the energy of the overfalling water is dissipated in turbulence in the side channel. Example of side channel spillway is Pancheshwar Project.

A typical layout of a side channel spillway is illustrated in figure below:

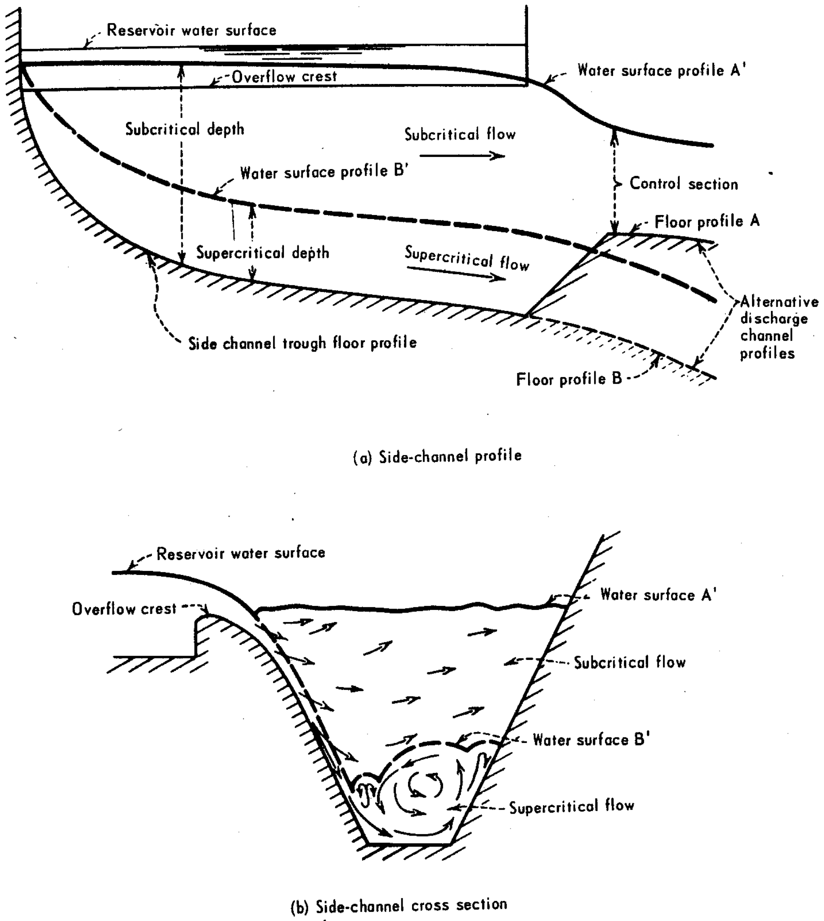


FIG. 25

4.5 Tunnel/Shaft or Morning Glory Spillway

In this type of spillway water enters over the lip of a horizontal circular crest and drops through a vertical or sloping shaft and then flows downstream through a horizontal conduit or tunnel. The spillway is suitable to dam sites in narrow canyons where room for a spillway restricted.

In some instances advantage of the existing diversion tunnel has been taken for conversion into tunnel spillway. A disadvantage of this type is that the discharge beyond a certain point increases only slightly with increased depth of overflow and therefore does not give much factor of safety against underestimation flood discharge as compared to the other types.

Examples of Tunnel/Shaft spillways are Tehri Dam, Itaipau Dam etc. A typical Tunnel/Shaft Spillway is shown in figure below:

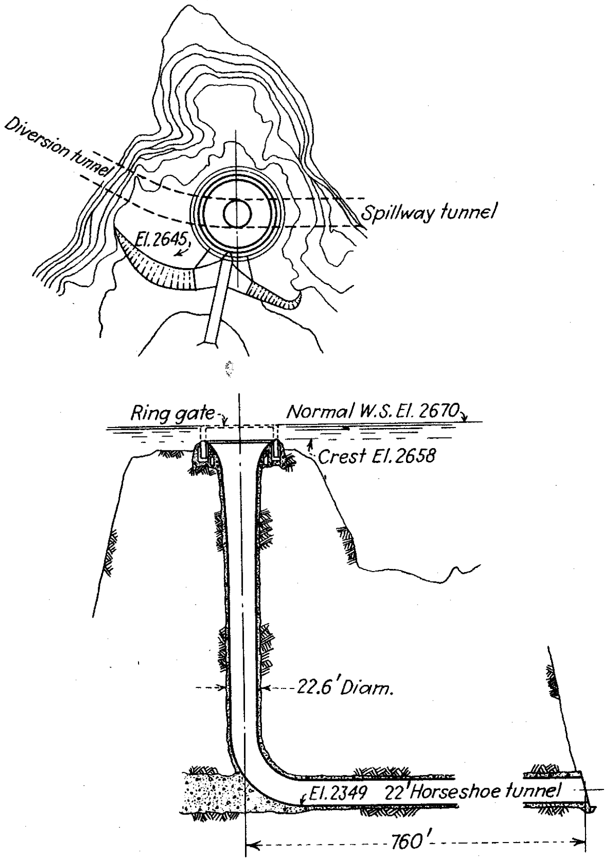


FIG. 27. Owyhee morning-glory and shaft spillway.

4.6 Siphon Spillway

Siphon spillways are based on the principle of siphonic action in an inverted bent pipe. If such pipe is once filled with water, it will continue to flow so long as the liquid surface is higher than the lower leg of the pipe unless of course, the upper leg gets exposed earlier.

Siphon spillways are often superior to other forms where the available space is limited and the discharge is not extremely large. They are also useful in providing automatic surface-level regulation within narrow limits. The siphon spillways prime rapidly and bring into action their full capacity. Therefore, they are especially useful at the power house end of long power channels with limited forebay capacity where a considerable discharge capacity is necessary within a very short time in order to avoid overflow of the channel banks.

However, siphon spillways are not very common mainly because of:

i) Possibility of clogging of the siphon passage way and siphon breaker vents with debris, leaves etc.
ii) The occurrence of sudden surges and stoppages of outflow as a result of the erratic make and break action of the siphon, thus causing fluctuations in the downstream river stage.
iii) The release of outflows in excess of reservoir inflows whenever the siphon operates, if a single siphon is used. Closer regulation which will more nearly balance outflow and inflow can be obtained by providing a series of smaller

siphons with their siphon breaker vents set to prime at gradually increasing reservoir heads.

iv) Vibration disturbances which are more pronounced than in other types of spillways.

A siphon spillway through a dam is shown in figure below:

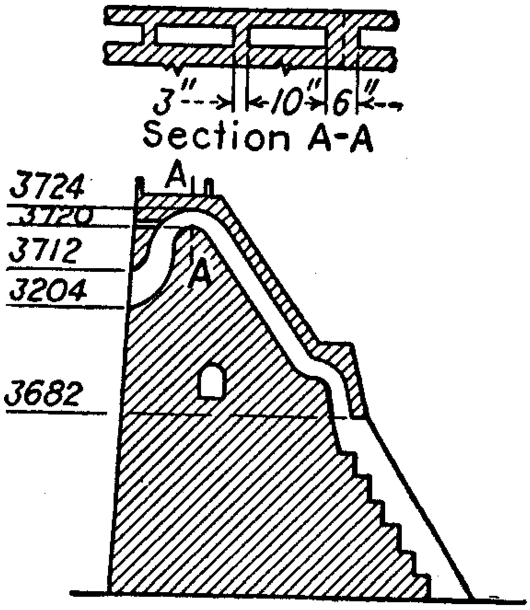


Fig. 35. Siphon spillway in O'Shaughnessy Dam.

5.0 HYDRAULIC DESIGN OF OVERFALL OGEE SPILLWAYS (Refer IS: 6934)

Overfall ogee spillway has its overflow profile conforming, as nearly as possible, to the profile of the lower nappe of a ventilated jet of water over a sharp crested weir. These spillways are classified as high and low depending on whether the ratio of height of the spillway crest measured from the river bed to the design head is greater than or equal or less than 1.33 respectively. In the case of high overflow spillways the velocity of approach head may be considered negligible.

5.1 Shape of Ogee Profile

i) Spillways with vertical upstream face

Upstream Quadrant

The upstream quadrant of the crest may conform to the ellipse:

$$\left(\frac{X_1}{A_1}\right)^2 + \left(\frac{Y_1}{B_1}\right)^2 = 1$$

The magnitude of A_1 and B_1 are determined from the graph P/H_d vs A_1/H_d and B_1/H_d respectively in fig.2 of IS:6934, where,

P = Height of crest from the river bed

H_d = Design Head

Downstream Quadrant

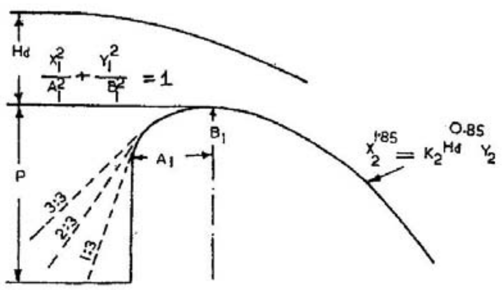
The downstream profile of the crest may conform to the equation:

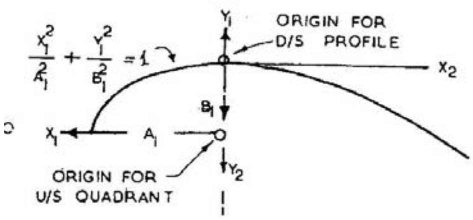
$$X_2^{\,1.85} = K_2 . H_d^{\,0.85} . Y_2$$

The magnitude of K_2 is determined from the graph P/H_d vs K_2 in fig.2 of IS:6934.

ii) Spillways with sloping upstream face

In the case of sloping upstream face, the desired inclination of the face is fitted tangentially to the elliptical profile described under (i) above, with the appropriate tangent point worked out from the equation. The profile of the downstream quadrant remains unchanged.





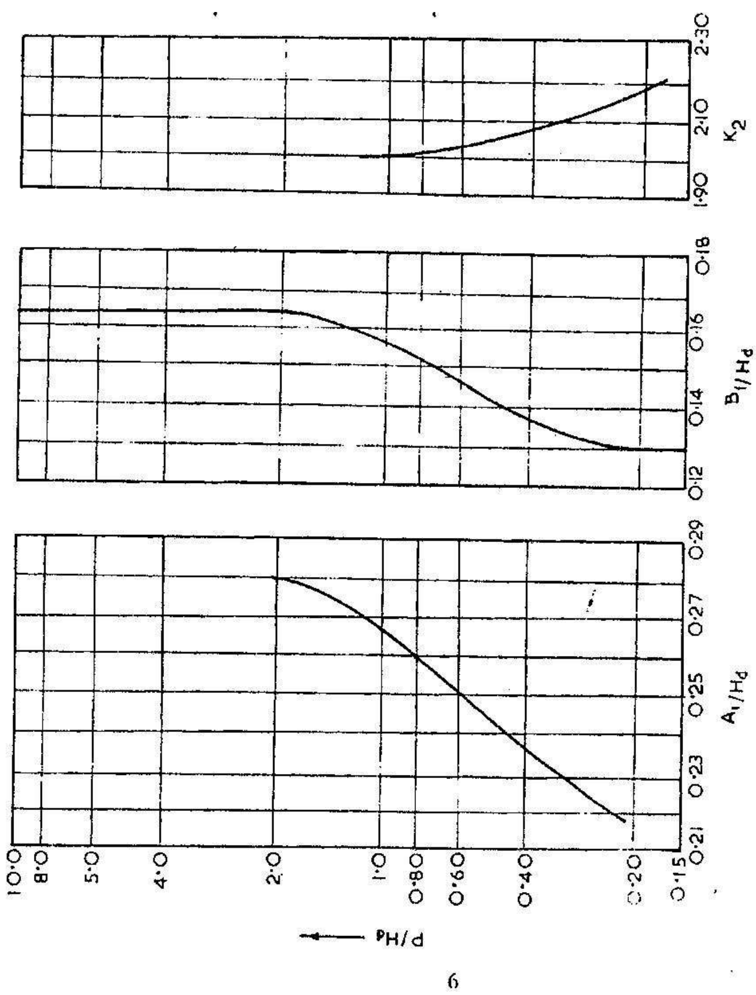
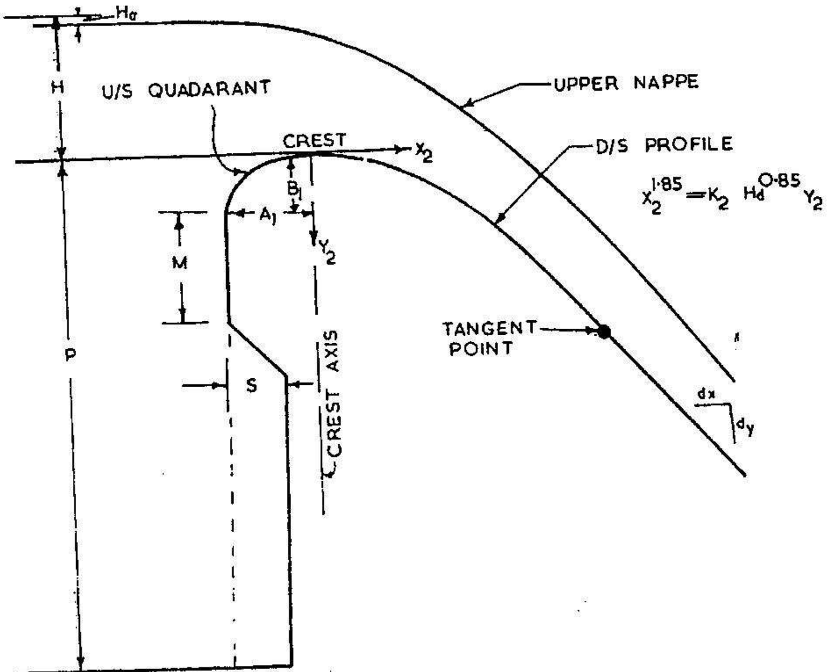


Figure 2 – IS:6934

iii) Spillways with crest offsets and risers

Whenever structural requirements permit, removal of some mass from the upstream face leading to offsets and risers as shown in fig.1 of IS:6934 results in economy. The ratio of risers M to the design head H_d i.e. M/H_d should be at least 0.6 or larger, for the flow condition to be stable. The shapes of u/s and d/s quadrants defined for spillways with vertical upstream face are also applicable to overhanging crests, for the ratio $M/H_d > 0.6$.



1A OVERFLOW SPILLWAY

5.2 Discharge Computations

The discharge over the spillway is generally computed by the equation

$$Q = \frac{2}{3}\sqrt{2g}C.L.H^{3/2}$$

where,

C	=	Coefficient of Discharge
L	=	Effective length of crest
H	=	Head over crest

i) Effective Length of Overflow Crest

The net length of overflow crest is reduced due to contraction caused by abutment and crest piers. The effective length L of the crest may be calculated as follows:

$$L = L' - 2H(N.K_p + K_a)$$

where,

L'	=	Overall length of the crest excluding piers
H	=	Head over crest
N	=	Number of piers
Kp	=	End contraction coefficient of piers
Ka	=	End contraction coefficient of abutment

The pier contraction coefficient, Kp is affected by the shape and location of the pier nose, thickness of pier, the actual head in relation to the design head and the approach velocity. The average pier contraction coefficients may be taken as follows:

Type	Kp

Square-nosed piers with rounded corners of radius about 0.1 times pier thickness	0.02
Round-nosed piers	0.01
Pointed-nosed piers	0.0

The abutment contraction coefficient, Ka is affected by the shape of the abutment, the angle between the upstream approach wall and the axis of flow, the actual head in relation to the design head and the approach velocity. The average abutment coefficient may be taken as follows:

Type	Ka
Square abutments with head wall at 90^o to direction of flow	0.2
Rounded abutments with head wall at 90^o to direction of flow, when 0.5Hd > R > 0.15Hd	0.1
Rounded abutments where R > 0.5Hd and head wall not more than 45^o to direction of flow	0.0

ii) Coefficient of Discharge

The value of coefficient of discharge depends on the following:

a) Shape of the crest
b) Depth of overflow in relation to design head
c) Depth of approach
d) Extent of submergence due to tail water
e) Inclination of the upstream face

Fig. 3 of IS:6934 gives the coefficient of discharge C for the design head as a function of approach depth and inclination of upstream face of the spillway. These curves can be used for preliminary design purposes.

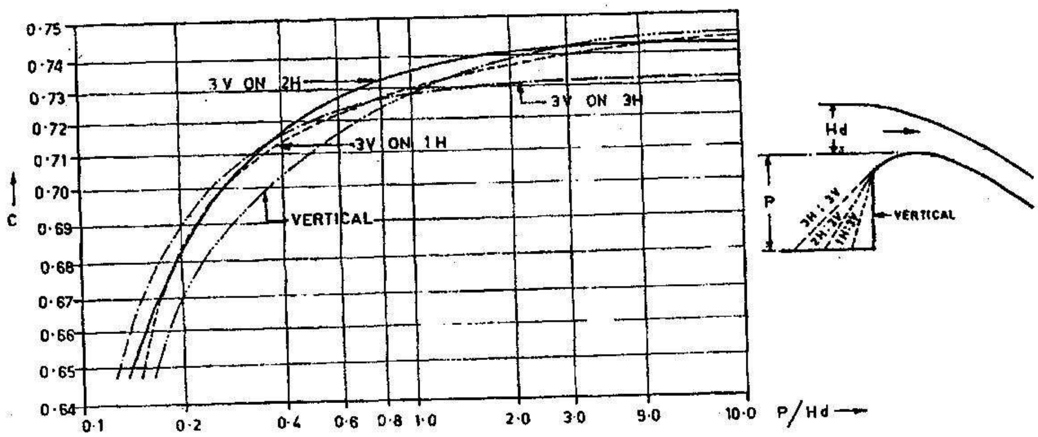


FIG. 3 DISCHARGE COEFFICIENT FOR DESIGN HEAD

Fig. 4 of IS:6934 gives the variation of coefficient of discharge as a function of ratio of the actual head to the design head (i.e. H/H_d). This curve can be used to estimate C for heads other than design head.

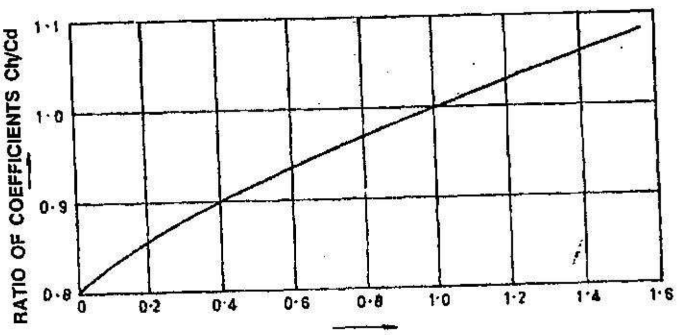
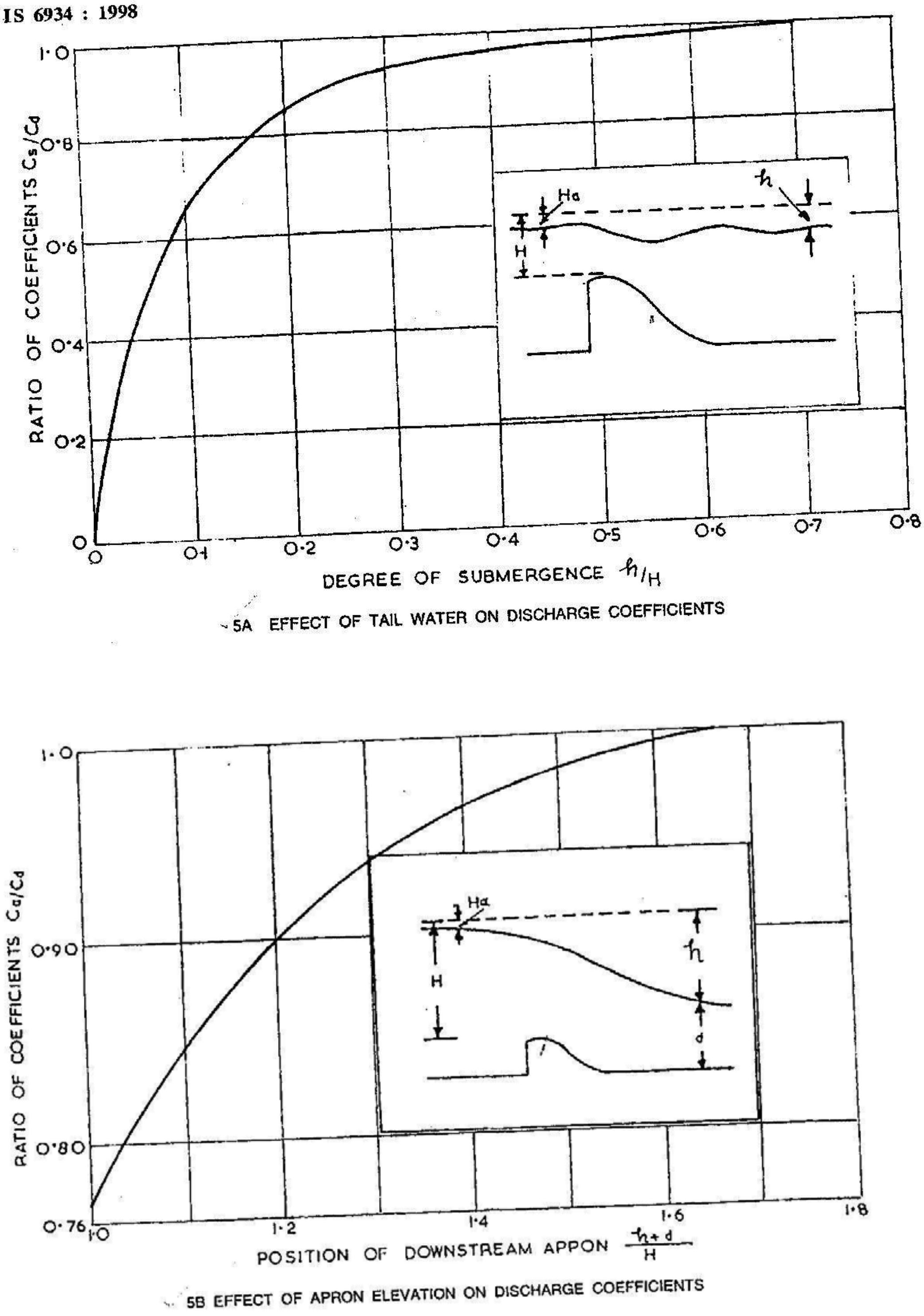


FIG. 4 RATIO OF HEAD ON CREST TO DESIGN HEAD (H/H_d)

The coefficient of discharge is reduced due to submergence by the tail water. The position of the downstream apron relative to the crest level also has an effect on the discharge coefficient. Fig. 5A and 5B of IS:6934 give the variation of C with the above parameters.

IS 6934 : 1998



5A EFFECT OF TAIL WATER ON DISCHARGE COEFFICIENTS

5B EFFECT OF APRON ELEVATION ON DISCHARGE COEFFICIENTS

FIG. 5 COEFFICIENT OF DISCHARGE

iii) Design Head

When the ogee crest is formed to a shape differing from the ideal shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ.

A design head grater than the actual head will push the crest surface into the theoretical nappe and result in greater pressure along the curved surfaces and in lower discharge capacities. Conversely, a design head lower than the actual head pulls the crest surface below the theoretical nappe, resulting in sub-atmospheric pressures over some portion of the crest curve. At the same time the discharge capacity of such a crest curve is increased.

Excessive sub-atmospheric pressures can result in pulsating, inefficient spillway operation, and possibly damage to the structure as a result of cavitations. A certain amount of sub-atmospheric pressure can be attained without undesirable effects. Figure provides a guide for determining the minimum pressures on the crest for various ratios of design head and actual head on the crest.

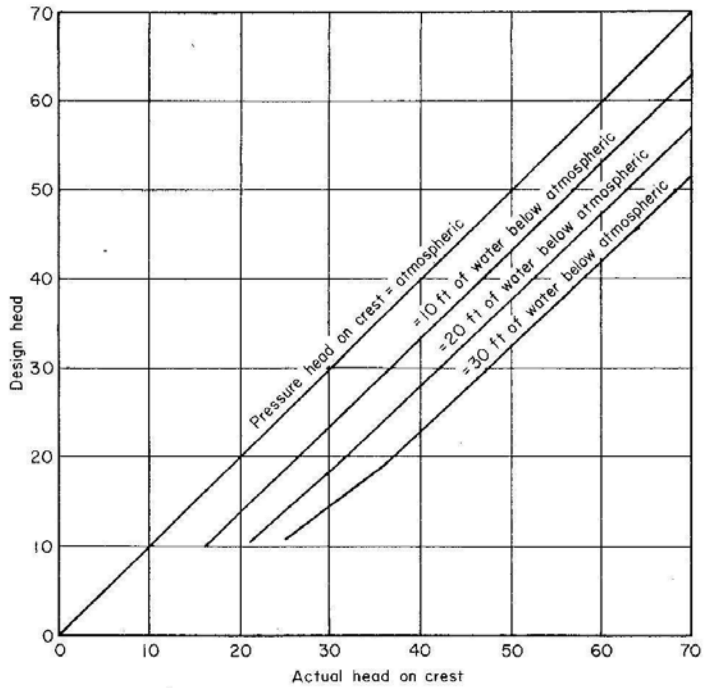


FIG. 4. Minimum pressure on crest as function of heads.

Designing the crest shape to fit the nappe for a head less than maximum head expected often results in economies in construction. The resulting increase in unit discharge may make possible a shortening of the crest length, or a reduction in freeboard allowance for reservoir surcharge under extreme flood conditions.

Because the occurrence of design floods is usually so infrequent, the spillway crests are fitted to the lower nappe of a head which is 75% of that resulting from the actual discharge capacity. Tests have shown that the sub-atmospheric pressures on a nappe-shaped crest do not exceed about half of the design head when the design head is not less that about 75% of the maximum head. An approximate diagram of the sub-atmospheric pressures, as determined from model tests, is shown by figure. The design head is normally kept as 80% to 90% of the maximum head corresponding to MWL.

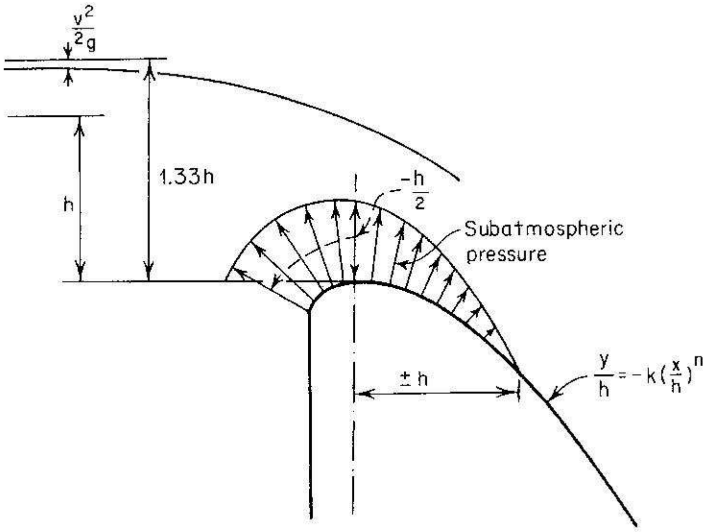


FIG. 5. Subatmospheric crest pressures ratio.

The minimum crest pressure must be greater than cavitation pressure. It is suggested that the minimum pressure allowable for design purposes be 20ft of water below sea level atmospheric pressure and that the altitude of the project site be taken into account in making the calculation. For example, assume a site where the atmospheric pressure if 5ft of water less than sea level pressure, and in which the maximum head contemplated is 60ft; then, only 15 additional feet of sub-atmospheric pressure is allowable.

DESIGN OF ENERGY DISSIPATORS

1.0 General

The waters flowing down the spillway have very high energy. The same if not dissipated can cause considerable erosion/scour downstream which can endanger the dam stability. It is, therefore, necessary to provide adequate downstream protection work or energy dissipation arrangements (EDA) for dissipating the energy downstream of the spillway and minimize erosion of natural river bed.

As per IS : 11223 – 1985 “Guidelines for fixing spillway capacity”, the energy dissipation works should be designed for a flood which may be lower than the inflow design flood for the safety of the dam. The E.D.A. should be designed to work most efficiently for dominant floods. The designs are invariably checked for lower discharges which would correspond to various percentages of the dominant flood.

The problem of designing energy dissipators is one essentially of reducing the high velocity of flow to a velocity low enough to minimize erosion of downstream river bed. This reduction in velocity may be accomplished by any or a combination of the following, depending upon the head, discharge intensity, tail water conditions and the type of the bed rock or the bed material.

The Energy Dissipation Arrangements generally adopted consist of :

i) Stilling Basin
   a) Horizontal apron type
   b) Sloping apron type

ii) Bucket Type Energy Dissipators
   a) Solid Roller Bucket
   b) Slotted Roller Bucket
   c) Flip/Ski Jump Bucket

2.0 Stilling Basin

These are one of the most efficient and commonly adopted Energy Dissipation Arrangements. In stilling basins, the energy is dissipated through the well known phenomenon of hydraulic jump which is the most effective way of dissipating the energy of flowing water. The simplest kind of protection could be used if a jump would form at all stages on a horizontal floor, at the stream-bed level, extending from the dam to the downstream end of the jump. The height of the tail water for each discharge seldom corresponds to the height of a perfect jump. In some cases the sloping apron will permit a hydraulic jump to form at proper depth within the limits of the apron throughout the entire range of spillway discharges and corresponding tail water depths.

IS : 4997 – 1968 (reaffirmed 1995) gives the criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons.

2.1 Stilling Basin with Horizontal Apron

When the tail water rating curve approximately follows the hydraulic jump curve or is slightly above or below it, then hydraulic jump type stilling basin with horizontal apron provides the best solution for energy dissipation. In this case the requisite depth may be obtained on an apron near or at the ground level which is quite economical. For spillways on weak bed rock and weirs and barrages on sand or loose gravel, hydraulic jump type stilling basins are recommended.

Hydraulic Jump type stilling basins with horizontal apron may be classified into the following two categories:

a) Stilling basin in which the Froude number of the incoming flow is less than 4.5. This case is generally encountered on weirs and barrages. The basin is called Basin-I.

b) Stilling basins in which the Froude number of the incoming flow is greater than 4.5. This case is generally encountered in dams. This basin is called Basin-II.

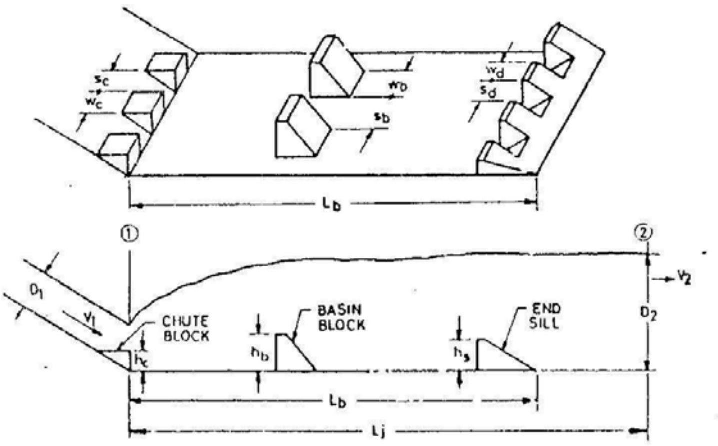


FIG. 1 DEFINITION SKETCH BASIN I AND II

Design Criteria

Factors involved in the design of stilling basins include the determination of the elevation of the basin floor, the basin length and basin appurtenances.

Determination of Level of Basin Floor

Knowing H_L and q; D_c, D_1 and D_2 can be determined from the following formulae or from fig.7 of IS:4997:

$$H_L = \frac{(D_2 - D_1)^3}{4 D_1 D_2} \qquad D_c = \left(\frac{q^2}{g}\right)^{1/3}$$

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2q^2}{D_1 g} + \frac{D_1{}^2}{4}}$$

where,

H_L = Head Loss

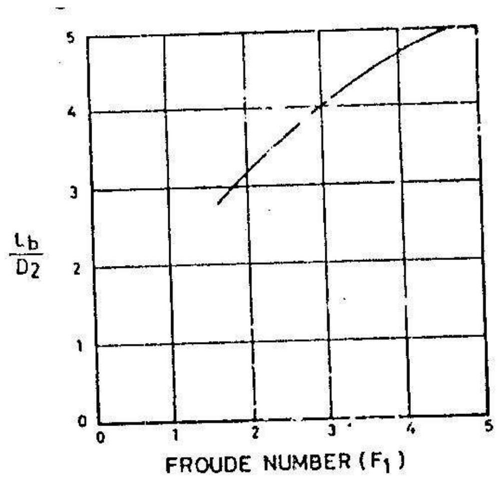
D_1	=	Prejump depth
D_2	=	Post jump depth
q	=	Discharge per unit length
g	=	Acceleration due to gravity

Having obtained D_1 & D_2, the elevation of basin floor may be calculated.

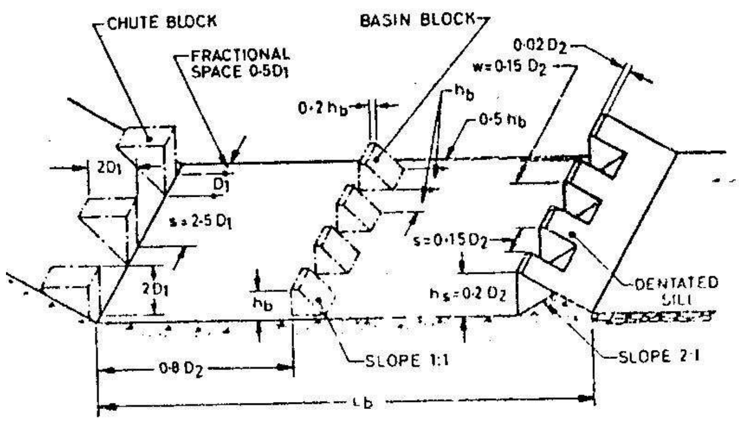
Basin – I

Requirements of basin length, depth and appurtenances for basin-I are as under:

Basin Length and Depth: Length of basin may be determined from the curve in Fig. 8A of IS:4997. The basin is provided with an end sill preferably dentated one. In the boulder reach the sloping face of the end sill is generally kept on the upstream side. Generally the basin floor should not be raised above the level required from sequent depth consideration. If the raising of the floor becomes obligatory due to site conditions, the same should not exceed 15% of D_2 and the basin in that case should be further supplemented by chute blocks and basin blocks. The basin blocks should not be used if the velocity of flow at the location of basin blocks exceeds 15 m/s and in that case the floor of the basin should be kept at a depth equal to D2 below the tail water level. The tail water depth should not generally exceed 10% of D_2.



8A RECOMMENDED LENGTH FOR BASIN I



8B APPURTENANCES FOR BASIN I

FIG. 8 DIMENSION SKETCH FOR BASIN I

Basin Appurtenances: Requirements for basin appurtenances, such as chute blocks, basin blocks and end sill are as below:

a) Chute blocks: Height and top length of chute blocks should $2D_1$ while width should be D_1. The spacing of chute blocks should be kept as 2.5D1 and a space $D_1/2$ should be left along each side wall.

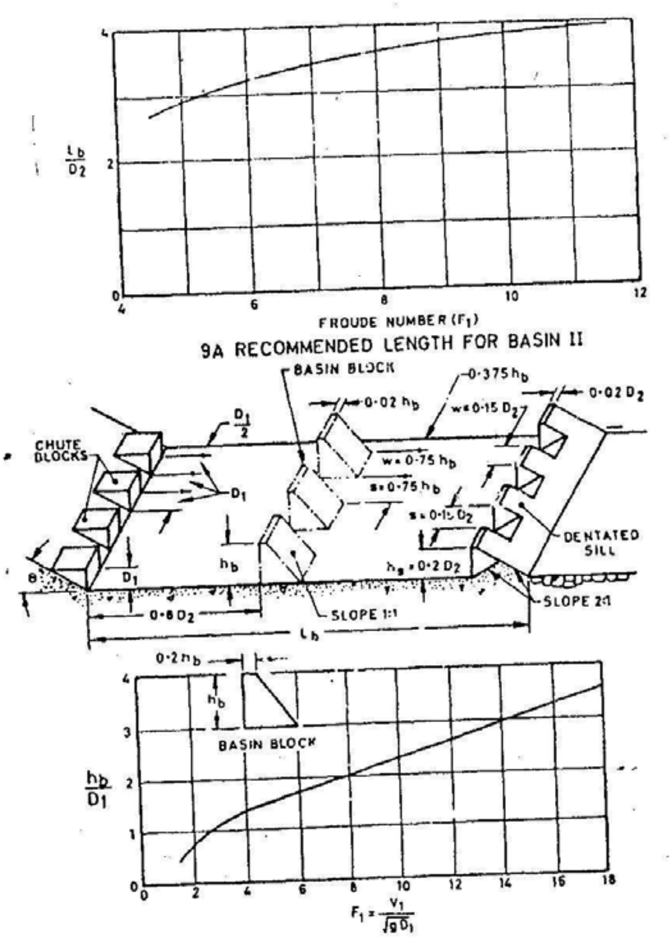
b) Basin blocks: Height of basin blocks in terms of D1 may be obtained from fig.9B of IS:4997. Width and spacing should be equal to their height. A half space is recommended adjacent to the walls. Upstream face of the basin blocks should be vertical. The blocks should be set at a distance of $0.8D_2$ downstream from chute blocks.

c) End sill: Height of the dentated end sill should be $0.2D_2$. Maximum width and spacing should be $0.15D_2$. In the case of a narrow basin, the width and spacing can be reduced but in the same proportion. A dent is recommended adjacent to each side wall.

Basin – II

Requirements for basin length, depth and appurtenances for Basin-II are as under:

Basin Length and Depth: Length of the basin will be determined from the curve in Fig. 9A of IS:4997. The basin should be provided with chute blocks and end sill. The maximum raising of the basin floor shall not exceed 15% of D_2 and basin in that case will be further supplemented by basin blocks. However, when the velocity of flow at the location of basin blocks exceeds 15 m/s, no basin blocks are recommended and in that case the floor of the basin should be kept at a depth equal to D_2 below the tail water level. The tail water depth should not generally exceed 10% of D_2.



9B APPURTENANCES FOR BASIN II

FIG. 9 DIMENSION SKETCH FOR BASIN II

Basin Appurtenances: Requirements for basin appurtenances, such as chute blocks, basin blocks and end sill are as below:

a) Chute blocks: Height, width and spacing of chute blocks should be kept as D_1. The width and spacing may be varied to eliminate fractional blocks. A space $D_1/2$ should be left along each side wall.

b) Basin blocks: Height of basin blocks in terms of D_1 may be obtained from fig.9B of IS:4997. Width and spacing should be equal to three-fourth of the height. A half space is recommended adjacent to the walls. Upstream face of the basin blocks should be vertical. The blocks should be set at a distance of $0.8D_2$ downstream from chute blocks.

c) End sill: Same as Basin I.

2.2 Stilling Basin with Sloping Apron

When the tail water is too deep as compared to the sequent depth D_2, the jet left at the natural ground level would continue to go as a strong current near the bed forming a drowned jump which is harmful to the river bed. In such a case, a hydraulic jump type stilling basin with sloping apron should be preferred as it would allow an efficient jump to be formed at suitable level on the sloping apron.

Hydraulic Jump type stilling basins with sloping apron may be classified into the following two categories:

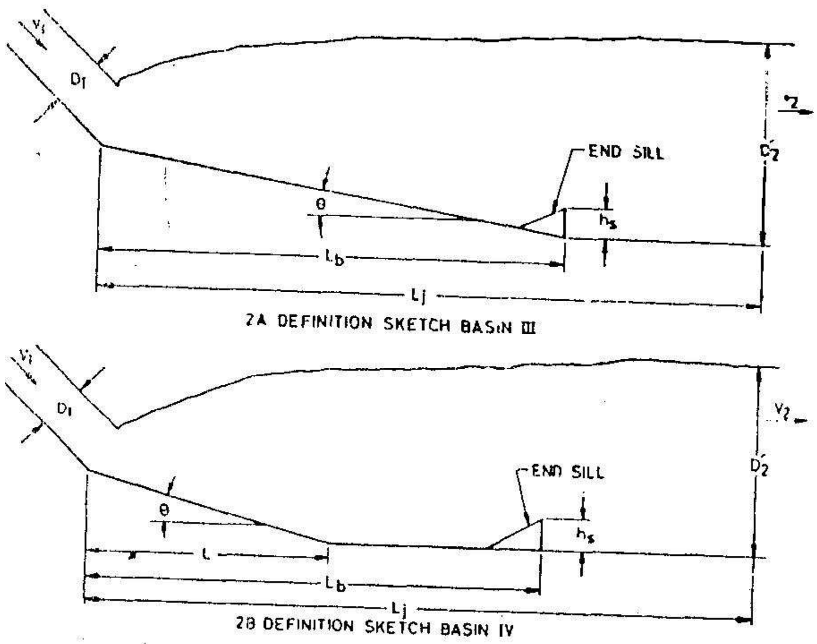


FIG. 2 DEFINITION SKETCHES OF BASIN III AND IV

Basin – III: This is recommended for the case where tail water curve is higher than D_2 curve at all discharges.

Basin – IV: This is recommended for the case where tail water depth at maximum discharge exceeds D_2 considerably but is equal to or slightly greater than D_2 at lower discharges.

Design Criteria

The slope and overall shape of the apron are determined from economic consideration, the length being judged by the type and soundness of river bed downstream. The following criteria should be used only as a guide in proportioning the sloping apron designs.

Basin III

a) Assume a certain level at which the front of jump will form for the maximum tail water depth and discharge.

b) Determine D_1 from the known upstream total energy line by applying Burnoulli's theorem and calculate F_1.

c) Assume a certain slope and determine the conjugate depth D'_2 and length of jump for the above Froude number from fig.5 and fig.3 of IS:6977 respectively. The length of apron should be kept 60% of the jump length.

d) Compare the available tail water depth with D'_2. If they do not match, change the slope or the level of upstream end of the apron or both. Several trails may be required for arriving at final figures.

e) The apron designed for maximum discharge may then be tested for lower discharges, say 25%, 50% and 75% of maximum discharge. If the tail water depth is sufficient or in excess of the conjugate depth for intermediate discharges, the design is acceptable. If not, a flatter slope at the lower apron level should be tried or Basin IV may be adopted.

f) The basin should be supplemented by a solid or dentate end sill of height 0.05 to 0.2D'2 with an upstream slope of 2:1 to 3:1.

IS : 4997 - 1968

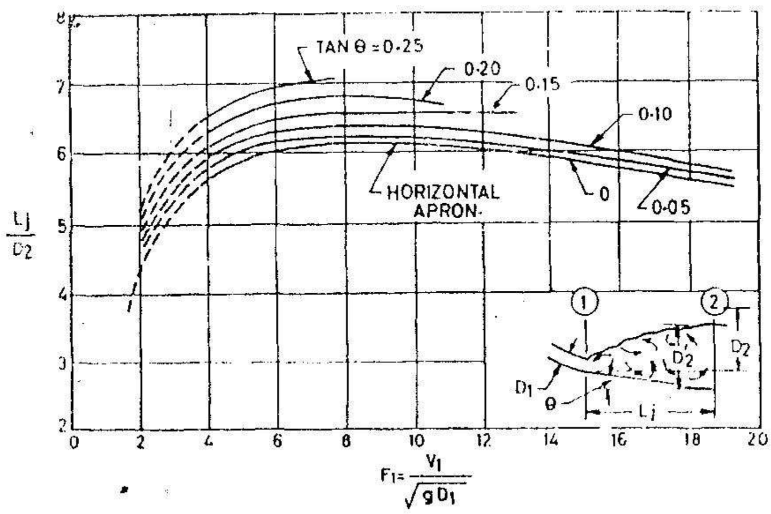


FIG. 3 LENGTH OF JUMP IN TERMS OF CONJUGATE DEPTH D_2 (BASIN III)

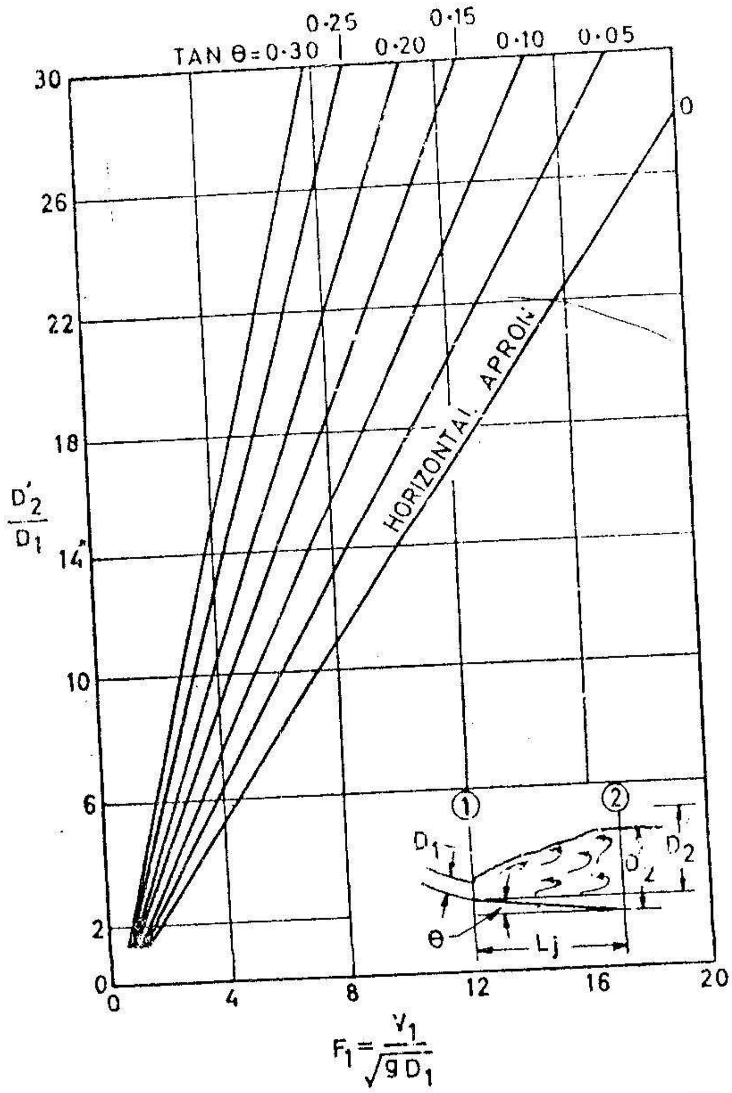


FIG. 5 RATIO OF CONJUGATE DEPTH D'_2 TO D_1 (BASIN III)

Basin IV

a) Determine the discharge at which the tail water depth is most deficient.

b) For the above discharge, determine the level and length of horizontal portion of apron by criteria for horizontal apron.

c) Assume a certain level at which the front of jump will form for the maximum tail water depth and discharge.

d) Determine D_1 from the known upstream total energy line by applying Burnoulli's theorem and calculate F_1. Then find out the conjugate depth D_2 from equation 3.3 of IS:4997.

e) Determine a suitable slope (by trial and error) so that the available tail water depth matches the required conjugate depth D'_2 determined from fig.6 of IS:4997.

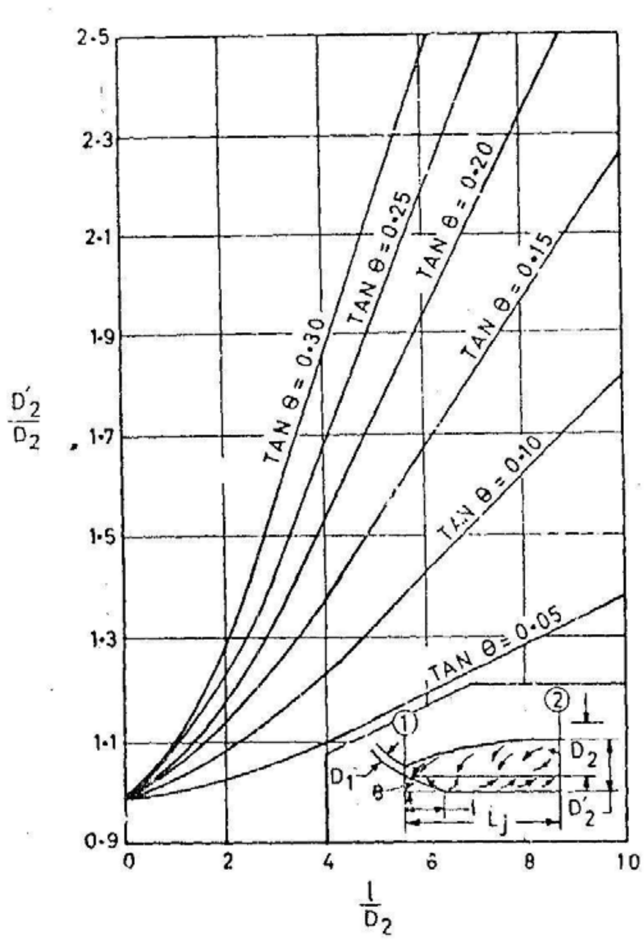


FIG. 6 TAIL WATER REQUIREMENT FOR SLOPING APRONS (BASIN IV)

f) Determine the length of jump for the above slope from fig.3 of IS:4997. If the sum of the lengths of inclined and horizontal portion is equal to about 60% of the jump length, the design is acceptable. If not, fresh trials may be done by changing the level of upstream end of jump formation.

g) The basin should be supplemented by a solid or dentate end sill of height 0.05 to $0.2D_2$ with an upstream slope of 2:1 to 3:1.

3.0 Bucket Type Energy Dissipators

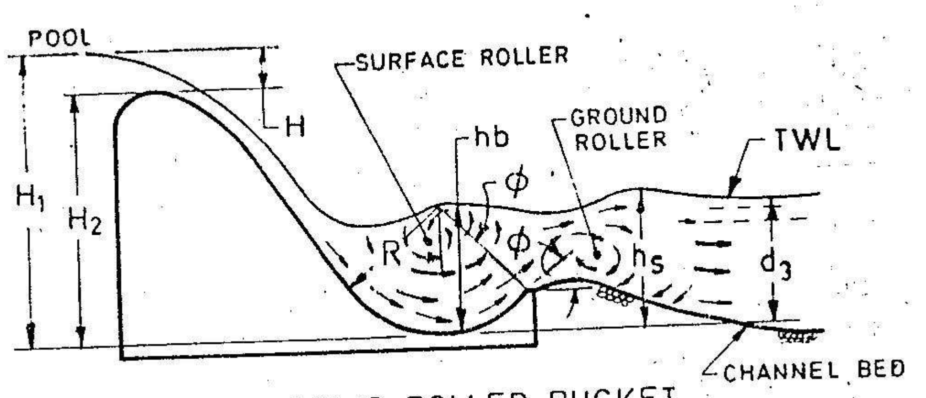
The bucket type energy dissipators generally used are:

i) Solid Roller Bucket
ii) Slotted Roller Bucket
iii) Ski-Jump/Flip Bucket

IS:7365–1985 "Criteria for Hydraulic Design of Bucket Type Energy Dissipators" is normally used for carrying out hydraulic design.

3.1 Solid Roller Bucket

An upturn solid roller bucket is used when the tail water depth is much in excess of the sequent depth. The dissipation energy occurs as a result of formation of two complementary elliptical rollers, one in bucket proper called the surface roller, which is anti clockwise (if the flow is to the right) and the other downstream of the bucket called the ground roller, which is clockwise.



SOLID ROLLER BUCKET

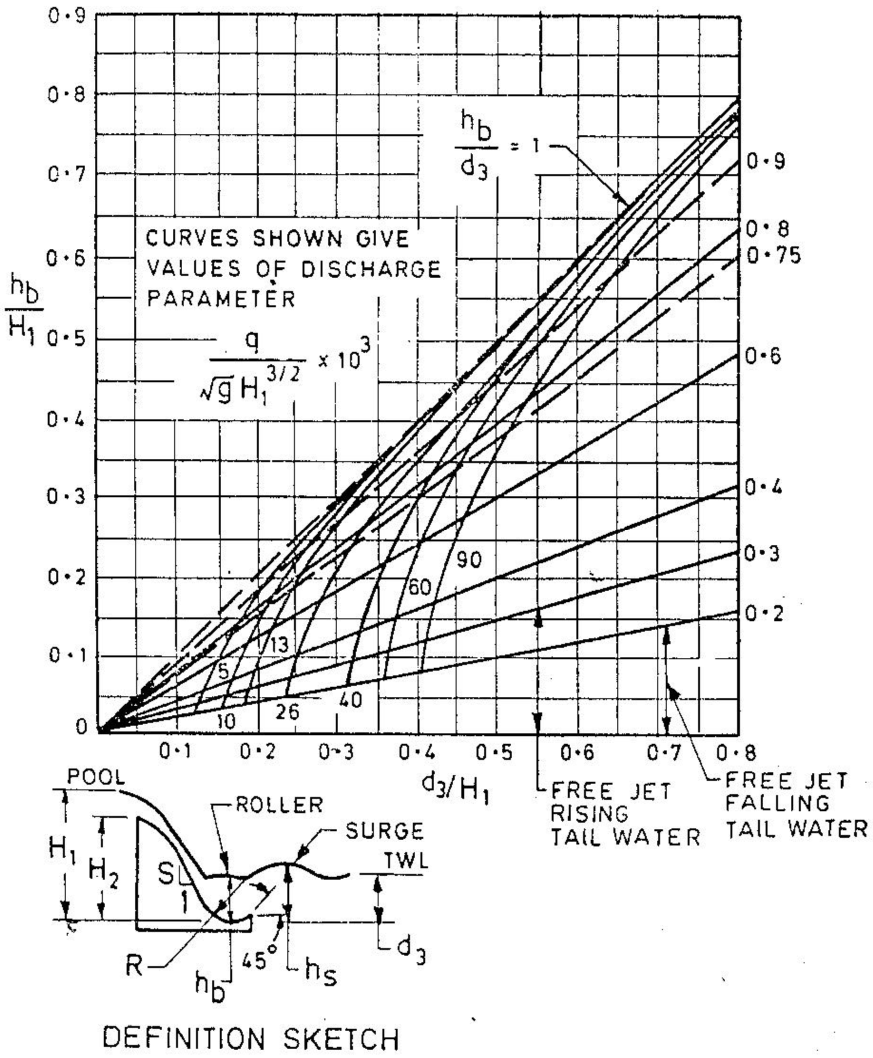
The hydraulic design involves determination of

a) Bucket invert elevation
b) Radius of bucket
c) Bucket lip shape and lip angle

a) Bucket Invert Elevation

Normally the invert level of a roller bucket is so fixed that the difference in the design maximum tail water level and the invert level (d_3) is between 1.1 to 1.4 times the sequent depth (d_2). It has been seen that a satisfactory energy dissipation is obtained when the roller height (h_b) is between 75 and 90 percent of the tail water depth (d_3). If the aforesaid two criteria are satisfied, then the surge height (h_s) measured above the invert level is 105 to 130 percent of the tail water d_3, that is, h_s/d_3 = 1.05 to 1.3.

Charts at fig.4 and fig.5 of IS:7365 are used for determining the roller depth (h_b) and the surge height (h_s) respectively.



b) Radius of the Bucket

The values given in fig.4 of IS:7365 shows the range of H1/R for which good roller action can be expected. One formula which has been found to be widely applicable for a bucket lip angle of 45° is as under:

$$\frac{R}{H_1} = 8.26X10^{-2} + 2.07X10^{-3}F_D + 1.4X10^{-5}F_D^{\ 2}$$

where,

F_D = Discharge parameter $= \frac{q}{\sqrt{gH_1^{\ 3}}}X10^3$

H_1 = Reservoir Pool Level – Bucket I

There are many other empirical formulae available for calculating the radius of bucket. The bucket radius is chosen to fall within the recommended ranges (fig.4, IS:7365) consistent with economical and structural considerations.

RANGE OF H_1/R FOR SATISFACTORY PERFORMANCE OF BUCKET

$\frac{q}{\sqrt{g}\ H_1^{3/2}}$	Equivalent F	Range of H_1/R
0·090	5·4	2 to 5
0·060	6·7	2 to 6
0·040	8·3	2 to 6
0·026	10·3	3 to 6
0·013	14·7	3 to 8

Range of variables:

Spillway slopes (S : 1)	1 : 1 to 1·67 : 1
Lip angle (ϕ)	45° [The curves may also be used for other lip angles (*see* 4.2.2.3)]
H_2/H_1	0·68 to 0·93

c) Bucket Lip

A flat topped lip tends to lower the jet after it leaves the lip and the size and strength of the ground roller would reduce. This is not desirable from the point of view of prevention of erosion near the lip. Therefore, a downstream slope of 1 in 10 or slightly steeper than that may be given to the bucket lip. The width of the lip should not be more than one tenth of the radius of bucket. However the minimum width may be kept as one metre.

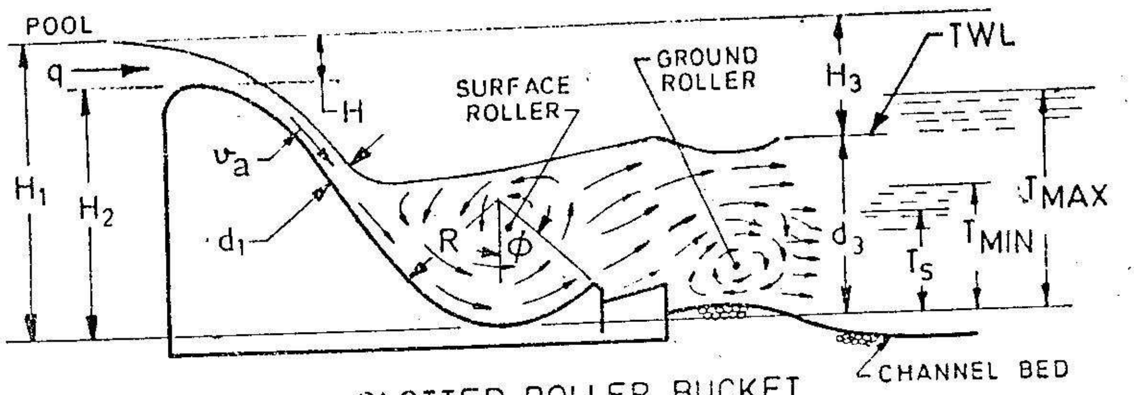
A 45^o bucket lip angle with the horizontal is generally found to be satisfactory for most cases where the discharge parameter lies between 30 and 80.

Model studies are desirable for finalizing the parameters/arrangements.

3.2 Slotted Roller Bucket

This is an improvement over the solid roller bucket arrangement. In the slotted roller bucket, a part of the flow passes through the slots, spread laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area resulting in a less violent flow concentrations compared to those in a solid roller bucket. The height of boil is also reduced in case of slotted roller bucket. The slotted roller bucket provides a self cleaning action to reduce abrasion in the bucket.

Although a slotted roller bucket is an improvement over the solid roller bucket, experience has shown that bucket teeth are vulnerable to damage on account of various reasons like boulders rolling down the spillway, unsymmetric gate operation causing heavy discharge intensities etc. Slotted roller buckets are not recommended in bouldery stages of the river.



SLOTTED ROLLER BUCKET

The hydraulic design involves determination of

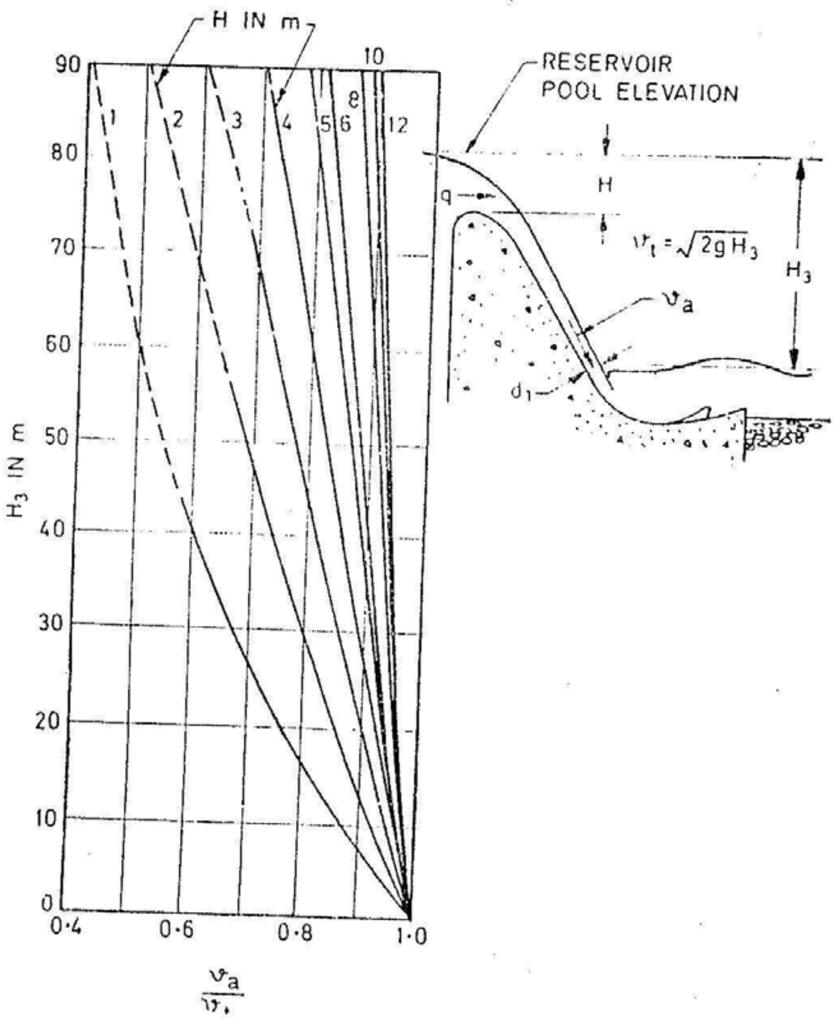
a) Bucket radius
b) Bucket invert elevation
c) Bucket lip angle
d) Tooth dimensions

a) Bucket Radius

- Calculate q, the discharge per meter width of bucket.
- Calculate v_t , the theoretical velocity of flow entering the bucket using the formula,

  $$v_t = \sqrt{2gH_3}$$

  where, H_3 is difference in reservoir pool elevation and tail water level.
- From fig.7 of IS:7365, find v_a, the actual velocity of flow entering the bucket.
- Find $d_1=q/v_a$ and F1.
- From fig.8 of IS:7365, find minimum allowable bucket radius.



Curves for determination of velocity entering bucket for steep slopes 0·8 : 1 to 0·6 :

FIG. 7 DESIGN OF SLOTTED ROLLER BUCKET — VELOCITY OF JET ENTERING BUCKET

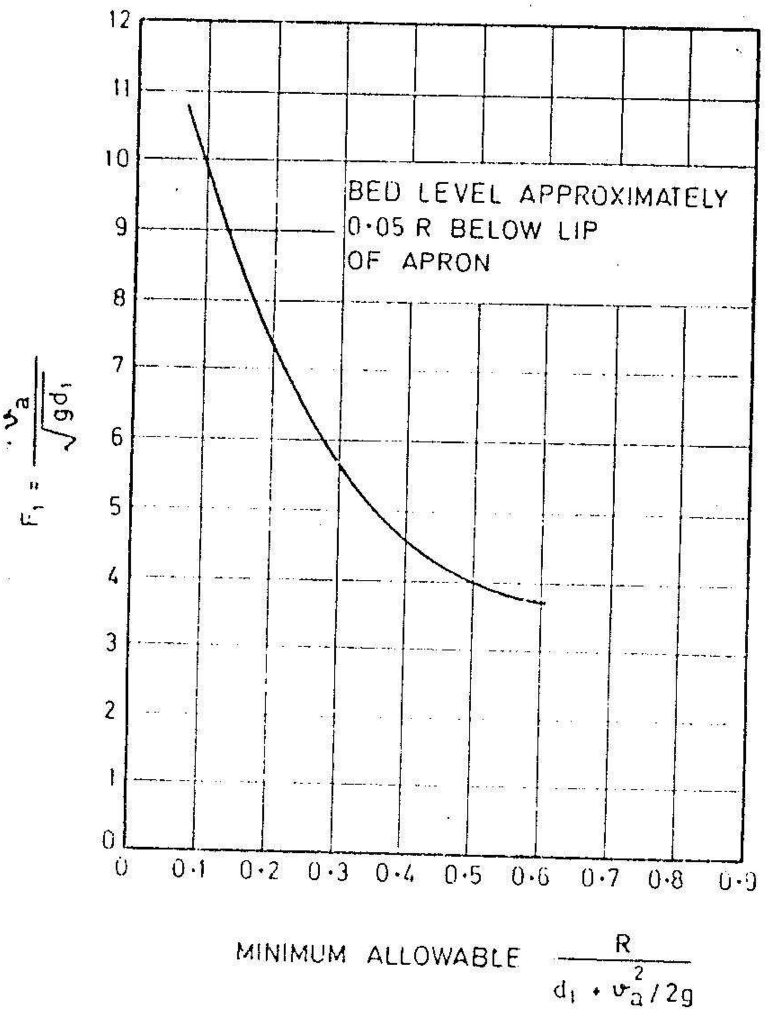


FIG. 8 DESIGN OF SLOTTED ROLLER BUCKET — MINIMUM ALLOWABLE BUCKET RADIUS

b) Bucket invert elevation

- Find the minimum tail water depth T_{min} and maximum tail water depth T_{max} from fig.9 and fig.10 respectively of IS:7365.
- Set such bucket invert elevation for which tail water elevations are between tail water depth limits.

c) Bucket lip angle

Same as solid roller bucket.

d) Tooth dimensions

- Width of tooth is kept as o.125R and spacing of tooth is kept as 0.05R, where R is the radius of the bucket.
- Detailed tooth dimensions are given in fig.12A and fig.12B of IS:7365.

Model studies are desirable for confirming the design parameters. It shall be ensured that the teeth perform cavitation free.

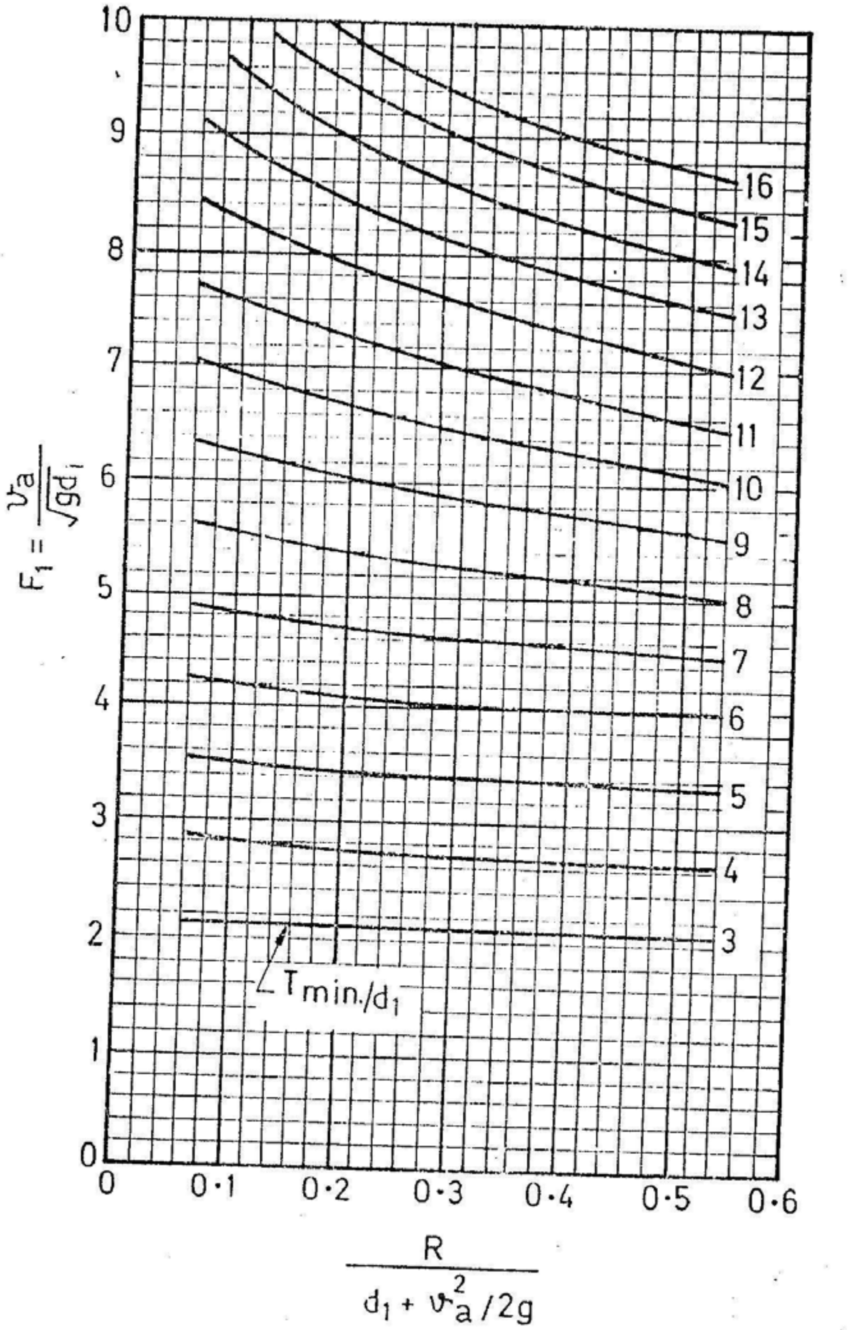


FIG. 9 DESIGN OF SLOTTED ROLLER BUCKET — MINIMUM TAIL WATER LIMIT

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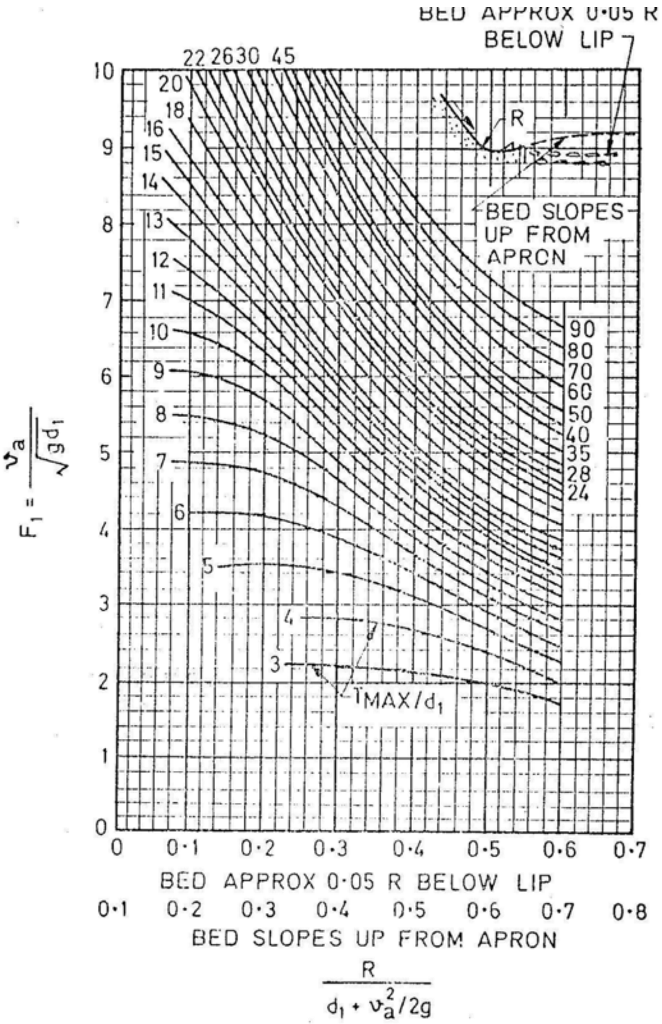
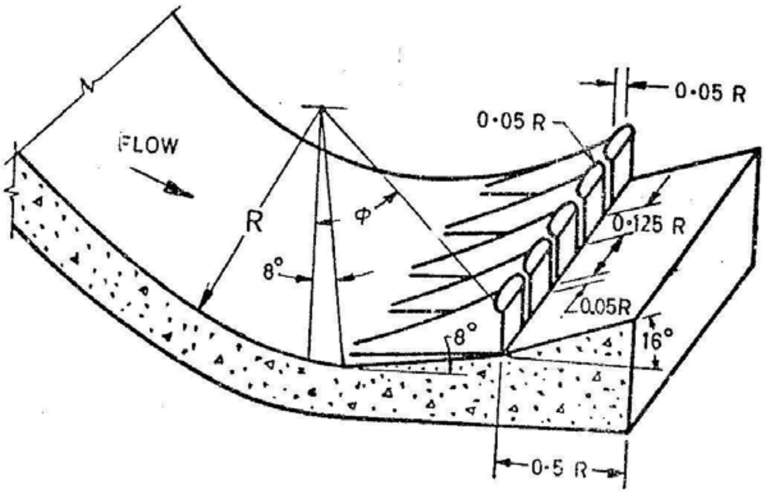


NOTE 1 — For channel bed elevation 0·05 *R* below apron lip or lower, use coordinates for bed approx 0·05 *R* below lip.

NOTE 2 — For channel bed elevation higher than 0·05 *R* below apron lip, use coordinates for bed sloping up from apron.

FIG. 10 DESIGN OF SLOTTED ROLLER BUCKET — MAXIMUM TAIL WATER LIMIT

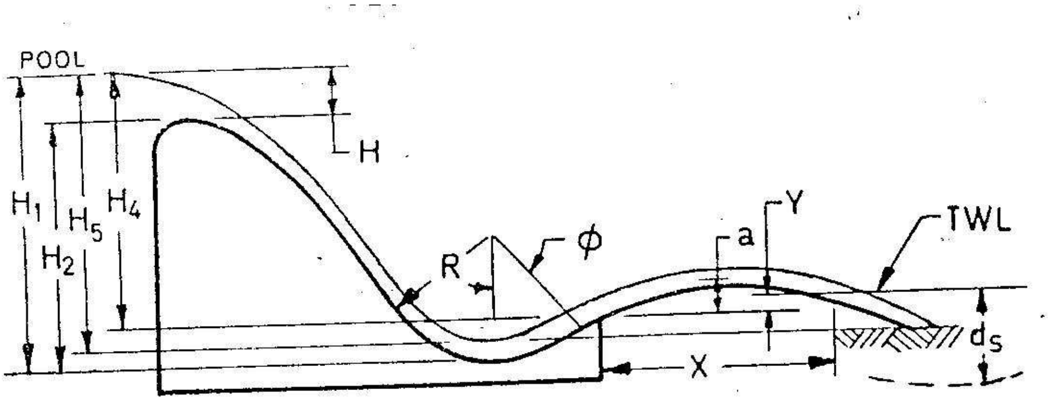


12A Dimensions of Slotted Roller Bucket

FIG. 12 SLOTTED ROLLER BUCKET — *Contd*

3.3 Ski Jump Bucket

This bucket is used when the tail water depth is insufficient for the formation of hydraulic jump and when the bed of the river channel downstream consists of sound rock capable of withstanding the impact of high velocity jet. The flow coming down the spillway is thrown away from the toe of the dam to a considerable distance downstream as a free discharging upturned jet which falls on the channel bed downstream. There is no energy dissipation within the bucket. The device is used mainly to increase the distance from the structure to the place where the jet hits the channel bed. In the ski jump bucket, only part of the energy is dissipated through interaction of the jet with the surrounding air. The remaining energy is imparted to the channel bed below.



TRAJECTORY BUCKET

Design Criteria

The principal features of hydraulic design of trajectory bucket consist of the following:

a) Bucket shape
b) Bucket invert elevation
c) Bucket radius
d) Bucket lip elevation and exit angle
e) Trajectory length
f) Estimation of downstream scour

a) Bucket shape

The performance of the trajectory bucket is judged mainly by the trajectory height and length of throw. Generally a circular shape is preferred from practical consideration.

b) Bucket invert elevation

This depends on the site and tail water condition. For a clear flip action, the lip should be kept above the maximum tail water level. However this may not always be possible. Some of the various considerations which are taken into account while fixing bucket invert elevation are as under:

i) A minimum concrete cover of 1.5 metes over the bed rock

ii) A submergence of not more than 70% of the sequent depth over the lip of the bucket

iii) A safe maximum submergence equal to critical depth over the bucket lip elevation

An attempt is made to keep the bucket invert of the trajectory bucket as high as possible consistent with economy. The hydraulic performance is normally verified in a model.

c) Bucket radius

The radius of bucket should not be less than 3 times the maximum depth of flow (d_1) entering the bucket to avoid separation tendency.

The formula generally used for determining the radius of bucket is as under:

$$R = 0.6 to 0.8 \sqrt{H.H_5}$$

where,

H	=	Depth of overflow over the spillway crest
H_5	=	Reservoir Pool Elevation – Jet Surface Elevation at bucket invert

d) Bucket lip elevation and lip angle

The lip angle affects the horizontal throw distance. The factors affecting the horizontal throw distance also include the initial velocity of the jet and the difference in elevation between the lip and the tail water. Normally adopted lip angle is between 30^o and 40^o. Greater the exit angle grater will be the distance of throw. However the jet impinges on the tail water at a steeper angle which results in deeper scour. For submerged lips the lower lip angle of 30^o may be adopted to minimize sub-atmospheric pressures on the lip.

The lip shall be made flat in case tail water level is lower than the lip level. However, if the tail water level is higher than the lip level, the lip shall slope downstream about 1 in 10. In some cases necessity of aeration may arise which may be finalized after model studies.

e) Trajectory length

The following expression may be used for calculating the horizontal throw distance:

$$\frac{X}{H_v} = Sin2\phi + 2Cos\phi\sqrt{Sin^2\phi + \frac{Y}{H_v}}$$

where,

X = Horizontal throw distance from bucket lip to the centre – point of impact with tail water

Y = Difference between the lip level and tail water level,

sign taken as positive for tail water below the lip level
and negative for tail water level above the lip level

H_v= Velocity head of jet at bucket lip

Φ = Bucket lip angle with the horizontal in degrees

For the conditions when Y is negative, model studies may be carried out to confirm the value of horizontal throw distance (X) and vertical distance of throw (a).

Vertical distance of throw above the lip level may be calculated from the following formula:

$$a = \frac{v_a^{\ 2} Sin^2\phi}{2g}$$

where,

a = Vertical distance from the lip level to the highest point of the centre of jet

V_a = Actual velocity of flow entering the bucket

Φ = Bucket lip angle

g = Acceleration due to gravity

f) Estimation of downstream scour

The factors governing scour below trajectory buckets are the discharge intensity, height of fall, water level, lip angle, mode of operation of spillway, degree of homogeneity of rock, type of rock, time factor etc. However, restricting the analysis of correlating the scour depth with two dominant factors, namely discharge intensity (q) and the total head (H_4), the scour depth can be worked out by the following equation:

$$d_s = m(qH_4)^{0.5}$$

where,

d_s = depth of scour

m = constant (0.36 for minimum scour, 0.54 probable scour under sustained operation, 0.65 for ultimate scour)

q = discharge intensity

H_4 = reservoir pool elevation – bucket endsill elevation

Experience has shown that erosion upto the ultimate scour level will take place irrespective of the type of rock etc. The process is merely a function of time. In case of mega projects, pre-formed plunge pool upto the scour level is being invariably considered.

FINITE ELEMENT ANALYSIS OF DAMS

In India there are more then 5000 large dams and numerous medium and small dams constructed for various purposes like flood forecasting, irrigation, hydropower, water supply etc. Construction of dams for water resources development and management is an on-going process and many more projects are at present in planning stage. Design and analysis of dam is a vital aspect, not only for new dams but also for the old dams requiring rehabilitation and strengthening. Dams are classified as; Gravity Dams, Arch dam, Buttress dam and Embankment (Earth &Rockfill)dams. The type of dam depends on construction materials used, method of construction, etc. The design procedure and criteria used for different type of dams is specified in relevant design codes.

The Gravity Dams are designed with a consideration that all forces coming upon the body of the dam are resisted by its weight alone. Whereas, in the Arch Dam forces are transmitted to the thrust pads and to the abutments. Earth and Rockfill Dams are built using locally available soils, rock masses. Earth and Rockfill dams are also Gravity Structures as they resist all the forces coming upon them by virtue of their weight alone. However, their design is quite complex as compared to the concrete dam, due the heterogeneous nature of the material used, complexity in the material properties and difficulties in predicting soil-foundation interaction .

Normally, for the design of gravity dams, conventional approach which is based on the laws of Statics is widely used. This approach is simple in conception and calculations. Gravity dam is designed against overturning and sliding. It is analysedfor tensile stress, shear stress and maximum comprehensive stress induced in the body and foundation of the dam. However, the conventional approach of the analysis does not fully take into account the elastic properties of the foundation and dam material. The variation in the material properties of the foundationis complex and missed out by this method. The results obtained by this approachare very conservative. It does not give the complete picture of the distribution of stresses, strains and displacements developed throughout the body of the dam and the foundation. The conventional analysis gives quite acceptable design foroverturning and sliding. However, calculation of stresses by conventional methods are highly approximate, moreover effects of openings like inspection galleries, drainagesetcand heterogeneous nature

&non-linear behavior of the material cannot be accounted for the analysis. The deficiencies of the conventional methods can be overcome by using numerical methods. The numerical methods combined with computational power of the computers make it possible to do the complete stress analysis of any type of dam with a desired accuracy.

The problems involved in the analysis of continuous structure such as dams and their foundation are generally analyzed by differential equation or integral statement for which closed solution is possible. Analytical / Mathematical solutions are available or possible only for Simplified situations. The Finite Element analysis of discretizing and approximating the continuum is one of the most general and useful numerical method for such analysis.FEM gives approximate but acceptable solution for the unknown quantities at the discrete number of Points in the continuum.

Finite Element models are used for linear elastic static and dynamic analyses and for nonlinear analyses that account for interaction of the dam and foundation. The advantage of Finite Element Analysis is the capability of modeling complex geometries including discontinuities, corners, openings, galleries and wide variations in material properties.It can also model thermal behavior and couple thermal stresses with other loads. An important advantage of this method is that complicated foundations involving various materials, weak joints,fractures and faults can also be readily modeled.

The Finite Element (FE) Analysis in theory involves the following steps:

1. Discretisation and selecting element configuration
2. Selection of Displacement models
3. Derivation of Element stiffness matrix by using Variational Principles
4. Assembly of element equations to obtain global or assemblage equation
5. Solve for Primary Unknowns
6. Calculating Secondary unknowns
7. Interpretation of results

Performing FE analysis by hand involves lot of computations and as the number of elements in the model increases beyond say 10 elements, the computations becomes too complex to handle manually. However the main advantage with FEM is that many of the steps above are amenable to automation and can be programmed on a computer as modules to be used in any FEanalysis. The basic elements and their element stiffness matrices can be stored in the program as element library. A standard program can be written for assembly of n number of elements. The only input that is required is the co-ordinates of the nodes, element connectivity and material properties. Once the boundary conditions are implemented in the form of nodal loads and nodal restraints/constraints, the problem finally takes a form of solving n number of simultaneous equations. A simple algorithm based on Gauss Elimination Method can easily take care of solving these equations. The commercially available software has made the complex looking FEM very easy and simple for the user. The typical steps for FEM using any standard software are as follows

1. Creating geometric model with real dimensions to the extent possible.
2. Discretisation of the geometric model using finite elements of appropriate type and order (meshing).
3. Applying restrains and constrains i.e. defining degrees of the freedom of the nodes lying on the restrained surfaces
4. Applying the loads like hydrostatic load, dead weight, uplift pressure etc.
5. Solving the model
6. Generation of results for secondary unknowns like principle stresses, strains, strain energy, Von-Mises stresses etc.
7. Interpretation of the results

1.0 Discretisation for Gravity Dams

Concrete/Masonry Dams are typically constructed in the form of independent blocks across the river. Each block may have different geometrical dimensions, material properties and geological conditionsfor foundation. Analysis of each block can be performed separately using the exact cross section of the block, local foundationconditions &geological properties. Fig 1 shows the typical gravity dam constructed in the form of independent blocks. It is advantageous to perform a 3 Dimensional analysis for each of the block independently using 8 noded linear brick element or 20 noded parabolic brick element. As foundation rock plays a major role in supporting the dam weight, it is important that rock foundation is taken into account upto a appropriate distance. Normally, the depth below the foundation considered in theFEM analysis is kept equal to the height of the dam and width of the foundation equal to three times the width of the dam at the base level. In case of non-uniform geological conditions, this range can be however increased to an appropriate level.

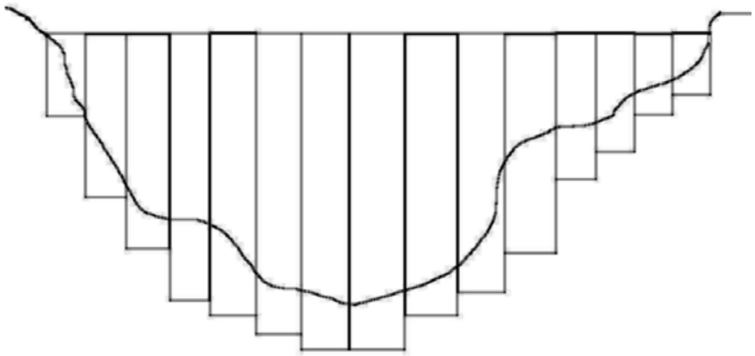


Fig 1. Blocks of a concrete dam

Figure 2shows a typical case of a concrete dam discretised with 8 nodedbrick elements considering the depth of the foundation equal to dam height and width of the foundation equal to three times the width of the dam. Typically, the dam foundation can have different material characteristics at different levels and even dam body can have different types of concretes in different parts. The variation in the material properties in foundation as well as dams can be easily taken into account by partitioning the dam and foundation in appropriate zones and assigning the respective material properties to the elements associated with the respective zones.

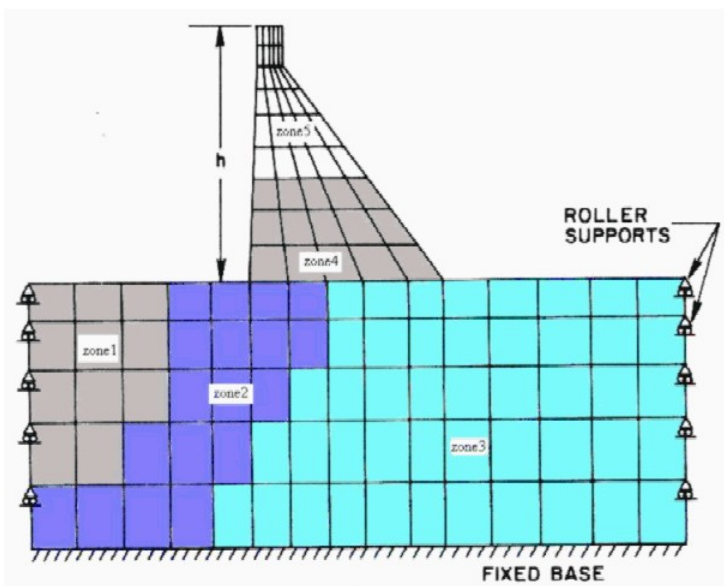


Fig 2 : A concrete dam block discretised with 8 noded brick elements

In the **Figure 2** shown above, material properties given in Zone1, 2 and 3 pertains to the rock occurring in the respective zone, whereas, material properties given in Zone 4 and 5 represent M20 and M15 concrete respectively. Though it is advised to do analysis for each block separately as 3 dimensional analysis, however, in case there is not much variation in the cross section along the length and the length is considerable with respective to the cross section, then it is possible to do simple 2 dimensional analysis for each block. However, it may result in a bit approximate solution.

2.0 Discretisationof Earth Dams

The earth dam is most economical as it uses locally available soils, rocks for its construction, but the design and analysis is more involved, as it involves materials with complex properties. Typically,an earthen dam is designed against overtopping, internalerosion and piping, cracking in the embankment and against seepage. Basically, the Earth Dam is a gravity dam and all aspects of designing of gravity dam are also applicable to earth dam. However, the earth dam posses more specific problems which are not faced by gravity dam like seepage, internal cracks and dam-foundation interaction. Finite Element analysis can be used in analysis of earth dam for following phenomena:

1. Stress analysis in the body of the dam and foundation
2. Cracking in the embankment
3. Seepage analysis
4. Slope stability Analysis, etc.

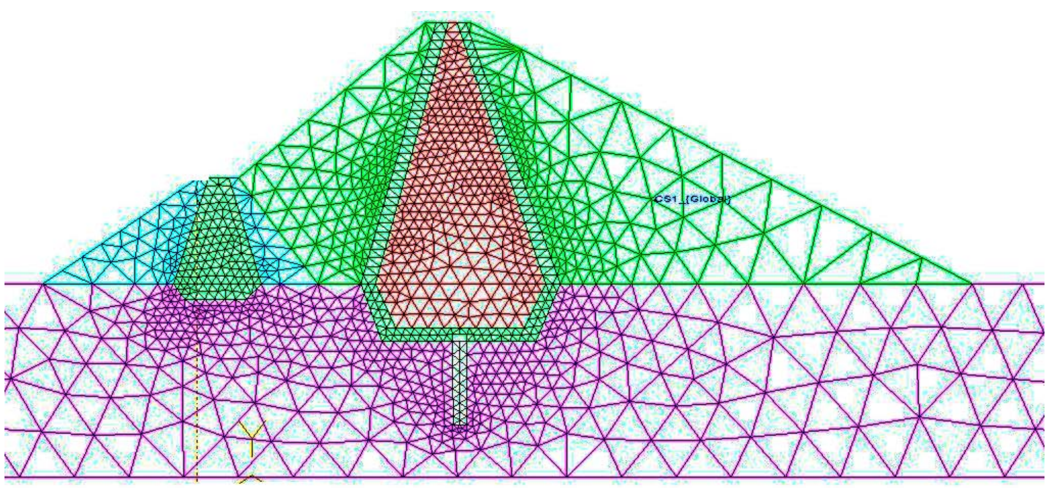


Fig 3 Discretisationfor multi core earth dam

Typically, an earth dam longitudinal structure having length dimensionsmany timesthan the cross sectional dimensions, moreover, the cross section is mostly uniform throughout the length. This continuous structure do not have joints in between. So a 2 dimensional approach give good result for earth dam. The two dimensional plain strain elements are appropriate. The 3nodedlinear elements or 6noded second order elementsgives the acceptable results, though 4 noded linear element or 8 noded second order element are preferable.

3.0 Discretisation for Arch Dams

When the valley is narrow and the abutments are stable & strong, an economical section of the dam in the form of horizontal arch can be provided . In such type of construction, the hydrostatic load is transmitted to the abutments through the arch dam. Typically, thickness of the arch is small as compared to the other dimensions. There are simple conventional methods of analyzing the arch dam considering it as apart of thin or thick cylinder. In the conventional analysis, the dam is considered to be made up of number of sectors (part of a ring) one over the above and each sector is analysed separately. Depending upon the type of construction, the dam can be analysed as constant angle arch dam and constant radius arch dam. However the results are highly approximate and counter productive to the economy being offered by the arch dam. Moreover such conventional analysis does not throw any light on the stress distribution across the dam body, foundation and abutments. The only appropriate method of analysis and design of arch dam is finite element analysis.

For designing an arch dam using FEM , the analysis has to be 3 dimensional with full details of the dam body, foundation and abutment. The geometry of the dam is also required to be reproduced accurately in the model.

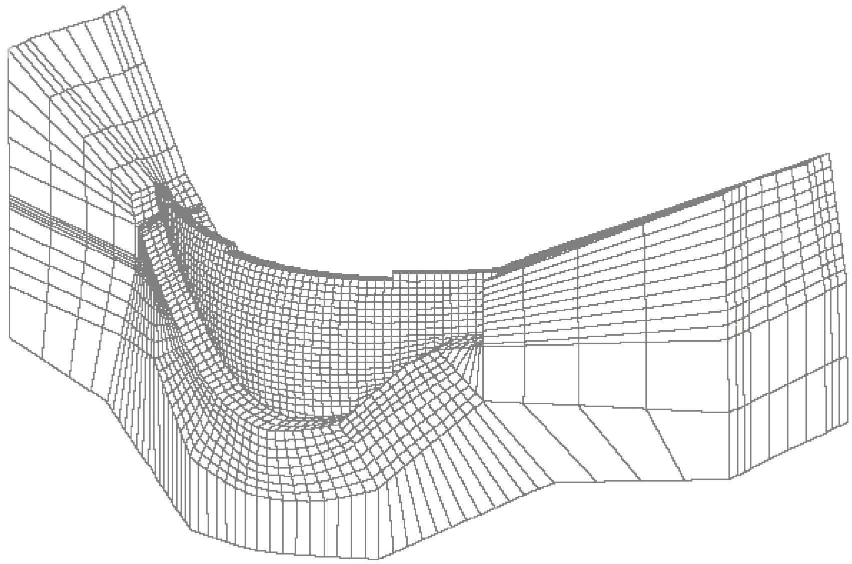


Fig 5 Arch dam Discretisation.

The 3 dimensional analysis used for analysis of arch dam have be with parabolic or cubic element so as to conform to the profile of the dam. Use of linear element may be avoided. Though, FEM makes it possible to incorporate the exact geometrical features for the foundation as well as abutments, it may lead to quite a complex geometric model, which can pose difficulty in discretisation. The tetrahedral element offers ease in meshing using the automatic meshing feature of the software. However, this element is not considered to be a good element for structural analysis as it is a stiffer element. If inevitable, only a parabolic tetrahedral should be used. However, the results should be interpreted in the light of the stiffer element.Parabolic or cubic brick elements are always preferred.

Data required for Finite Element Analysis

To conduct FEM analysis, the following design data is required:

i. **Geometry of the dam foundation, abutments etc.**: To the extent possible the smaller dimensions and intricate features can be neglected. However, bigger openings like inspection galleries, foundation galleries, sluices should taken into account in geometric modeling as they have profound effect on the stress concentrations.

ii. **Material properties**: Structural analysis of the dam requires material properties for each type of the concrete used and for different zones of the rock encountered in the foundation and abutment. The properties required are Poisson's ration, Young's modulus, shear modulus, coefficient of thermal expansion, density etc. In a model where the thermal loads are to be taken into account, the analysis requires ambient temperature, temperature within the dam body, tail water temperature etc.

iii. **Loads**: The various significant loads acting on the dam body and required for analysis are as below:

a. **Dead load of the dam**: This load is generally calculated automatically by the FEM Software using the geometric properties of the dam section and density defined for various elements.

b. **Uplift pressure**: The uplift pressure is considered to be acting along with the base width of the dam with maximum intensity at the heel and minimum intensity at toe. Profile of the uplift pressure is determined based the drainage facilities provided and the uplift intensity factor considered. The pressure with this profile is applied on the base of the dam acting in upward direction.

c. **Hydrostatic load**. The main purpose of dam is to hold water, the hydrostatic pressure acts on the dam body on its upstream face with maximum intensity at the base and with zero intensity at the free surface. The uniformly varying hydrostatic load is applied on the upstream face of the dam and also at the bottom of the reservoir. There are simple ways of applying hydrostatic loads in the standard FEM software.

d. **Earthquake force**: Design codes recommendthat an earthquake force should be computed as per IS:1893. The Pseudo-static approach for computation of the earthquake forces considers the equivalent static forces in place of the actual dynamic force. Typically for dam, earthquake forces are maximum at the top and minimum at the bottom and these forces computed as per the IS : 1893 are added to the other static forces.

e. **Silt Pressure**. The silt deposited in the vicinity of the upstream faces of the reservoir exerts lateral pressure on the dam face. This pressure depends on the depth of the silt deposited, specific gravity of the silt soil and porosity of the silt deposited.

f. **Ice pressure** : Big sheets of ice floating on the reservoir can exert large thrust on the dam body. This force depends upon the thickness of the ice sheets and intensity of the ice pressure.

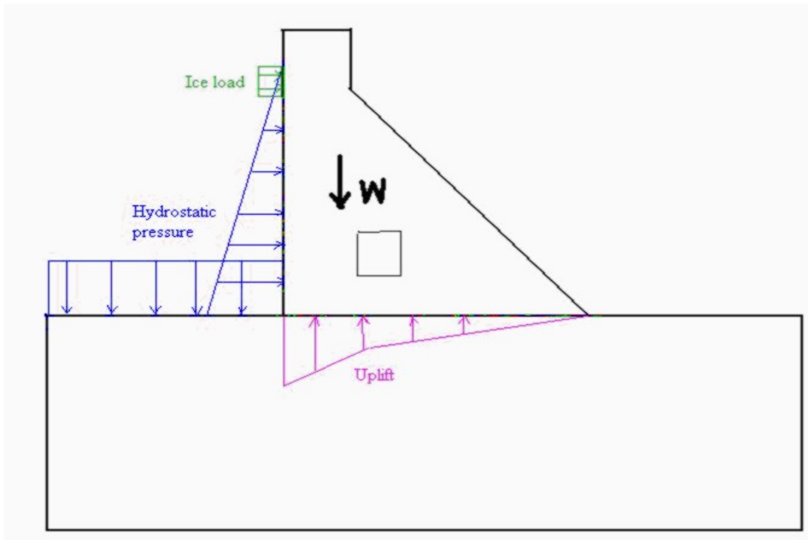


Fig5 : Loads acting on Dam

Critical load combination

Typically, the dam is analysed for various load combination of the above forces as given in the design codes. FEM analysis makes it possible to combine any load in given proportion and create a load combination. All the anticipated forces acting on the dam are created as individual loads and they are combined in various load sets for analysis.

Applying constrains and restrains

In dam analysis the bottom face of the foundation of the foundation are restrained in translation as well as rotation i.e. the X, Y, Z translation and rotation for nodes on this plane are made zero where as the side faces are allowed displacements only in vertical direction.

It is possible to use symmetry in arch dams to economize on the model if the arch dam is fairly symmetrical about the vertical plan. In such a situation instead of doing analysis for whole dam, FEM model is prepared only for ½ portion of the dam and appropriate constraint are given on the plane of the symmetry. This results in ½ number of elements in the model, thus saving on the time required for solution.

Presentation of the results

An important aspect of any finite element analysis is that of selecting and presenting essential information from the extensive results produced. It is extremely helpful to have the results presented in graphical form, both for checking and evaluation purposes. The results should contain information for the complete structure to make a judgment regarding the dam safety, as well as to determine whether the boundary conditions given were appropriate or whether there are inconsistencies in the stress distribution.

The basic results of a typical static analysis of a dam consist of nodal displacements and element stresses computed at various element locations. Nodal displacements are computed in most computer analyses and are directly available. They are simply presented as deflected shapes across selected group of elements. The stress quantities usually are plotted as stress contours (principal, von-mises, axial etc.) on the dam body. Fig 6 shows a sample result for stress distribution depicted graphically.

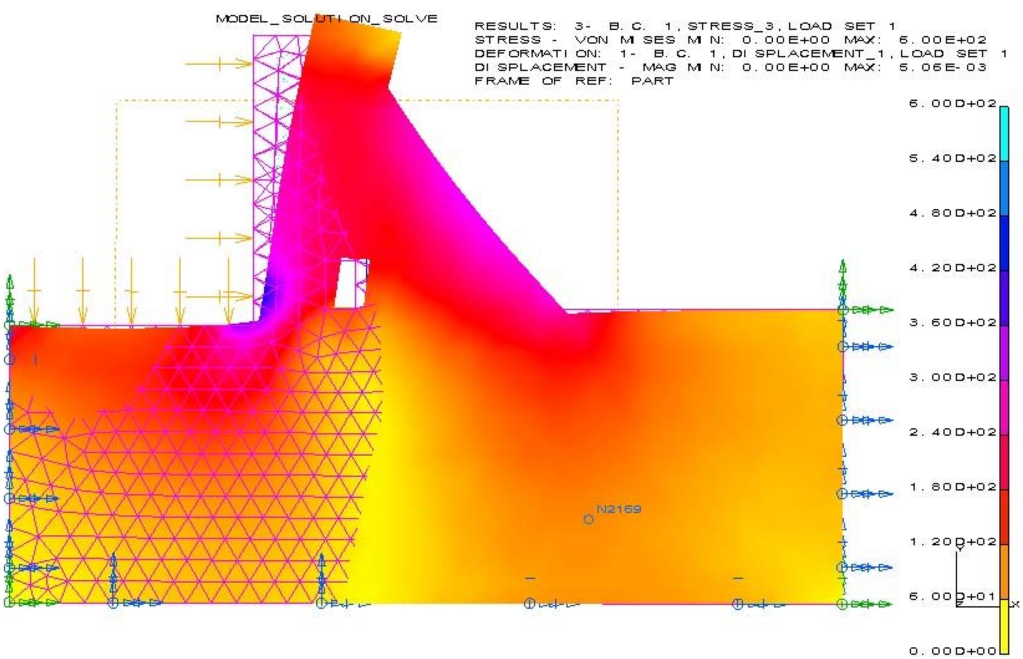


Fig6 : Presentation of results

Application of FEM in Dam analysis is not only restricted to stress analysis, in fact any problems handled by modern digital computers connected with static and dynamic analysis of complicated machines or structures are generally of the form

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = \{F(t)\}$$

Any physical phenomenon which can be described by equation similar to above equation is a possible case of analysis by FEM (e.g. Solid mechanics, heat flux, fluid flow, seepage, acoustics, magnetic/electric field etc.)

In water resources engineering, the problems represented by above equation can be classified as

i. Equilibrium or steady state Problems, where the entities given in above equation are not function of time. i.e. the problem is in the form of [K] [U] =[F] e.g.

1. Analysis of Hydraulic structures e.g. Dams, tunnels, Gates, power houses, Underground cavities, etc.
2. Construction and excavation problems
3. Soil-Structure Interaction
4. Slope stability Analysis

ii. Eigen value Problems or natural frequency Analysis where the first term of the above equation is zeroand problem is in the form of [C]{Ù} + [K]{U} = {F(t)} e.g.

5. Seiche of lakes and harbors
6. Natural frequency and modes of vibrations of structures.

iii. Propagation Problems or Dynamic Analysis where all the terms of the above equation are significant. E.g.

7. Sediment transport
8. Wave propagation
9. Seismic analysis of W.R. structures
10. Transient seepage in soil and rock
11. Dynamic soil-Structure Interaction

Though this lecture notes deals only with stress analysis (steady state) analysis of dams, the Finite Element Method can be used for all of the above analysis problems effectively.

DESIGN OF OPENINGS IN GRAVITY DAMS

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1.0 Openings in Gravity Dams:

A system of galleries, outlets /sluices, adits, chambers, and shafts within the body of the dam provides means of access and space for drilling and grouting, collecting drainage and the installation, operation and maintenance of the accessories and utilities of the dam. Following types of openings are provided for different purposes:

i. **Foundation Gallery**: It is a gallery which generally extends over the length of the dam near the rock profile , it is near and parallel to the axis of the dam.
ii. **Drainage gallery**: This is supplementary gallery sometimes provided downstream at about 2/3rd the base width from the upstream face for the purpose of draining the downstream portion of the foundation
iii. **Inspection Gallery** : Provide access to the interior mass of the dam in order to inspect the structure and study the structural behavior of the dam.
iv. **Gate Gallery**: Gallery made in a dam to provide access to or room for, the mechanical and electrical equipment required for the operation of gates.
v. **Outlet/Sluices**: Outlet is a structure in a dam to draw required amount of water from reservoir for intended purpose of Irrigation, power generation or Drinking Water safely.
vi. **Sump Well**: Sump well is provided to drain-off the seepage water, collected through formed drain in inspection & foundation galleries, out of dam body. The collected water is drained to the downstream side generally by gravity through suitable arrangements. Their number and size depends upon the quantity of water seeping through the foundation and body of the dam. Usually a sump well is also provided provided in the deepest location.
vii. **Pump Chamber:** Pumps of suitable capacity should be provided to pump-off the water collected in the sump well. As far as possible, the pumps are located in a chamber adjacent to an inspection gallery above the foundation gallery so that in the contingency of the foundation gallery getting flooded the pump-chamber remains approachable.
viii. **Elevator Tower/Stair and Shaft:** Elevator towers are generally provided at the end of spillway portion in the NOF block to provide access to the galleries from top of the dam. Generally, only a lift well is provided for which a size of 3m x 3m should normally suffice. Sometimes, a stair-well may also be provided either separately or around the lift well, if considered necessary. The size of the elevator tower should accordingly be modified to include a stair-case.
ix. **Adits:**Adits are approach tunnels to the main dam body or to the galleries. Adit to galleries should be provided for approaching them from downstream side of the NOF dam at suitable elevation above the tail water level.

2.0 Galleries

The general size of the gallery varies from 1.5 m x 2.25m to 2.0 mx2.5 m
Galleries are generally provided at a distance equal to 5% of the head or 3m whichever is more from the u/s of the dam. Minimum concrete cover between the foundation rock and the gallery is kept about 2 to 3 m.
Wherever possible seepage water collected is drained by gravity through adits provided in the dam above maximum tail water level.
Grouting and drainage gallery normally extends the full length of the dam. The gallery is located near the upstream face as close to the foundation surface as feasible but with a minimum of 2m of concrete between the floor of the gallery and the foundation. A minimum of 3m clear distanced is usually provided between the upstream face and the gallery.
In high gravity dams, a second drainage gallery is sometimes provided at about two-third of the base width downstream of the grouting and drainage gallery. The sizes of gate chambers, located directly over service and emergency sluice gates are determined by the sizes of gates and hoists.

(Ref: 12966:1992 Code of practice for galleries and other openings in dams)

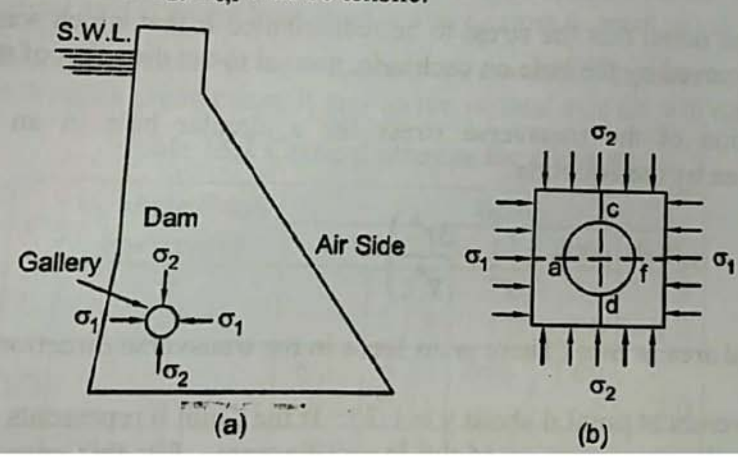
Structural Design of openings:

An opening in a structure develops discontinuity in stress field and creats zones of tensile and compressive stress. Therefore, reinforcement has to be provided for resisting tensile stresses so as to avoid the cracks in concrete near boundary of the opening.

$$\sigma_{c,d} = 3\,\sigma_1 - \sigma_2 \quad (15.8)$$

$$\sigma_{e,f} = 3\,\sigma_2 - \sigma_1 \quad (15.9)$$

when σ_1 is more than 3 σ_2, $\sigma_{e,f}$ will be tensile.



The structural design of an opening in general can be summarized in following four steps:

A. Determine prevalent stress field in the dam section at that location in the absence of the opening considering uniform stress field
B. Determination of stress distribution of around the opening

i. Therory of elasticity
ii. Stress Coefficients
iii. Photo elastic Methods
iv. Finite Element Method

C. Computation of total tension (Ft) there from
D. Computation of reinforcement for critical loading
Ast = Ft/ σst

Stress calculations:

For concrete/masonry Gravity dams with plane transverse joints which are either keyed or grouted, the Gravity method of stability analysis is used. IS 6512 is used for carrying out stability analysis and finding out the stresses in dam sections.

The Gravity method of analysis assumes a trapezoidal variation of vertical stress and a parabolic distribution of shear stress along a horizontal plane. The assumptions are substantially correct for the upper parts of the dam. However, near the base of the dam the flexibility of the rock foundation affects the stress distribution. As such Finite Element Analysis or any other comparable method of analysis is prescribed for computing the stresses at the location of opening in the dam.
The stresses are calculated for following loads and load combinations and critical of the all is used to design the reinforcement:

5.0 Loads acting on the dam

The following forces are considered for the design of dam:-

a) Dead load
b) Reservoir and Tail Water load
c) Uplift pressure
d) Earthquake forces
e) Earth and silt pressure
f) Ice pressure
g) Wave pressure
h) Wind pressure
i) Thermal load

2.0 Loading Combination:

In our country the stability of Gravity dams used to be checked for a few load combinations/forces only. BIS 6512-1984 has defined load combinations of A,B,C,D,E,F and G (table 1) as given below.

Load Combination A – Construction condition
Load Combination B – Normal Operating Condition (drains operative)
Load Combination C – Flood Discharge Condition (drains operative)

Load Combination D – Combination A with earthquake
Load Combination E – Combination B with earthquake but no ice (drains operative)
Load Combination F – Combination C but with extreme uplift (drains inoperative)
Load Combination G – Combination E but with extreme uplift (drains inoperative).

Reinforcement:

For calculating reinforcement around galleries and other openings tensile area is normally calculated using the relation

Tensile area=0.149 A σ_n

Where,

A = size of opening

σ_n = Maximum normal stress at the location of opening due to critical loads

IS code 12966-1990 part II describes in detail the basis of design of galleries and other openings in dams.

Reinforcement around gallery

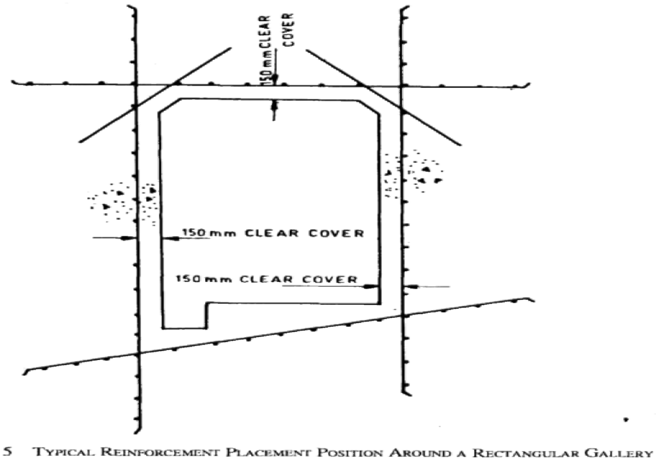


FIG. 5 TYPICAL REINFORCEMENT PLACEMENT POSITION AROUND A RECTANGULAR GALLERY

Design of Outlets/Sluices: The outlet works through the dam body of Embankment dams are usually called outlet. However, for concrete dams are usually called as conduits or sluices. Outlets / sluices are provided to release the flow when reservoir level is below crest.

Functions:

a) Irrigation
b) Power Generation
c) Water supply
d) Desilting the reservoir by draining out silt laden water

e) River diversion during construction
f) Emptying the reservoir for inspection

Components: Intake , Conveyance structure, Control structure and Terminal structure

Design of condut /Sluice: Since the conduits are meant to carry discharge from reservoir to canal or down stream river, its design is carried out in two stages.

A. **Hydraulic Design:** Before going to structural design the varios components of an outlet shall be designed such that it is able to carry design discharge considering various losses during conveyance. Also, the outlet shall hydraulically perform well without any damage to concrete and other control structure such as gate etc..

   The profile of sluice, transitions, intake, exit and air vent size of the sluice shall be designed as per IS 11485:1985.

B. **Structural Design:** The structural design shall include

   I. (i)Stresses due to dam load and hydrostatic pressure(σn)
      (ii)Stresses due to internal pressure (p)

   II. (i)Stresses due to dam load and hydrostatic pressure(σn)
       (ii)Stresses due to internal pressure (p)
       (iii)Stresses due to temperature gradient

List of BIS:

i. IS 6512-1984 Stability analysis of gravity dam
ii. IS code 12966-1990 part II Code of practice for design of galleries and other openings in dams
iii. IS 11485:1985 Criteria for hydraulic design of sluices in concrete gravity dam
iv. IS 10135:1985 Code of practice for drainage system for gravity dam, their foundation and abutments

INVESTIGATION OF GRAVITY DAMS

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1.0 INTRODUCTION:

Detailed Project Reports (DPRs) projects prepared by all concerned developers/agency are submitted to Central Water Commission (CWC) or CEA for techno-economic approval. These are examined by CWC for hydrology design, safety and cost estimates of civil structure aspects.

Besides, these DPRs need to get clearance from the Central Soil and Material Research Station (CSMRS) from construction material aspects, the Geological Survey of India (GSI) from geological aspects and Seismic aspects from NCSDP.

Presently, the Survey and Investigation (S&I) and preparation of detailed project report of hydro projects are required to be carried out as per " Guidelines for preparation of Detailed Project Reports (DPRs) of irrigation and multipurpose projects" issued by Ministry of Water Resources. These guidelines were reviewed, updated and finalised in April 2000, by a working group was constituted by Ministry of Water Resources.

With the recent technological advances, the practice of investigations has undergone radical changes. Detailed explorations may sometimes reveal adverse geological features which in turn may either lead to drastic changes in the design or even render a particular structure un-feasible. An ideal approach in phasing out the total gamut of survey and investigation of hydropower projects constitute a four stage programme viz. (i) Pre-feasibility State, (ii) Feasibility Stage, (iii) Detailed Investigation (DPR) Stage and (iv) Construction Stage.

2.0 VARIOUS ACTIVITIES OF SURVEY & INVESTIGATION (S&I)

2.1 The following activities are carried out for the various alternatives considered to justify the final choice of the location of different components of the project:

a) River surveys
b) Reservoir surveys
c) Head works surveys (dams, barrage, weir, etc)
d) Plant sites and colonies
e) Canal, branch canals, and water conductor system
f) Major canal structures
g) Power house, switch yard, surge shaft, tail race tunnel(s), adits, penstocks etc.
i) Surveys for command area including Ground Confirmation Survey
j) Soil surveys

k) Soil conservation
l) Construction material surveys.

2.2 The different activities of survey and investigations which are carried out before preparing a Detailed Project Report are given in Sub-para 2.2.1 to 2.2.4.

2.2.1 Topographical Survey:

The Survey of India (SOI) has published topographical maps covering the whole country in 1:2,50,000 and 1:50,000 scale. Besides SOI has also published topographical maps of many parts of the country in1:25,000 scale and they are already on the job of producing maps in 1:25,000 scale for the whole country.

2.2.2 **Engineering geological, geophysical, Seismological and Construction material survey**

(i) General

These investigations are now considered as a fundamental requirement of planning & design of large civil engineering structures pertaining to hydroelectric Projects. All dam sites, power house locations, tunnel alignments, major bridges etc. need to be thoroughly explored before arriving at their techno-economic feasibility. Subsurface exploration, comprising particularly of diamond core drilling and exploratory drifts are the main stay of geological investigations.

Geological investigations of hydroelectric projects are of paramount importance in understanding the geological set up of varied terrains and their geo-dynamic development. The purpose of most engineering geological work is to ensure that a proposed structure is built at the lowest cost consistent with currently accepted safety standards. The “need base” of survey and investigation for a project constitutes delineation of lithology, stratigraphy and geological structure of the area, geo-mechanical properties of the ground and identification of extraordinary phenomenon, if any. The extent of survey and investigation depends on the Stage of investigation, a common approach being preliminary and reconnaissance investigation in the initial phase while detailing and accuracy can be improved subsequent phase/stage of investigation.

Adequate engineering geological investigation is the pre-requisite for the safe and economic design of the structures. Geology dominates the feasibility, behaviour and cost of the structures. Hence, engineering geologist must be able to answer the following broad questions to have an insight to the geological conditions at any project site:-

- What is the depth of overburden mass that must be removed to reach the acceptable foundation for the structures?
- What are the rocks which make up the foundation and to what extent are they affected by surface weathering?
- What are the engineering properties of the rocks and rock masses at the foundation grade (e.g. strength, deformability etc.)?
- What are the geological structures of the foundation (i.e. joints, faults, folds, shears etc.). A full description of the defect pattern in rock mass should include orientation, spacing, aperture, extent and persistence?
- How permeable is the foundation rock?
- Whether the dam abutments are stable and there is no anticipated stability problem?

In order to be able to answer the above questions, the project site must be investigated and explored by experienced engineering geologist, using the following methods / techniques:

i. Geological mapping on large scale of surface rock outcrops
ii. Geophysical surveys wherever required to explore the depth of overburden mass and sub-surface geological conditions.
iii. Exploration by excavating trenches and pits.
iv. Exploration by drilling to know the geological conditions at different depth.
v. Exploration by drifting.
vi. Extent of work to be undertaken will depend on the complexity of geological conditions.

Key Inputs Required:

The following inputs are of overriding importance since they have significant bearing on cost, safety and useful life of surface and underground structures :

a. Geology and structural / tectonic features
b. Rock mass and intact rock properties
c. Permeability of rock mass and Hydrogeological data
d. In-situ stresses
e. Design details
f. Construction methodology
g. Stabilization measures
h. Instrumentation (monitoring) data

ii) Engineering Geological Survey

a) Surface investigations

After ascertaining the regional geology of the area the site specific geological mapping is taken up by intensive surface traverses of the project area and also with the aid of aerial photographs & satellite imagery, for coverage of inaccessible area comprehensively. Observations and measurements of the items such as nomenclature and classification of rock, stratigraphy and geological structure, properties of the ground are recorded and the data that is necessary for knowledge of the general geological condition is gathered.

b) Sub surface investigations

The direct tools include exploratory pits, trenches, drill holes, drifts, which provide a detailed information of the ground under survey. Test pits and trenches are best suited for shallow exploration on moderately steep slopes. Rotary drilling is the most extensive and common technique employed for detailed exploration to know the 11 condition of soil and rock. Water tightness of the bed rock is determined by conducting water pressure tests. These tests are normally carried out at proposed dam sites, powerhouse, other caverns and reservoir locations etc. Drifts are generally made to explore dam abutments, adit / tunnel alignments.

(iii) Geotechnical Survey

Estimation of Rock classes is done by Geomechanical classification for tunnels/ underground works and slope stability measures to evolve type & quantity of support system.

(iv) Geophysical Survey

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. Besides, resistivity measurements are also utilised for determination of the true resistivity values for the use of earthmat design of switchyard and power house areas. Vibration monitoring studies are conducted for controlled blast design and for safe excavation of major structures. Slope stability design and analysis is carried out by utilizing the inputs from inclinometer studies.

State-of-the-art computer aided seismic tomography is utilized for scanning the rockmass conditions in the dam foundation and powerhouse areas. Resistivity imaging techniques are also employed for scanning the rockmass in terms of resistivity values. Some of the widely used methods in hydroelectric project investigation are as under :

a) Seismic refraction / reflection method

b) Seismic tomography involving P-wave and S-wave measurements.
c) Resistivity imaging and resistivity sounding.
d) Micro Earthquake (MEQ) studies.
e) Vibration monitoring studies.
f) Inclinometer studies.

(v) Seismological Survey

Details of seismological events are collected from IMD, New Delhi and site specific design earthquake parameters are evaluated either by the institutes such as IIT, Roorkee or CWPRS, Pune, for working out the site specific design earthquake parameters. These parameters are further put up to National Committee on Seismic Design Parameter (NCSDP) for final approval before adopting the same in detailed design of the project. Micro Earthquake studies are also conducted for the assessment of seismicity of the area and demarcation of the active faults in and around the project area, depending upon the sensitivity & magnitude of the project.

(vi) Rock & Soil Mechanic Testing and Construction Material Survey

a) Laboratory Testing

It is conducted to evaluate the engineering properties. Some of the laboratory tests are

i) Determination of deformability of rock materials in uniaxial compression
ii) Triaxial compression tests
iii) Determination of tensile strength
iv) Determination of Direct Shear Strength
v) Rheologic properties of rocks

b) Field Testing

These include the following:

a. Rock stress determination using flat jack, over coring and hydro fracturing
b. Determination of modulus of deformation by Goodman Jack test and cyclic plate load test

Preference should be given to identify quarry sites in area of submergence (reservoir) keeping in view the environmental aspects and other related complexity of forest clearance / land acquisition, etc.

2.2.3 Hydrological & Meteorological Survey

These surveys are carried out to establish

i. Rainfall
ii. Gauge
iii. Discharge
iv. Sediments

v. Water quality
vi. Evaporation
vii. Availability of water for the benefits envisaged
viii. Design flood for various structures

2.2.4 Environment and Forest surveys

These surveys / studies are carried out on the following aspects

1. Environmental survey
2. Forest area involved
3. Likely displaced persons
4. Environment impact assessment
5. Environment management plan

3.0 STAGE OF PROJECT INVESTIGATION

The different stages of project for which investigations are carried out are:

a. Pre-Feasibility Stage
b. Feasibility Stage
c. Detailed Investigation (DPR) Stage
d. Construction Stage

3.1 Pre-Feasibility Stage

It is more of a desk study with limited field checks. Based on the 1:50000 or 1:25000 scale Survey of India toposheets, possible hydroelectric sites are marked. These sites are examined by preliminary field traverses wherein topography, broad geological aspects in terms of locating the project components are looked into. If required, broad assessment of the terrain at the likely site is also carried out by geophysical survey to understand the sub surface condition of the rocks.

Reconnaissance of the Area: *This stage of investigation forms the basis for taking up preliminary stage investigations. The reconnaissance must be carried out by experienced and competent engineering geologists to study the merits and demerits of the various sites proposed by the sponsoring agency. Later on based on the techno-economic feasibility, one site may be taken up for further evaluation by geological investigation.*
The identification and feasibility of the water resources projects, whether storage or run of the river schemes, depends on location, geology, gradient of river, hydrology and seismicity.

For example in the higher Himalaya (1000-6000 m asl), which lies in the north of the Main Central Thrust (MCT), the terrain is characterized by high river gradient (> 10m/km), less river discharge, high river velocity, steep hill slopes, high seismicity, very thick overburden of fluvio-glacical deposits over hard rocks of Central Crystalline Group. This sector is best suited for small diversion dams and run of the river schemes because of deeper rock foundation level and availability of more head for power generation due to steep river gradient, which otherwise will have less storage capacity if high dams are proposed instead.

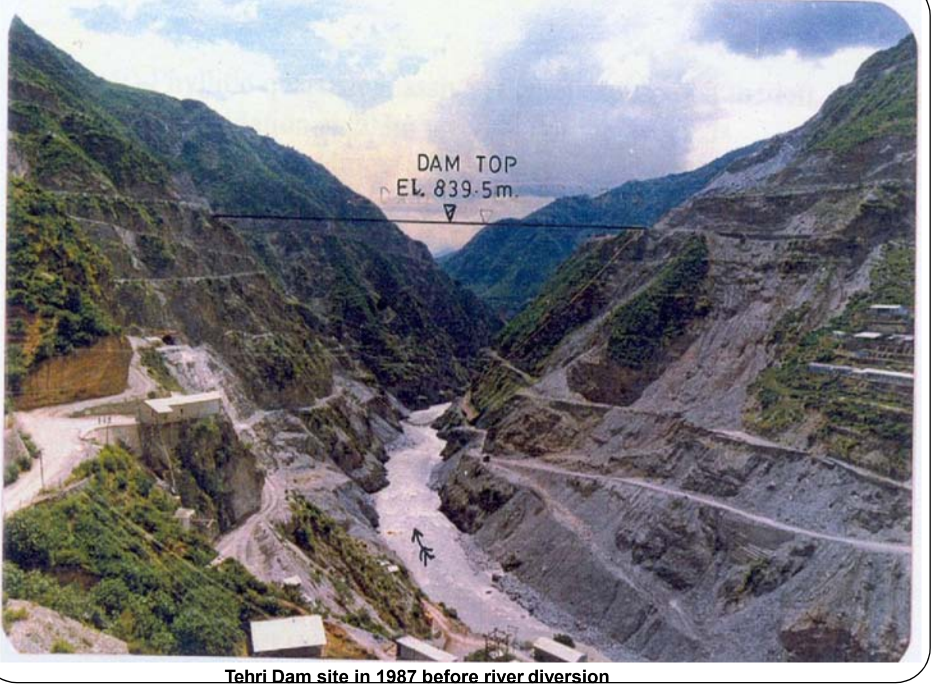
On the other hand, high storage dams are feasible in the Lesser Himalayan Terrain (400m – 1000 m asl) which is bounded by MCT in the north and MBF (Main Boundary Fault) in the south and is characterised by high river discharge, less river gradient (2-10m/km, average 5m/km) with adequate storage capacity, rock foundation level at shallow depths (as river bed deposits ate 7 to 10 m thick).

In Lesser Himalayan sector, run of the river schemes may not to be technoeconomically feasible as for achieving the required head for power generation very long head race tunnels (HRTs) will be required, due to less river gradient.

3.2 Feasibility Stage

After selecting the site during Pre-feasibility stage, intensive field traverses are under taken. Detailed survey like contour plans and sections in 1: 1000 to 1:5000 scale are prepared for various important structures such as dam, tunnels, powerhouse, etc. One or two alternate axis are probed by detailed geophysical survey and a few drill holes to ascertain depth and quality of bed rock besides carrying out broad geological mapping & collection of regional geological information. The hydrometeorological data collection is also started during this stage. Based on this data, lay-out of the project is prepared and its techno-economic viability is established.

The first consideration of the selection of a dam site is whether the topography of the river valley provides a feasible dam and reservoir site with adequate storage capacity and is in conformity with the geology and structure of foundation rocks.



Tehri Dam site in 1987 before river diversion

3.3 Detailed Investigation (DPR) Stage

Detailed geological mapping is undertaken during this stage. *These include geological investigations at the finally selected project site, covering both regional geological and local geotechnical aspects. Important geological considerations are:*

In the area around Dam Site (Including Reservoir Area): For reservoir area, construction material sites, access roads the mapping is carried out on 1 :5,000 to 1: 15,000 with 5m to 10m contour interval. Depending upon the requirements, a suitable scale is adopted within the above range, considering the size of the study area & availability of topographic plans. Geological mapping of areas of special importance in the reservoir is recommended by using 1 :5,000 to 1 :2000 scales with 2.5m contour interval. Sufficient geological data is collected to decide on the remedial measures for reservoir rim treatment and to comment on water tightness & mineral submergence. All the selected Borrow Areas & Quarry Sites are mapped on 1 :2000 and 1 :5000 scales with 2.5m and 5m contour interval respectively.

- *Study of regional geology and tectonics using existing geological maps and also latest satellite imageries of high resolution. Lineament pattern picked up from satellite images and ground validation.*
- *Identification and delineation of neotectonic features -active faults/thrust. Time-lapse measurements by geodetic surveys to record movements along active faults.*
- *Categorization of tectonic features/dislocation identified in the area in different orders based on their strike continuity and seismic status.*
- *Seismotectonic evaluation of the area to find the linkage between tectonic features and a seismic events in the area.*
- *Landslide zonation in reservoir area and study of slope stability around reservoir rim.*
- *Identification and detailed investigations of areas rich in construction materials. The investigations include geological mapping, geophysical profiling, pitting, sampling and testing.*
- *Long-term siltation studies in the reservoir area.*
- *Micro seismic investigations by establishing a net work of seismographs/accelerographs in and around project site.*

At Dam Site: *Detailed engineering geological investigations are carried out at the dam site to study rock mass behavior under different geological setting.* For dam, ancillary structures & power house it is recommended to use 1 :1000 scale with 2.5m contour interval for geotechnical assessment. *Geological sections ate prepared along axis of dam, spillway, tunnel alignment, water conductor system, power house cavities etc. depicting geotechnical information required by design engineers. The geomechanical properties and permeability values of the rock masses are determined to assess rock mass strength and reliability.*
The following are some important investigations at the project sites.

a. *Detailed Topographic Survey*
b. *Geomorphological Studies.*
c. *Geological Mapping.*
d. *Sub-Surface Explorations*
e. *Geophysical Exploration.*
f. *Drilling.*
g. *Drifting.*
h. *Laboratory Tests.*
i. *In-situ Tests.*

a. Detailed Topographic Survey

- *The entire dam site should be covered by accurate topographical surveys on 1:1000 scale using Total Station (which is a less time consuming technique), after establishing adequate number of reference pillars/bench marks. The errors in preparing topographical map may lead to discrepancies in geological mapping, planning and layout of project features.*

- *Topographical surveys on different scales are required for different project features. For dam , spillways, powerhouse etc. map on 1:500 scale are required whereas for tunnel maps on 1:5000 scale are appropriate. For reservoir area, covering hill slopes above maximum reservoir level, maps on 1:5000 to 1:10000 scale are required.*

b. Geomorphological Studies

- *The geomorphological expressions sometimes give important clues for unraveling the existence of major structural and tectonic features present in the area. The straight course of the river, sudden change in the course of the river, break in slope etc. are the expressions which indicate existence of a fault or mega shear zone.*
- *But detailed geological investigation are necessary for proving or disproving such features, as these structural features have direct bearing on the feasibility of the project and sometimes these problems may delay the decision.*
- *At Tehri dam site, a river bed shear zone was interpreted on the basis of geomorphic expression which was later (after 25 years) disproved by subsurface exploration, but caused delay in project formulation and design decision.*

c. Sub-surface Explorations

In order to unravel the depth of bed rock below the thick apron of overburden mass/slide mass, to decipher the limits of weathering/destressing of rock mass, to prove or disprove the interpreted structural features like faults.

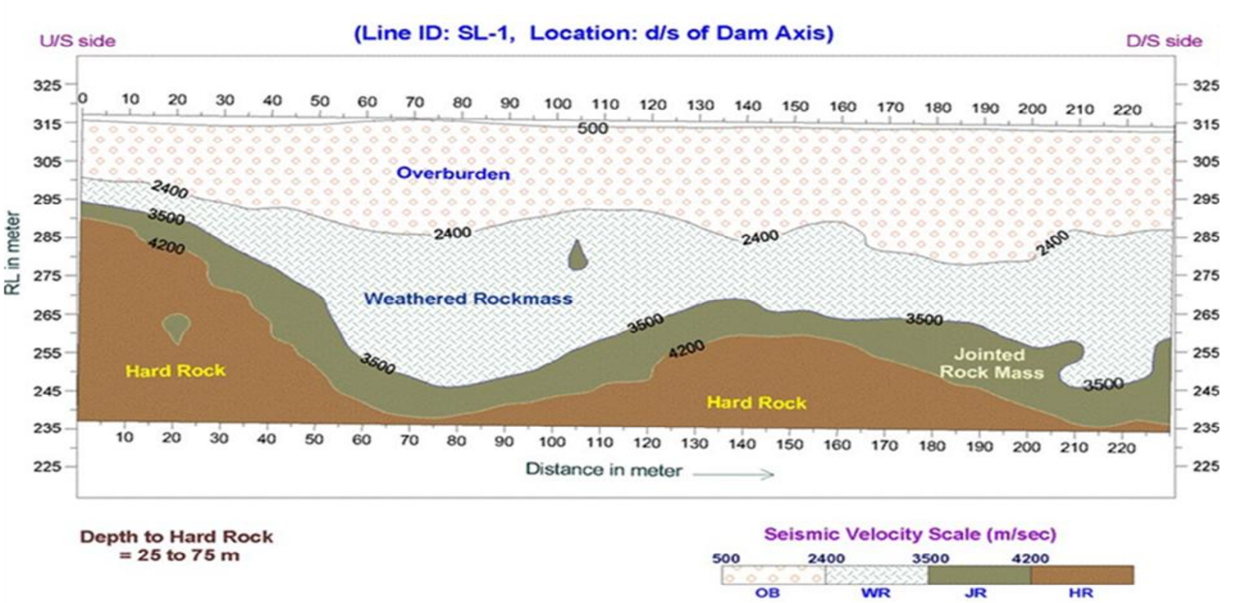
d. Geophysical Exploration

- *These are indirect methods of exploration for getting useful information related to subsurface geology of a larger area.*
- *The geophysical surveys include seismic (refraction/reflection) profiling, seismic tomography by ground penetrating radar (GPR), electric resistivity sounding and ultrasonic logging.*
- *The seismic and resistivity surveys are used for determining depth to bed rock, disposition of fault of shear zone and for assessing modulus values (modulus of deformation) of rock mass.*
- *Modulus value of rock mass can also be obtained by ultrasonic logging of drill cores. The geophysical interpretation should be substantiated by drilling and drifting.*

For delineation of subsurface structural features:

- *.Cross-hole geophysics*
- *.Cross-hole Tomography*
- *Geophysical surveys should be done intensively and extensively in order to reduce drilling quantum.*

Typical Seismic refraction survey section



A shear zone is suspected in this section.

e. ***Exploratory Drilling :*** Exploratory drilling for a dam is carried out by drilling few holes on either abutment and in the river. The holes are generally aligned along the axis or in a grid pattern depending upon the expected geological features, ascertained during geological mapping covering the base upto downstream toe of the dam. The depth of the holes depends on the geological set up and type and height of dam, but generally the holes are drilled into fresh and sound rock to the extent of 10 meters.

During exploratory drilling maximum core recovery is attempted and presence of weak and weathered seams, faults, shear zones, is deciphered.

- *Exploratory drill holes are planned at project sites with the objective of exploring quality of rock mass likely to be encountered, depth of overburden, depth of weathering / destressing, existence of shear zone / weak zone, rock cover above the proposed underground structures.*
- *It may be mentioned that drilling gives very limited information about the sub-surface geological conditions at required depths. It is a point information.*
- *Drilling is a time consuming operation and mostly the delay is due to poor and less drilling rate/day.*

f. Exploratory Drifting

- *Exploration by drifting has an edge over drilling particularly in demarcating slump zones and weathering / destressing limits.*
- *Current practice of driving drifts manually which takes more time and causes enormous delays, be replaced by mechanized drifting. Tunnel boring machine (TBM) of smaller diameter can be used.*
- *The explorations by drifting are direct and reliable. The drifts are planned at different levels to assess the actual rock mass behavior by undertaking detailed 3-D mapping 1:100 to 1:200 scale and rating the rock mass in terms of Q, RMR, etc.*
- *Drifts are also used for conducting in-situ tests to obtain information on strength and deformation properties of rock mass which are utilised in design of underground structures. The geological information obtained in regard to weathering/distressing limits for ascertaining limit of stripping to achieve foundation grade.*
- *Exploratory drifts and pilot tunnel should be provided for ascertaining geological conditions in tunnelling.*

g. Laboratory Tests:

- *The drill core samples should be tested for determining the physical and mechanical properties of intact rocks by different methods in the geotechnical laboratory.*
- *The core samples should be tested for determining specific gravity, modulus of elasticity, poisson's ratio, unconfined compressive strength (in dry/wet state) tensile strength, swelling index and hardness.*

h. In-situ Tests :

At project sites, the in-situ tests are done to determine the key parameters:

- *In-situ stresses*
- *Modulus of deformation*
- *Shear parameters*
- *Permeability*

Both the abutments are probed further by two or three level exploration drifts with cross cuts. The number and length vary depending upon the size of the dam. Nowadays, the dam base is scanned by seismic tomography depending upon the site conditions/requirements. All other tests like grouting, shear tests are also carried out. Insitu test for ascertaining bearing capacity of the foundation may be required. In some cases SPT tests may also be necessary depending on geological conditions.

At least one drill hole shall be placed specifically for energy dissipation arrangement immediately after the spillway. A provision of at least three drill holes is generally kept separately for spillway in case it is a side channel spillway, away from the main body of the dam. The location of coffer dam in the river bed portions especially for the upstream one is required to be probed by a drillhole to understand nature of overburden material and bedrock depth. Similarly, for diversion tunnel also, a provision of minimum three drill holes is kept with one each at the inlet & outlet portals and the third one suitably placed on the tunnel alignment, depending on the geological set-up and topographic considerations.

Exploratory drilling for water conductor tunnels (HRT/TRT) is carried out to establish rock cover available above the proposed crown level of the tunnel especially in low cover zones such as in the beds of stream and to know the substrata along the proposed alignment of the tunnel. Drill holes are proposed at the intake and portal sites at outlet and along proposed alignment of the tunnel. These holes are drilled to the proposed invert level of the tunnel. It would be desirable to probe the expected tunneling media to the extent possible. But, in view of the high rock cover above the tunnel alignment, normally drilling of holes down to tunnel grade is a difficult task. Hence, majority of the tunnel forecast is based on detailed geological map & projections. However, certain major weak zones such as faults or thrusts etc should be probed by advance core drilling during the tunneling activity to identify the exact condition and to minimize geological uncertainties.

Tunnel Portals are mapped using 1: 500 scale with 2m contour interval instead of 1: 1 000. For Adits / Access Roads, Borrow Areas / Quarry Sites all construction as well as access adits are mapped on 1 :2000 scale with 4m contour interval. Vulnerable areas on access roads are also mapped on the above scale. As such, all geological maps are updated in this stage. If there have been certain changes in the layout, additional areas are covered.

For powerhouses, sub-surface exploration is carried out by holes drilled at least 5m below the proposed level in order to assess the nature of the 17 rock for the foundation of the power house if rock cover is less. Similarly, drill holes for ascertaining the geology of surge shaft and pressure shaft etc. are also carried out.

Permeability tests should be conducted in all the drill holes selective pits & trenches. Geological logging of Drill Holes, Drifts, Trenches & Pits is carried out using 1:100 scale.

Groutability tests to assess the grout intake and reduction in permeability values are generally done in this stage of investigation. Generally the rock mechanic or soil tests are carried out in the DPR stage for detailed design. Moreover, some of the detailed & expensive tests like mortar bar test for construction material survey and insitu rock stress measurements for design input are done at this stage. The seismotectonic studies are pursued in detailed manner during this stage for getting site specific earthquake parameters.

3.4 Construction Stage

Engineering geological investigations are continued during construction of project feature. This is done to ascertain actual rock mass conditions exposed at the foundation grades of the structure and to adopt minor changes in design, if need be, depending upon the variations recorded in geological and structural features of the foundation grade. During construction stage, detailed geological mapping (on 1:200 to 500 scale) of foundation of dam and spillway is carried out depicting lithological units, structural discontinuities and rock defects like faults, shear zones, weak zones. These rock defects are properly treated for making the foundation monolithic and impervious

In case of underground works especially for tunnels, major identified grey ares are probed by advance drilling or drifting to minimize geological uncertainities.

4.0 OPTIMISATION OF METHODS FOR SURVEY AND INVESTIGATION OF HYDROPOWER PROJECTS

An ideal method in optimising the investigation for hydropower projects emerge after due consideration of its ability to solve technological complexity of the area within the stipulated cost, time effectiveness, approachability and reliability of the practice. Some of the important practices are highlighted below.

4.1 Wider induction of remote sensing data in geological reconnaissance

Aerial photographs and satellite imagery are used for rapid interpretation of geomorphology, lithology, structure and preparation of land use thematic maps which are widely used in environmental impact assessment of the project.

4.2 Utilization of advance tools of positioning and analysis particularly in hostile terrain

Geographical Information System (GIS) is used for mapping and analyzing things that exist and events that happen on earth. GIS technology integrates common database operations and statistical analysis. The process of making maps by GIS is much more flexible than traditional manual or automated cartography approaches. Existing maps are digitized and computer compatible information are translated into GIS. Remote sensing using sensors such as cameras carried on aeroplanes, GPS receivers, or other devices collect data in the form of images and provide specialized capabilities for manipulating, analyzing, and visualizing these images.

5.0 CONCLUDING REMARKS

The cost and feasibility of the project is dominated by geology and geological complexities – which are generally inadequately explored. Hence, critical attention must be given to the prospective use of state-of-the-art techniques of geological / structural data collection, geophysical surveys, exploration by drilling, and in- situ / laboratory testing followed by numerical modeling. These are required to avoid the cost and time overruns.

The investigation programme, planned in different stages, must provide adequate geological and geotechnical data required for assesment of rock mass condition and also to identify potential geo-hazards that may exist at the project site. Information obtained from geological mapping, sub-surface explorations and rock mechanics investigation is of great help in finalization of project layout, design and construction methods; and thus in cost saving.

ANNEXURE-I

1. TOPOGRAPHY- The location and layout of the projects largely depend upon the topography of the project area. The detailed survey for preparation of the contour and L-section as required for the different components must be done. CWC has published guidelines for the scale and intervals to be adopted for the preparation of the contour plans and other topographical investigations for surveying different components of the hydropower/ multipurpose projects. Some important features are tabled below:-

Sl no.	Component	Extent of survey	Scale	Contour interval
1.	Dam	Grid plan with covering the area upto 250 m upstream and 500 m downstream of the axis extending upto an elevation of MWL + 5 m or more depending upto the site conditions.	1:2500	0.5m - 1.0
2.	Powerhouse, switch yard, surge shaft, tailrace etc.	Contour plan of the site to cover full area of the components of the various alternative layouts.	1:2500	0.5 - 3 m
3.	Tunnels & adits	Contour plan of the area covering the length of the tunnel & 500 m on the either side of the centre line of the tunnel/ adit.	1 : 2500	1m - 3m
		L -section	horizontal 1:2500 Vertical 1:100 to 1: 1000	
4.	Penstocks	Contour plan of the area covering the length of the structures and 150 m on either side of the centre line of penstocks.	1 : 2500	1m - 3m
		L -section	horizontal 1:2500 Vertical 1:100 to 1: 1000.	

ANNEXURE-II

Minimum Pattern of Drilling

Spacing of drill holes/Pits/Drifts	Depth of drill holes/Pits/Drifts
(a) Earth and Rock-fill Dam 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 60m 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 (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.	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.
(ii) 2-3 drill holes down stream of spillway.	10m deep in fresh rock or equal to maximum height of dam in absence of rock.
(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).

(c)Tunnels	
(i) Drill holes at each of the portal and adit sites and additional at-least one every 1-5 km interval depending on the length of tunnel.	Drill holes 5-10 m below the tunnel grade of maximum possible depth. Wherever it is not possible to drill along the central line of the tunnel the holes can be shifted.
Drifts one each at the portal and adit sites	The explorations shall be so planned as to satisfactorily portray the geological structure and tunneling conditions. Drifts shall be extended up to 10 m in fresh rock or up to tunnel face.

List of BIS Codes:

i. IS 4453:2009 Subsurface exploration by pits, trenches, drifts and shafts-Code of practice
ii. IS 4464:1985 Code of practice for presentation of drilling information and core description in foundation investigation

MATERIAL PROPOERITIES, TESTIMG AND DIVERSON ARRANGEMENT FOR GRAVITY DAMS

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A. MATERIAL PROPERTIES AND TESTING:

INTRODUCTION:

Any water resources development project involves interaction between the super structure and its foundation. The stability of the structure is hugely influenced by characteristics of foundation medium. The foundation strata may vary from loose sandy soils to dispersive, expansive soils to hard rock. The characteristics of foundation need to be evaluated for safe and economic design of the proposed structure. Numerous direct and indirect techniques are available for determining the material properties.

A gravity dam is keyed into the foundation so that the foundation will normally be adequate if it has enough bearing capacity to resist the loads from the dam. If, however, weak planes or zones of inferior rock are present within the foundation, the stability of the dam will be governed by the sliding resistance of the foundation. The following strength parameters for use in stability and stress analyses:

a. Bearing capacity (compressive strength).
b. Shear strengths along any discontinuities and the intact rock.
c. Deformation Modulus of the rock mass.
d. Hydrostatic pressure in rock joints.

These parameters are usually established by laboratory tests on samples obtained at the site. In some instances, in situ testing may be justified. In either instance, it is important that samples and testing methods be representative of the site conditions. The results of these tests will, generally, yield ultimate strength or peak values and must, therefore, be divided by the appropriate factors of safety in order to obtain the allowable working stresses.

Foundation borings and testing can be helpful in identification of weak zones in the foundation beneath the dam. The presence of such weak zones can cause problems under either of two conditions: (1) when differential displacement of rock blocks occurs on either side of weak zones and (2) when the width of a weak zone represents an excessive span for the dam to bridge over. Sliding failure may result when the rock foundation contains discontinuities and/or horizontal seams close to the surface. Such discontinuities are particularly dangerous when they contain clay, bentonite, or other similar substances, and when they are adversely oriented. Appropriate uplift pressures must be applied to failure planes in foundations. Associated with stability are problems of local over stressing in the dam due to foundation deficiencies.

Foundation permeability tests may be helpful in conjunction with the drilling program and to design an appropriate drainage system. Permeability testing programs should be designed to establish the permeability of the rock mass and not an isolated sample of the rock material. The mass permeability will usually be higher, due to jointing and faulting, than an individual sample.

The structural features which are suspected to be indications of poor foundation conditions are listed below:
a. Low RQD ratio (RQD = Rock Quality Designation).
b. Solution features such as caves, sinkholes and fissures.
c. Columnar jointing.
d. Closely spaced or weak horizontal seams or bedding planes.
e. Highly weathered and/or fractured material.
f. Shear zones or faults and adversely oriented joints.
g. Joints or bedding planes described as slickensided, or filled with gouge materials such as bentonite or other swelling clays.
h. Large water takes during pumping tests.
i. Large grout takes.
j. Rapid penetration rate during drilling.

Compressive - In general, the compressive strength of a rock foundation will be greater than the compressive strength of the concrete within the dam. Therefore, crushing (or compressive failure) of the concrete will usually occur prior to compression failure of the foundation material. When testing information is not available this can be assumed, and the allowable compressive strength of the rock may be taken as equal to that of the concrete. However, if testing data is available, the safety factors should be applied to the ultimate compressive strength to determine the allowable stress. Where the foundation rock is non-homogeneous, tests should be performed on each type of rock in the foundation.
Shear - Resistance to shear within the foundation and between the dam and its foundation depends upon the zero normal stress shear strength (cohesion) and internal friction inherent in the foundation materials, and in the bond between concrete and rock at the contact surface. Ideally, these properties are determined in the laboratory by triaxial or in the field through insitu testing.
The possible sliding surface may consist of several different materials, some intact and some fractured. Intact rock reaches its maximum break bond resistance with less deformation than is necessary for fractured materials to develop their maximum frictional resistances. Therefore, the shear resistance developed by each fractured material depends upon the displacement of the intact rock part of the surface. The shear resistance versus normal load relationship for each material along the potential sliding plane should be determined by testing wherever possible.

The foundation parameters of are assessed under geological parameters and geotechnical parameters. The dam foundation requirements are based on type of dam proposed and is largely dependent on the strength, deformation and permeability characteristics of site material. To determine the depth of excavation needed to achieve an adequate foundation, observation in borings and test pits, field testing of soil and rock, laboratory testing of representative samples and ultimately, analysis/design is needed.

The geotechnical properties of foundation can be assessed through several surface and sub-surface methods of investigation. Geophysical investigation includes seismic refraction method, electrical resistivity survey and imaging techniques, tomography and ground vibration techniques. These geophysical techniques are used as a tool for qualitative assessment of subsurface strata and stratification of bed rock.

Geological investigations are carried out by surface maping, drill holes, exploratory drifts. The properties of intact rock may be determined by subjecting the rock specimens to various tests in dry and saturated conditions in the laboratory. The strength and deformability characteristics of rock mass may be obtained from insitu direct shear tests and uniaxial jacking and Goodman jacking tests.

In situ stresses- In situ stresses play an important role in safe and economic design of structure created inside of rock mass. In situ stresses are rarely uniform in a rockmass. Underground works require knowledge of in-situ stresses. Many methods are available for determination of magnitude of principal stresses.

Empirical correlations for assessing the rock mass quality: In the absence of test data, some empirical correlations developed from time to time are also used for assessing the rock mass quality.

Intact Rock and Rock Mass: When studying the subject of rock mechanics, the terms Rock Mass and Intact Rock are generally used. It's important to distinguish between rock mass and intact rock.

a. **Intact Rock:** The term intact rock refers to rock which has no through going fractures significantly reducing its tensile strength. It is usually characterized by density, deformability Young's modulus and Poisson's ratio) and strength (unconfined compressive strength, cohesion and angle of friction).
b. Rock Mass: The term discontinuity is used in rock engineering for all such types of fractures to indicate that the rock is not continuous unlike the intact rock described above which is mechanically continuous. Clearly the nature, location and orientation of discontinuities profoundly affect most of the rock properties (deformability, strength, permeability, etc.) and, therefore, the rock engineering application. The rock mass means in-situ material, which consists of intact rock, joints and other discontinuities. The

greater the strength of intact rock, the more important is the discontinuities in determining the behaviour of rock mass.

2. Rock/Rock Mass Characterization

The rock mass is characterised by conducting in-situ testing along with testing in the laboratory. The objective of both the tests is to classify the rock for engineering design purposes. The characterization of rock and rock mass is essential to study the behavior of foundation materials on which the structure has to rest or which has to support the rock loads in case of underground caverns and tunnels.

Many researchers and rock mechanics experts have developed empirical correlations between rock mass classifications viz. RMR, Q, GSI with the engineering properties of rock mass like UCS, modulus of deformation, shear strength parameters etc. over a period of time. But these have limitations of limited data base and that too pertaining to certain specific regions only.
Moreover, empirical classifications of rock mass are also based on the judgments which contain human errors and limitations. Therefore, nothing can replace the actual test results. However in the event of non-availability of actual data, these may be helpful in preliminary design of any structure. The same may be validated later on based on actual testing and design may be reviewed.

2.1 Laboratory Investigation and Characterization of Rock

Properties of intact rock may be determined by subjecting the rock specimen to various laboratory tests. Tests conducted on the rock cores. These rock specimen can either be the cores obtained from drilling or the rock fragments. The following are the common type of tests performed in the laboratory:
1. Density (Grain and Bulk), Water Absorption, slake durability index,
2. Point Load Strength Index
Engineering Properties in Saturated State
3. **Uniaxial compressive strength (UCS)**
4. Elastic parameters: modulus of elasticity and Poisson's ratio
5. Triaxial compression test for c and ø
6. P and S Wave Velocity
7. Indirect tensile strength
8. Point Load Strength Index
9. Slake durability index

The following tests are being conducted frequently on the rock cores.

2.1.1 Strength (in Uniaxial Compression) and Deformability Characteristics

The saturated specimens (with length to diameter ratio 2.5 to 3) uniaxial compressive strength is evaluated, along with its deformability characteristics (modulus of elasticity and Poisson's ratio). In saturated condition, when loaded in uniaxial compression, the stress and strain are determined for Nx size rock specimens. The longitudinal and lateral strains of the specimen are measured with the help of electrical resistance strain gauges. Strain gauges are cemented at the middle of the specimen. The rock specimens are loaded - until failure - axially between the platens in a Universal Testing Machine. The stress value at failure is given by the relationship.

$\sigma_c = F/A$

Where,

σ_c = Compressive Strength of Specimen (MPa)

F = Applied force at failure (N) and

A = Initial x-sectional area of the specimen transverse to direction of force (mm2)

2.1.2. Tensile Strength

Specimens under compression often fail due to development of tensile stresses. Tensile failure is also an important phenomenon in drilling and blasting of rocks, failure of roof and floors etc. Tensile strength is measured directly by applying tensile load or indirectly under compression or in bending test. Due to difficulty of gripping the specimen and applying load parallel to its axis, indirect methods of estimation of tensile strength are commonly used. Several techniques for direct as well as indirect methods have been developed each having their own pros and cons. The indirect method in which compression load is applied diameteraly on a circular disc developed in South America known as Brazilian test is considered easy and reliable. Assuming homogenious, isotropic and linearly elastic material, the tensile stress developed at the centre of the disc when failure initiates is given by the following equation:

$T = P/ \pi. r. t$

Where, P : applied maximum compressive load,

r : radius and

t : thickness of the specimen.

2.1.2. Triaxial Compression Test

Tests on unconfined rock specimens were found to give an incomplete explanation of rock behaviour in situ. The conventional triaxial compression tests are carried out on rock core samples of Nx size, with length to diameter ratio of 2, to find out the shear strength parameters. Saturated specimens are tested in a Hoek's Triaxial Cell. The triaxial cell is an apparatus, in which the test specimen is enclosed in an impermeable flexible polyurethane membrane, and is placed between two hardened platens, one of which is spherically seated.

There is an arrangement for applying constant lateral fluid (oil) pressure to the specimen in the triaxial cell. After enclosing the specimen in the triaxial cell, the cell is placed in a compression-testing machine, and the specimen, under constant lateral hydraulic pressure, is loaded axially to failure. The specimens are tested under different lateral/confining pressures.
The normal stress at failure versus confining pressure (i.e., Strength Envelop), comprising all the tested specimens, is plotted. The modified failure envelope, which is the 'best-fit' of the tested data, is drawn to estimate shear strength parameters. In the sense of Coulomb's failure theory, the apparent cohesion (C) and the internal friction angle (Φ) are computed using the following formulae:

$\Phi = \arcsin [(m-1) / (m+1)]$

$C = b [(1- \sin \Phi) / 2 \cos \Phi]$

where,

m = slope of the modified failure envelop, and

b = intercept of the failure envelop on the axial stress axis

2.1.3. Point Load Strength Index

The Point Load Strength Test is intended as an index test for strength classification of rock materials. It may also be used to predict other strength parameters with which it is correlated, for example uniaxial tensile and compressive strength.

2.2 Field investigations and characterization of <u>rock mass</u>

Rock mass is a discontinuous, non-homogeneous and anisotropic geological medium containing fissures, fractures, joints, bedding planes, folds, shear seems and faults. The strength of rock mass is governed by the behaviour of these discontinuities and planes of weakness. The discontinuities may exist with or without gouge material. The infilling material in these discontinuities also varies. The frequency of joints, their orientation with respect to the engineering structures and the roughness of the joint have a significant impact on stability of the structures. Determination of rock mass characteristics is a great challenge e.g. difficulty in preparation of undisturbed specimen, limitation of testing methods, volume of rock mass involved, no. of major discontinuities involved, limitation of testing equipment, human errors, etc. However, compared to laboratory testing, in-situ testing represents the actual rock mass since it takes into account the effect of discontinuities, scale effect, natural conditions etc. Evaluation of these parameters on the basis of laboratory tests on tiny intact rock specimens or through indirect empirical relations may be often misleading and can result into inappropriate design.

It is necessary to conduct in-situ deformability and in-situ shear tests as it is impractical to simulate field conditions in laboratory. Deformability characteristics (deformation modulus and modulus of elasticity) are generally obtained by conducting the uniaxial jacking tests and recording the load-deformation data inside a drift or in a trench as per the site conditions.

Deformability characteristics can also be obtained by conducting borehole jack tests inside the drillholes. Determination of the shear strength parameters (cohesion and friction angle) for concrete to rock and rock mass involves the shearing of the block under constant vertical stress at the concrete to rock and rock to rock interface respectively. In-situ shear tests conducted either inside the small drift or in open trench take into account the effect of discontinuities; orientation of foliation or bedding planes and other joints.

Strength and deformation characteristics of rock mass containing the discontinuities may be found by conducting field tests. Field tests are essential for shear strength and deformability characteristics of rock mass. The following are the common tests employed to characterize the rock mass:

1. **Shear Strength parameters c and Φ by in-situ shear tests for**
    a. rock to rock and
    b. concrete to rock interfaces
2. **Modulus of deformation of rock mass by Uniaxial jacking tests or borehole methods**
3. In-situ Stress Measurements
4. Permeability test
5. Groutability test
6. Rock Bolt Pull out test
7. Instrumentation

In-situ shear strength and deformability characteristics of rock mass have been discussed in detail here.

2.2.1 Shear Strength Parameters: Shear strength parameters are required for design of any structure on rock. Shear strength parameters involve two components viz. cohesion (c) and friction angle (Φ). The shear failure may occur in two ways i.e. either in concrete/rock interface or within rock mass.

Methodology and Test Procedure

Blocks of rock mass are cut for rock over rock interface with no or minimal disturbance and concrete blocks are cast for concrete over rock interface for testing purpose in accordance with IS 7746:1991. During the preparation, separate the rock mass of block size (70cm x 70cm x 35cm) from parent rock are prepared in drift or trench. Similarly, after leveling the rock surface, the concrete block of 70cm x 70cm x 35cm are prepared by using steel frame. All the blocks are cured for 28 days before shearing.

For testing purpose, a set of minimum five blocks each of rock over rock and concrete over rock has to be prepared. Each block is sheared at constant but different normal load. Vertical and horizontal shear loads are applied by hydraulic jacks of respective adequate capacity. From the side reaction pad, the horizontal shear load is applied at an angle of 15^0 with the horizontal so that the resultant force passes through the center of the test block.

During the test, the shear force and the corresponding vertical, horizontal and lateral displacements of the block are measured by dial-gauges of 0.01 mm least-count. The observations are recorded even after the failure to the extent possible to get the information regarding residual frictional resistance.
Each test block is sheared till failure and beyond for establishing the peak and residual shear strength parameters as per IS 7746:1991. After completion of the test, block is overturned to measure the actual area under shear and to know the failure pattern.
The peak shear stress and residual shear stress both are plotted against the normal stress and using linear regression analysis 'best fit envelope' are drawn. From the equation of straight line obtained, the intercept on the Y- axis gives cohesion 'c' of the rock mass and the slope of the line gives the friction angle 'Φ' of the rock mass.

2.2.2 Deformability Characteristics Of Rock Mass

Commission of Terminology of the International Society for Rock Mechanics (ISRM) defined deformation modulus of rock mass in 1975.
Modulus of elasticity or Young's modulus: The ratio of stress to corresponding strain below the proportionality limit of a rock material.
Modulus of deformation of rock mass: The ratio of stress to corresponding strain during loading of a rock mass, including elastic and inelastic behaviour.
Modulus of elasticity of rock mass: The ratio of stress to corresponding strain during loading of a rock mass, including only the elastic behaviour. The modulus of rock mass is denoted in terms of deformation modulus rather than modulus of elasticity, because rock mass contain joints and during loading permanent deformations occur because of closing of the joint spaces.

Deformation characterisation of rock mass is required for load transfer from the proposed structure, be it foundation of any water retaining structure or any other civil engineering structure or heavy machinery or an underground tunnel/cavern. The response of the foundation strata due to cyclic pressure needs to be studied. The deformation viz. elastic or plastic needs to be analysed for better understanding of the rock mass behaviour. A number of methods exist for the determination of modulus of deformation of rock mass. The following in situ tests are conducted for determination of modulus of deformation:

- Plate loading test,
- Plate jacking test,
- Goodman Borehole jack test,
- Flat jack test,
- Cable jacking test,
- Radial jack test, and
- Dilatometer test.

List of BIS Codes for Laboratory Tests:

- IS:9143-1979: Method for the determination of unconfined compressive strength of rock materials
- IS:9179-1979: Method for the preparation of rock specimen for laboratory testing
- IS:9221-1979 Method for the determination of modulus of elasticity and Poisson's ratio for rock materials in uniaxial compression
- IS:10050-1981 Method for determination of slake durability index of rocks
- IS:10082-1981 Method of test for the determination of tensile strength by indirect tests on rock specimens
- IS:10782-1983 Method for laboratory determination of dynamic modulus of rock core specimens
- IS:12608-1979 Method of test for hardness of rock
- IS:12634-1989 Method of determination for direct shear strength of rock joints
- IS:13030-1991 Method of test for laboratory determination of water content, porosity, density and related properties of rock materials
- IS:19047-1991 Method for determination of strength of rock materials in triaxial compression
- IS:14396 Method for laboratory testing of argillaceous swelling rock

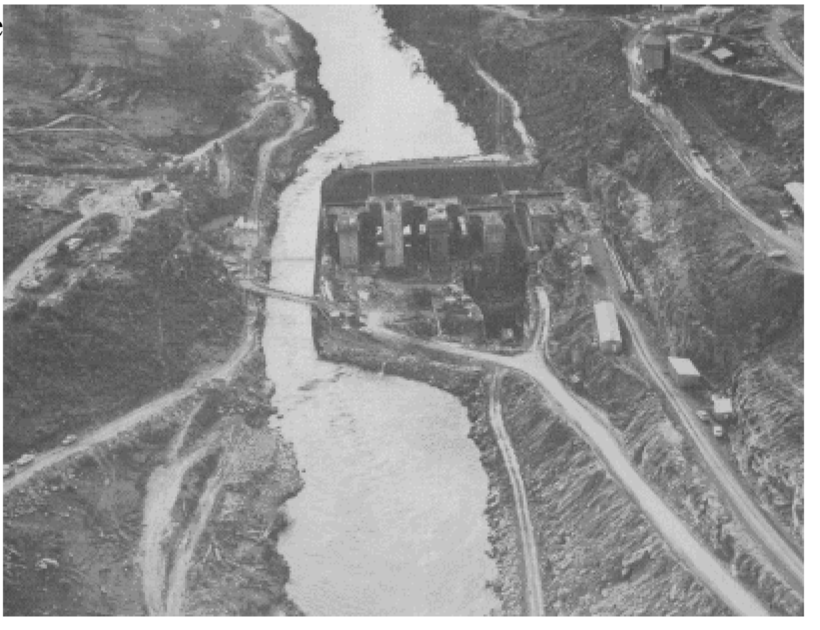
(PT.1)-1996 Part 1- method of sampling, storage and preparation of test specimens (PT.2)-1996 Part 2 - Determination of maximum axial swelling stress
(PT.3)-1996 Part 3 - Determination of axial and radial free swelling strain
(PT.4)-1996 Part 4- Determining axial swelling stress as a function of swelling strain

- IS14436-1997 Method of test for laboratory determination of resistivity on rock specimen
- IS:8764-1978 Method of determination of Point Load Strength Index for rock material
- IS:2386(PT.8)1963Methods of test for concrete aggregate - Petrographic examination
    - IS 7317 (1993): Indian Standard code of practice for Uniaxial Jacking test for modulus of deformation of rock
    - IS 7746 (1991): Indian Standard code of practice for In-situ Shear test on rock

B. DIVERSON ARRANGEMENT FOR GRAVITY DAMS

Introduction:

Prior to commencement of the actual construction of any work in river bed, it becomes obligatory to exclude temporarily the river flow from the proposed work area during construction so as to provide dry or semi dry work area. Regardless of the type of dam, it is necessary to de-water the site for final geological inspection, for foundation improvement and preparation, and for the first stage of dam construction. The magnitude, method and cost of river diversion works will depend upon the cross-section of the valley, the bed material in the river, the type of dam, the expected hydrological conditions during the time required for this phase of the work, and finally upon the consequences of failure of any part of the temporary works.



At most sites it will be necessary to move the river whilst part of the dam is constructed; this part will incorporate either permanent or temporary openings through which the river will be diverted in the second stage. If the first diversion is not large enough the initial stages of construction will be inundated, if the second stage outlets are too small, the whole works will be flooded.

Diversion During Construction:

Engineers must de-water the river where the dam is meant to be built. This is done by diverting the river through a system of temporary diversion dam and tunnel/channel that runs away from the intended construction zone. Diversion dams do not generally impound water in a reservoir; instead, the water is diverted through an artificial water course, channel or conduits/tunnels.
River diversion Scheme essentially consists of:

a. Coffer dam(s) built across the part or full width of the river to divert the flowing water away from the work area.

b. Tunnel or channel works to transfer the diverted water from upstream to downstream of the work area without affecting it.

Types of river diversion:

1. Scheduling part of the work area for construction and allowing the river to flow through the remaining area. Temporary diversion with laterally constriction of the river is done **by building a cofferdam**.

2. River diversion trough tunnel: The river diversion through tunnel has great applicability in embankment dams but it has the highest cost.
3. River diversion trough conduits: Alternative to tunnel diversion when surrounding rock has no enough quality to make a tunnel.
4. River diversion trough channel: When it's not economically unfeasible to carry out a tunnel- this solution can use where the topography is characterized by flattened valleys.
5. Opening left in the dam body: This method use in concrete dams, more especially arch dam.

Scheme of Diversion varies from project to project depending on various factors. Cost and economics of the scheme mainly decides over the available choice. A good & reliable diversion scheme is a pre-requirement for successful completion of the Project.

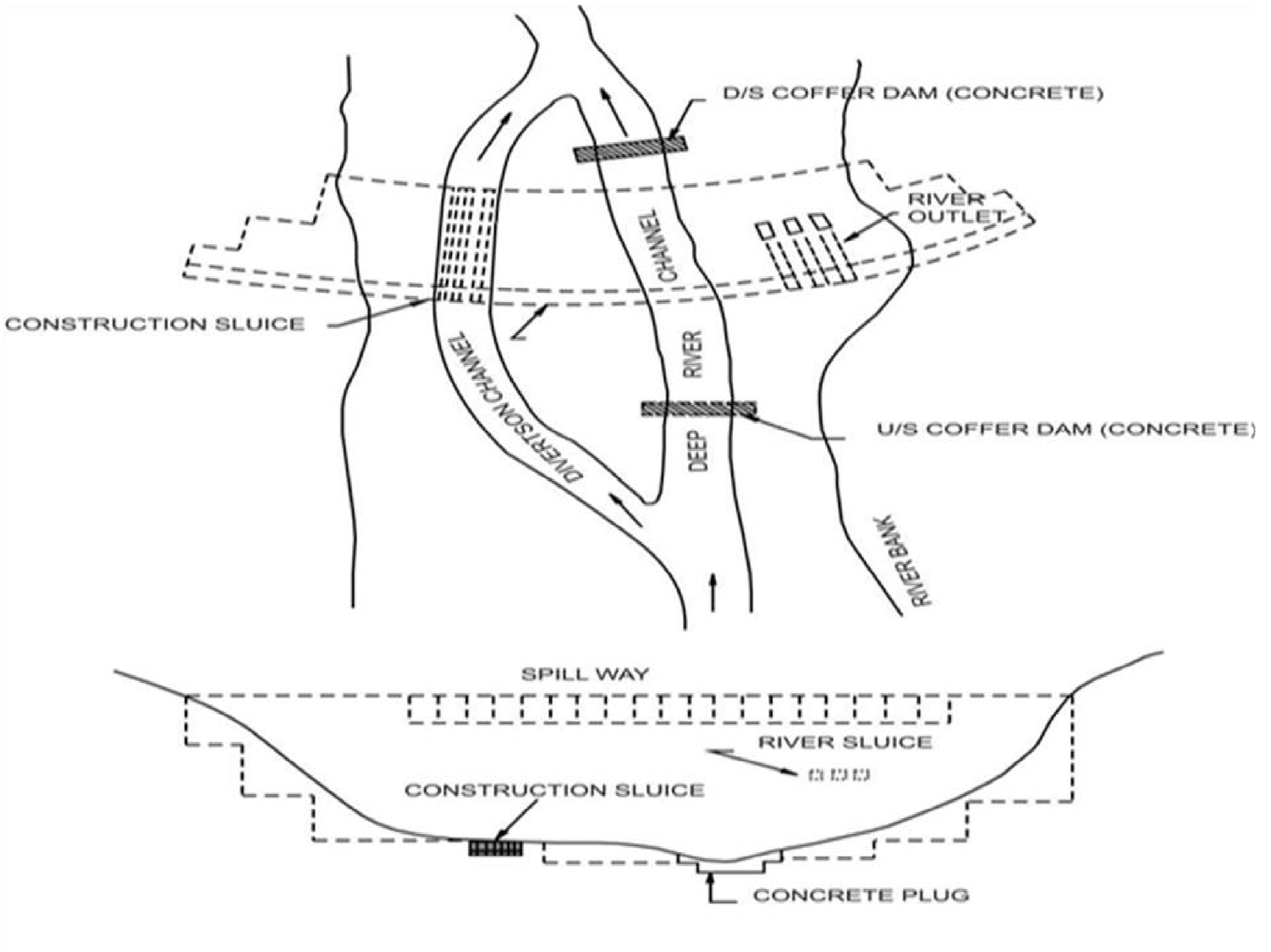


FIGURE 40. DIVERSION CHANNEL FOR CONCRETE DAM IN A WIDE RIVER

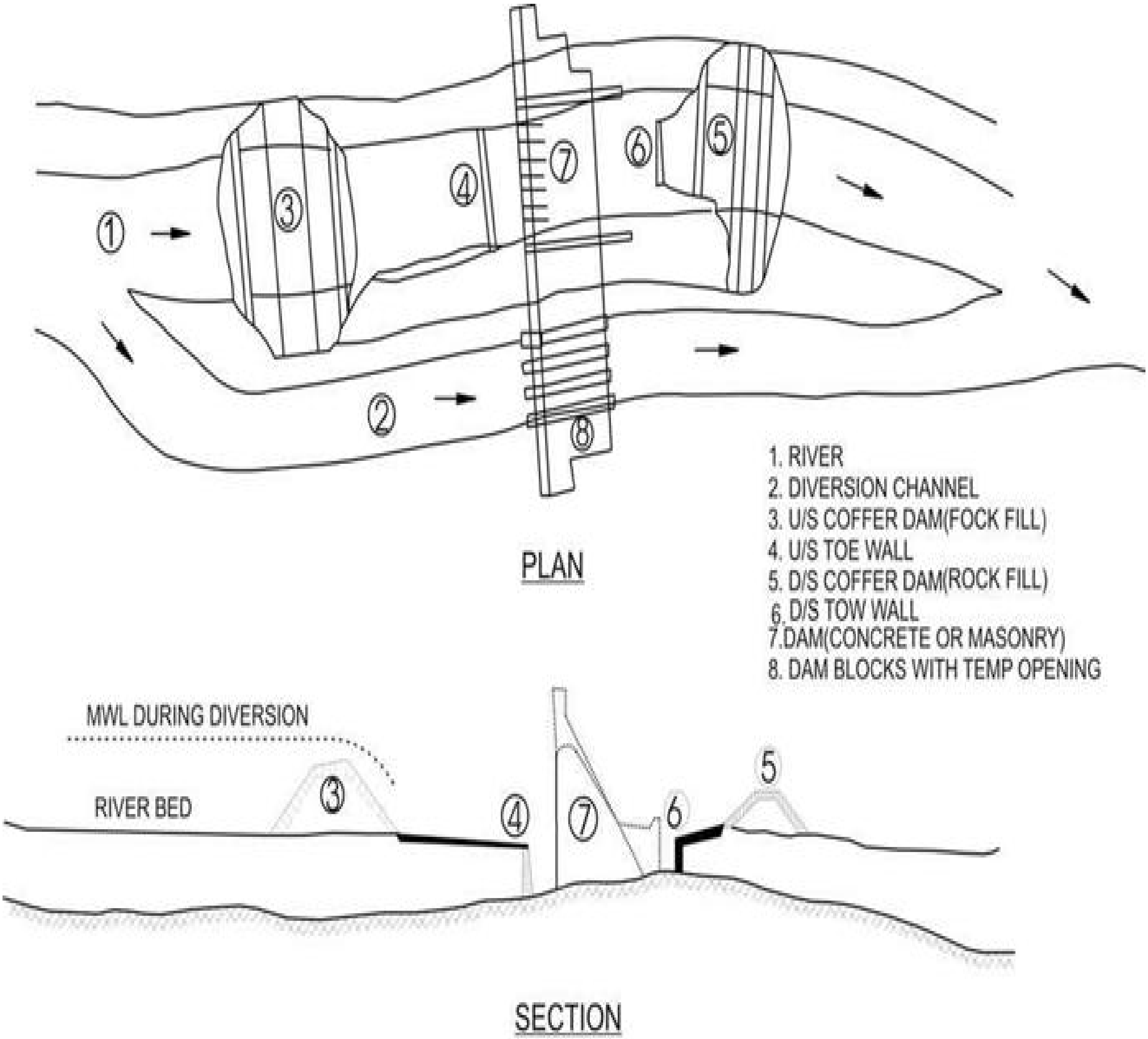


FIGURE 41.DIVERSION CHANNEL FOR CONCRETE DAM IN A NARROW RIVER

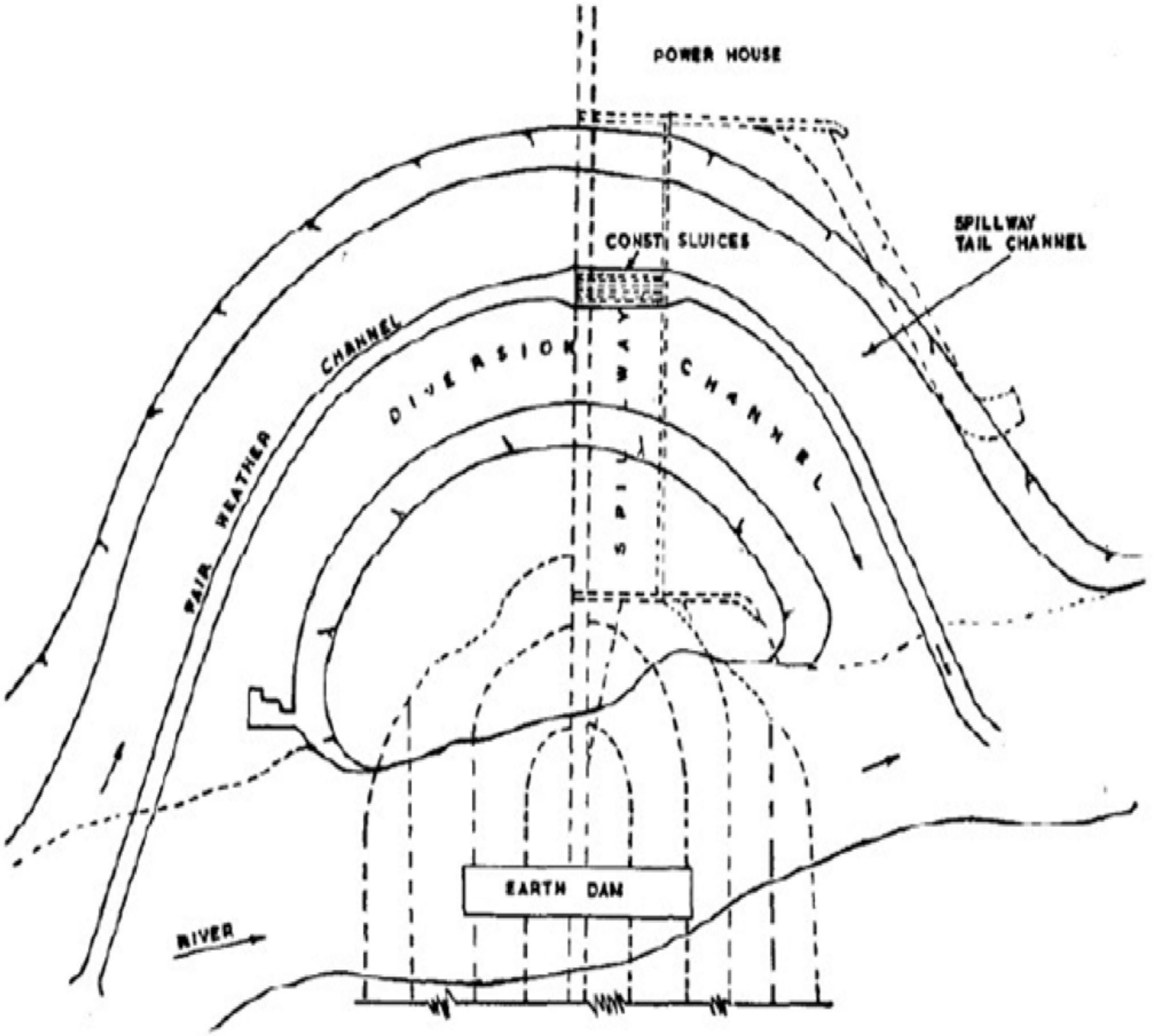


FIG. 3 DIVERSION CHANNEL FOR EARTH/ROCKFILL DAM IN A NARROW RIVER CHANNEL

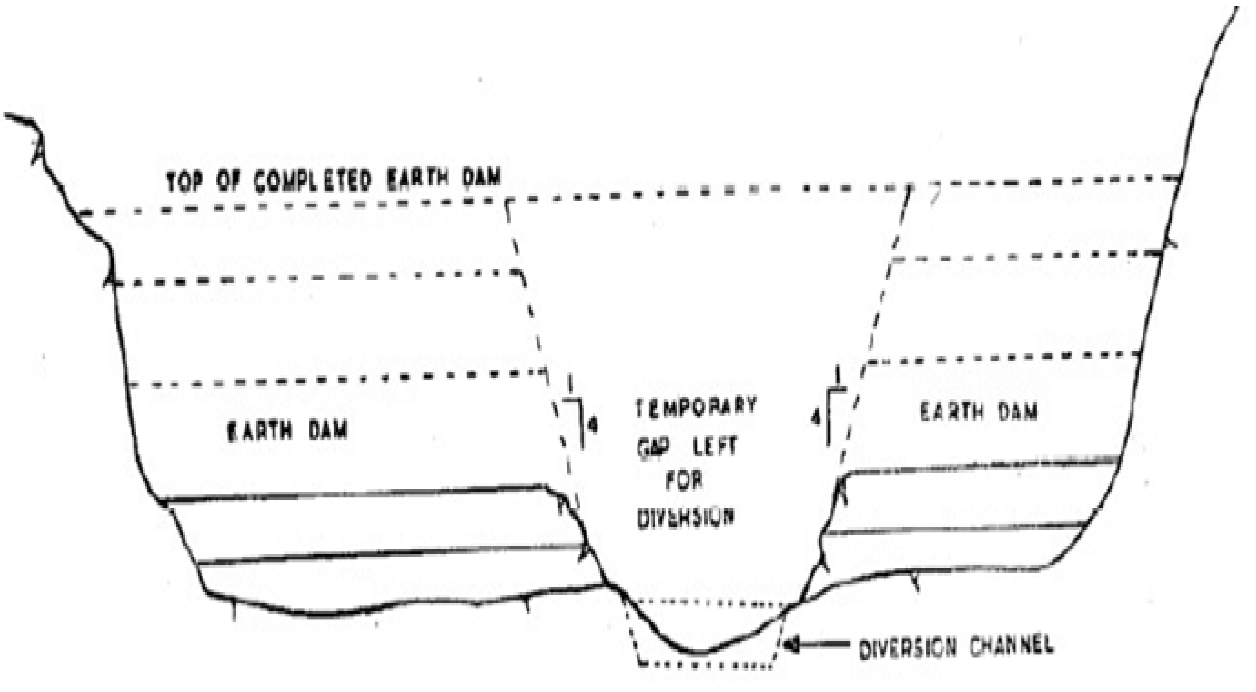


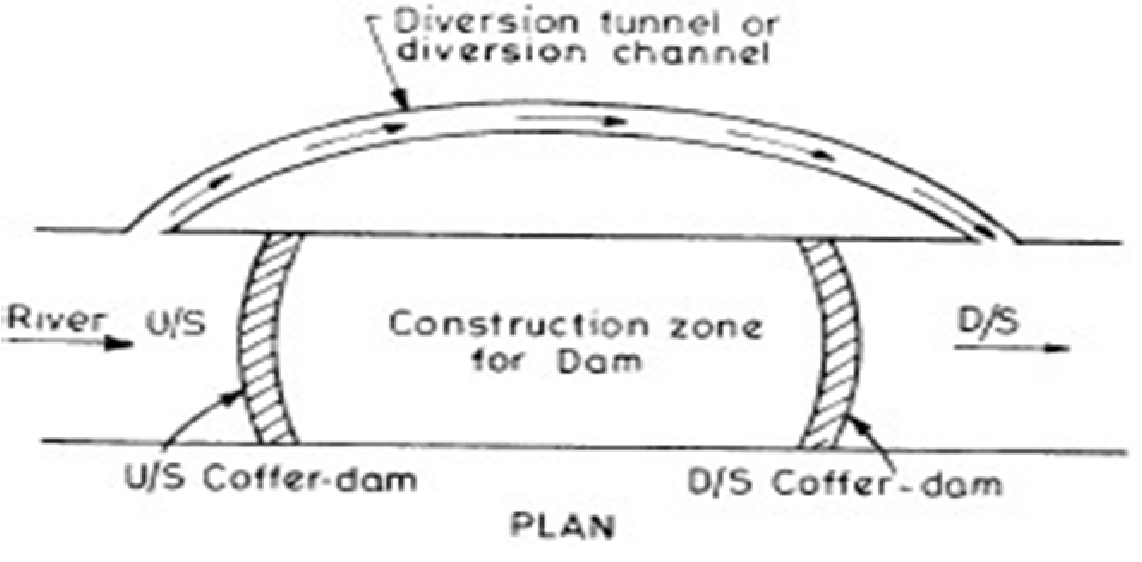
FIG. 4 DIVERSION THROUGH A GAP IN THE EARTH DAM IN A WIDE RIVER CHANNEL

Types of Diversion Methods: Generally two types of diversion arrangements:

I. Single Stage Diversion:
II. Multi Stage Diversion

I. Single Stage Diversion: For Narrow valley

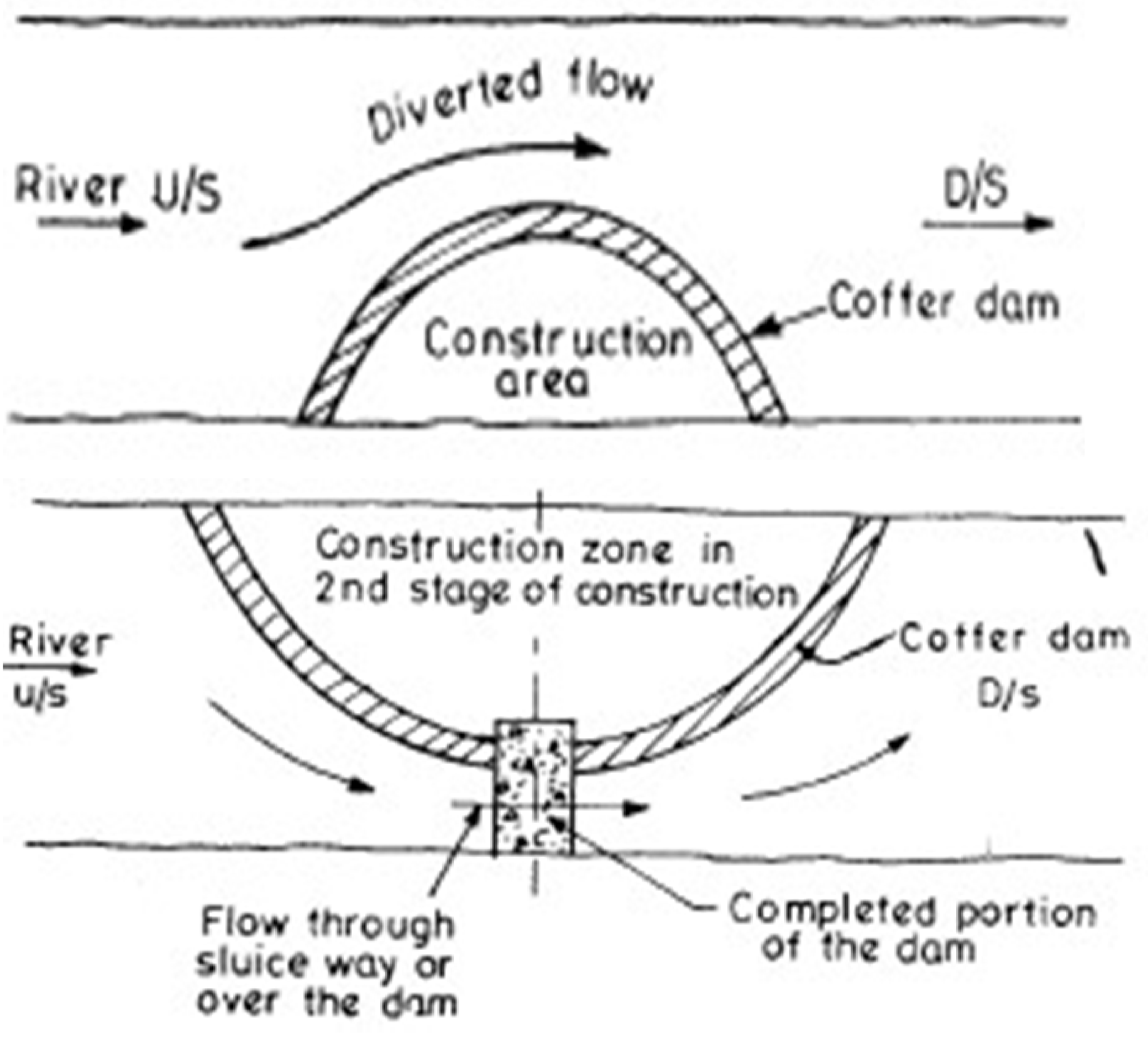
1. Build a partial coffer dam allowing construction of diversion tunnel, culverts flumes ad control works
2. Divert the flow through these structures.
3. Build full size of u/s & d/s coffer dams to proved dry working area for main dam .
4. Build the permanent works (main dam, spillway etc)
5. Close the diversion passage and start impounding the water.



II. Multi Stage Diversion: For wide valley

Diversion has to proceed in stages as follows:

a. Build a 1st stage coffer dam around a portion of work
b. A portion of dam is constructed within this area with sluices to take river flow during subsequent stage of construction.
c. After reaching the height of main dam where completion can be permitted after flooding, the coffer dam is removed to allow the flow though the sluices.
d. The remaining section of the river is enclosed within a 2nd coffer dam.
e. The sequence a to d may be repeated for completing the construction of dam over entire width of river.



Closure of Diversion Works:

Final closure of diversion work is an important stage in diversion work. Closure must take place under low-flow period.

- For small Head and flow- putting stoplogs in prepared slots.
- For large flows- by dropping bulkhead gate/sliding gate of steel

Once the gates are lowered and permanent concrete plug can be casted.

Diversion Flood:

The diversion structures work process need to guarantee that the river can bypass the dam site during construction. Design of this river diversion needs study of hydrological characteristic of the water course and morphological characteristic of the site.

The efficient scheme of diverting the river flow away from the work area should aim at limiting the seepage into the work area to minimum. The proper planning and design of such temporary diversion work would be greatly influenced by design flood in addition to other factors.

Generally the largest observed non-monsoon flood or non-monsoon flood of 25 year return period is adopted as a diversion flood.

Design Flood for Coffer dam:

Concrete /Masonry Dam = 1in 25 years non-monsoon return flood
Earthen/Rockfill Dam = 1in 100 years return flood

Fixation of Height of Coffer dam and Size of tunnel:

Flood routing and hydraulic model studies for evolving suitable arrangement of the diversion through open channel help in:

a. Discharging capacity (Numbers and size) of the diversion outlets/conduits.
b. Height of the upstream and downstream coffer dams. (MWL is known from flood routing, Height= MWL+Free Board)
c. Deciding the most efficient alignment of the diversion channel
d. Protection measures for the coffer dams if they are to be overtopped
e. Flow conditions in the diversion channel depending on its utility during diversion as well as during permanent stage.

Selection of Diversion Arrangement:

A concrete or masonry dams could be allowed to get overtopped during floods when construction activity is not in progress. Generally the largest observed non-monsoon flood or non-monsoon flood of 25 year return period is adopted as a diversion flood.
Advantage is also taken of passing the floods over partly completed dam or spillway blocks, thereby keeping the diversion channel of relatively smaller size than that is required for embankment dam. In such a case a small excavated channel either in the available width of the river or one of the banks of the river proves to be adequate. Construction sluices are located in such excavated channels which allow passage of non-monsoon flows without hindrance to the construction activity.
Such sluices are subsequently plugged when the dam has been raised to adequate height. If the diversion channel is excavated on one of the river banks, it is possible to use the same for locating an irrigation outlet, a power house or a spillway depending upon the magnitude and purpose of the project

Types of Coffer dams

a. **In situ Concrete / Masonry coffer dam**
b. **Earthen coffer dam**
c. **Rockfill coffer dam**
d. **Steel coffer dam**
e. **Timber Coffer dam**

Design criteria of masonry Coffer Dam: The dead load, upstream and downstream water loads, uplift, etc. considered in the design of coffer dam calculate according to IS:6512-1972. Forces considered in Design concrete coffer dam:

i. Dead load
ii. Hydrostatic pressure, including velocity head
iii. Uplift pressure
iv. Earth and Silt pressures
v. Reaction of foundation

Requirement of stability:

i. The coffer dam shall be safe against overturning
ii. The coffer dam shall be safe against sliding
iii. The unit stresses developed in the masonry or concrete of the coffer dam or In the foundation material shall not exceed the permissible values
iv. Resistance against Overturning: following criteria must be satisfied
    a. The resultant of all forces shall normally fall within the middle third of the base.
    b. The maximum compressive stresses at any point shall be within permissible limit.

Earthen Coffer Dam:

Sufficient freeboard against overtopping due to anticipated flood shall be provided. Seepage flow through the Dam body and its foundation to be controlled to avoid damages due to piping,sloghing and removal of finer particles . Stable slopes both at u/s and d/s of the coffer dams under all conditions of construction

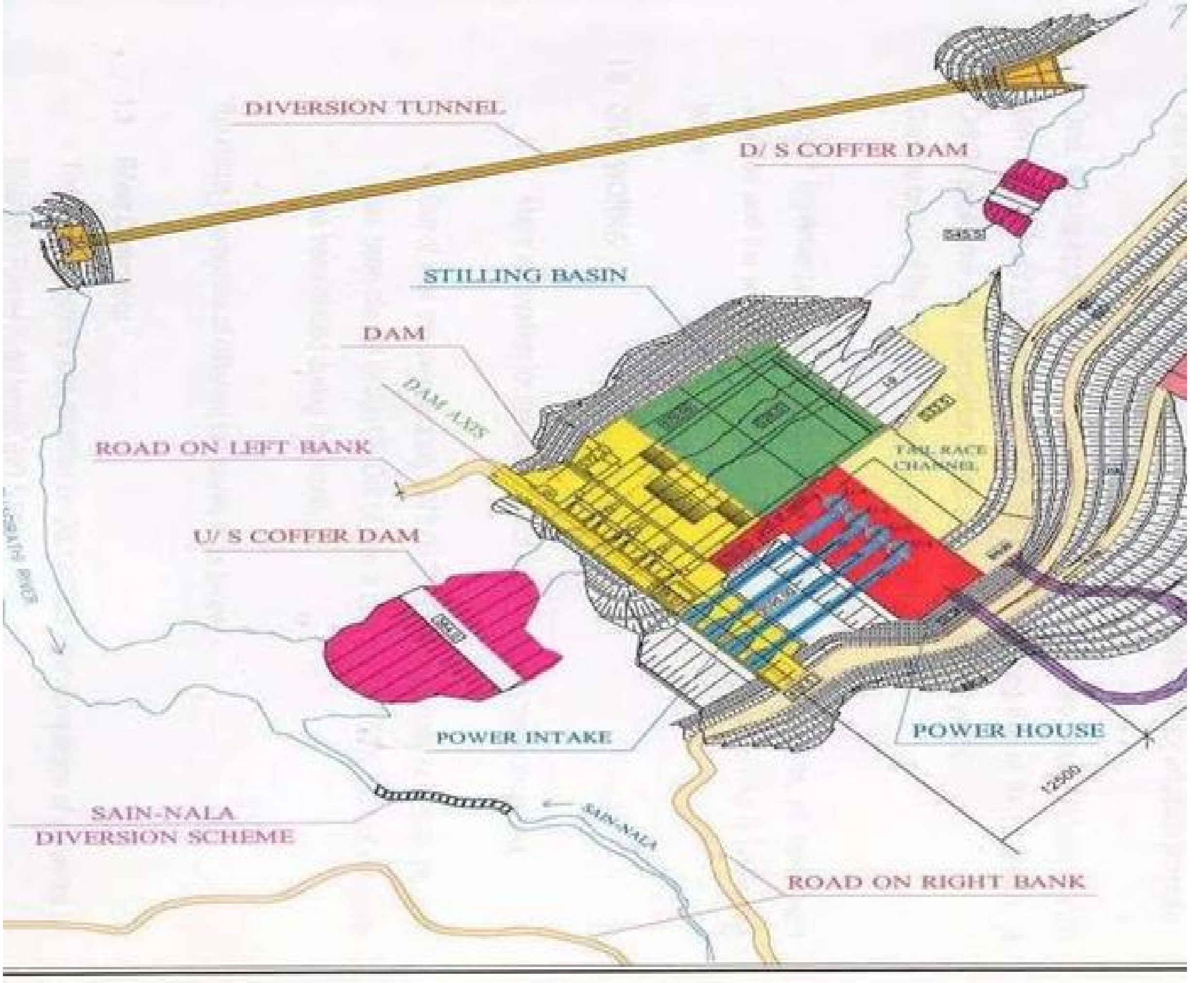
For minimizing seepage through the dam body an impervious central earth core and/or an impervious upstream membrane is usually provided. Adequate provision for passage of diverted river discharge should be made. Design and analysis of earthen coffer dam is carried out in accordance with IS:7894 & IS:8826

Diversion Tunnel:

Diversion Tunnel carries water from the mainstream to other channels or across the construction site. These can be permanent or temporary structures. The size of a tunnel depends upon magnitude of diversion flood and height of u/s coffer dam (higher the head smaller the size of the tunnel), Economics of height versus diameter of tunnel shall be worked out. A tunnel can be pressure or free flow type. Permanent closure of the tunnel is done by placing concrete plug in the tunnel.

Design considerations of Channel:

Channel are designed on the basis of manning's formula. The velocity in the unlined section should ot exceed 5m/s. in lined channels velocity may go up to 15m/s. For the diversion channel excavated in overburden, it is also necessary to ensure that the banks are not eroded due to flood flows.



KOTESHWAR HE PROJECT – LAYOUT PLAN

List of BIS Codes:

i. **IS 14815:2000 Design code for river diversion works-Guidelines**

AN INTRODUCTION TO RCC DAMS

1 INTRODUCTION

The term Roller-Compacted concrete (RCC) describes concrete used in the construction process, which combines the economical, and rapid placing techniques used for fill dams with the strength and durability of concrete. RCC is concrete with a no-slump consistency in its un-hardened state that is transported, placed and compacted using fill-dam construction equipment.

The RCC is a material composed by the same components that made a conventional concrete (cement, admixtures, sand, grave, water and Additives) but is transported, extended and compacted with the own technology of the earth's movement.

The properties of hardened RCC are similar to those of traditionally-placed concrete. Material properties such as elastic modulus, Poisson's ratio, coefficient of thermal expansion, and unit weight are similar to those of traditional dam concrete since they depend to a great extent on the aggregates used.

There are four main requirements for an RCC to be used in a water-retaining structure and two additional factors that should be considered. The four requirements are: impermeability, density, strength and most important the ability to be transported, spread and compacted without detrimental segregation. The two other factors are: durability, should the RCC be exposed to the environment and not completely protected by encapsulation by facing concrete or some other protection, and construction conditions

Generally, shear strength along the horizontal joints between the layers is more critical because of the "layered" method that is used in the construction of RCC dams. In addition, because of the construction technique, temperature distributions and the corresponding thermal stresses in the dam are very different from those of a traditional concrete dam. This is one of the major design considerations and is often investigated using finite-element method (FEM) analysis

The interest in RCC dams is driven by economic considerations and also where speed of construction is an important element. Given an adequate foundation, RCC dams commonly have a lower cost than the equivalent fill dams when the savings in cost of diversions and spillways are taken into account.

Although well-designed RCC dams are frequently the least-cost solution when compared to other forms of dam, there are conditions that can make RCC dams more expensive. Situations where RCC may not be appropriate include those where aggregate material is not reasonably available, the foundation rock is of poor quality or not close to the surface, where foundation conditions could lead to excessive differential settlement, or where the valley is very narrow and steep-sided leaving limited room in which to manoeuvre the equipment.

Requirements for galleries, instrumentation, and appurtenance for RCC dams are similar to those of traditional concrete dams. Nevertheless these features can impact on the construction programme. To ensure that RCC construction can proceed rapidly, thereby ensuring that low unit prices for the RCC are obtained, the design of the inserts in the dam in particular should be such that they have the minimum possible effect on rates of concrete placement.

2 DESIGN CONCEPTS

The lean (low-cementitious material content) RCC dam, with a low cementitious (i.e. Portland cement and mineral admixtures) content (< 100 kg/m3) The low-cementitious RCC dam uses an upstream watertight membrane to protect the low-cementitious roller-compacted interior concrete that is usually fairly permeable, particularly at the joints between the layers. This membrane can either be an immersion-vibrated concrete facing (up to 500 mm wide) placed at the same time as the interior concrete and cast against conventional formwork, or pre-cast concrete panels with or without an attached geo-membrane. Bedding mixes (concretes or mortars with a higher cementitious content) are frequently placed between each lift near the upstream face to improve the bond and reduce seepage between the layers of RCC.

The high-paste (high-cementitious material) content RCC, with a relatively high cementitious content (> 150 kg/m3).

The design philosophy of the high-cementitious content RCC dam is that the roller-compacted interior concrete should be the watertight barrier. Thus the RCC has to be designed to bond layer to layer and to have an in-situ permeability equivalent to that of a traditional concrete dam. Contraction joints are formed through the dam. If immersion-vibrated concrete is used on the faces of the dam, it is provided to give an improved finish and to contain the water stops at the upstream end of induced contraction joints.

A further classification of a medium-cementitious RCC dam, which has a cementitious content between 100 and 149 kg/m^3

3 ENGINEERING PROPERTIES

The most important properties of RCC when used in dam construction are density, permeability, compressive strength, shear strength and tensile strain capacity.

Density:

The in situ density of RCC is depends largely on specific gravity of the aggregates to be used. It also depends on void ratio of fine aggregate and the paste-mortar ratio. A lean Paste RCC has about 95% to 98% of Theoretical Air Free (TAF) density while in high paste ration it varies from 98% to 99.5%.

Unit weight of RCC is typically equal or slightly higher than for conventional non air entrained concrete.

Permeability:

Permeability value of RCC are generally lower than for conventional concrete due to presence of high cementatious material. Usual values of permeability in low paste RCC varies from 10^{-4} to 10^{-9} m/sec while same varies from 10^{-10} to 10^{-13} m/sec in high paste RCC.

Compressive Strength:

In general compressive strength of RCC mixes is higher at later ages than for a conventional concrete mix with the same cemntatitious content.

Tensile Strength:

Tensile strength as a percentage of compressive strength is generally lower for RCC than it is for conventional concrete. Tensile strength varies with aggregate quality, age of concrete, cement content and strength. In lean RCC it is about 0.5 MPa and it varies from 1.5 MPa to 2.5 MPa in high paste RCC.

Shear Strength:

The cohesion value (C) of RCC typically varies from 0.5 MPa to 4 MPa. Friction angle at bonded joint is usually more than 45 degree. For preliminary design typical value of Angle of Friction (φ) is 45 degree and Cohesion as 10 % of compressive strength of RCC.

Tensile Strength Capacity:

Strain is induced in concrete when a change in its volume is restrained. When the volume change results in, tensile strain that exceeds the capability of the material to absorb the strain, a crack occurs. The threshold strain value just prior to cracking is tensile strain capacity of material. It is influenced by rate of loading, type and shape of aggregates and cement content

RCC tensile strain capacity is lower because of less cement content.

Elastic properties:

A low value is desired to decrease the crack potential. Typical values are 15 to 25 GPa.

Poisson Ratio:

It is usually similar to conventional concrete and its values varies from 0.2 to 0.3.

4 DESIGN CONSIDERATIONS

The use of vibratory rollers to compact concrete instead of immersion vibrators does not change the basic design concepts for dams; nevertheless, it does affect construction procedures. Therefore, during construction planning, the structural design and layout of appurtenant structures and inserts and the methods that are to be used for the treatment of the joints between the layers must all be considered so that the advantages of the rapid method of construction that is possible with roller compacted concrete are not lost.

Important considerations that must be addressed before proceeding with the design of an RCC dam include the basic purpose of the dam and the owner's requirements for cost, programme, appearance, water-tightness, operation and maintenance. A review of these considerations should help to determine the optimum RCC mixture proportions, the type of layer surface treatment, the method of forming the face of the dam and the basic configuration of the structures. The overall design should be kept as simple as possible in order to fully utilise the advantage of the rapid method of construction using RCC. The Designer, in taking advantage of the flexibility afforded by RCC, must balance the potential cost savings against the technical requirements of the structure.

5 RCC GRAVITY DAMS

RCC gravity dams are designed to the same criteria as a traditional concrete gravity dams with respect to stability and allowable stresses in the concrete. Given the layered form of construction of RCC, the strength of lift joints and the potential for sliding on lift joints must be considered carefully. With high cementitious content RCC, good cohesion is achievable, but low-cementitious RCC and RCCs that segregate can have low cohesion. Low cohesion values also tend to lead to high permeability. The shear properties and permeability at lift surfaces are dependent on a number of factors that include material properties, mixture proportions, joint preparations, construction operations, and exposure conditions. Actual values used in final designs should be based on tests of the materials to be used or careful extrapolation from tests on RCC mixtures from other projects with similar aggregates, cementitious material contents, and aggregate gradings.

As with any dam design, the Designer of RCC structures must be sure that design assumptions are realistically achievable with the construction conditions anticipated and the materials available.

Gravity dams are normally analysed as two-dimensional structures using conventional plane-stress analysis or finite-element analysis. For all but the largest dams, the thermal performance of an RCC gravity dam does not affect the design of the dam cross-section as the section is monolithic in the upstream-downstream direction and no forces are assumed to be transmitted along the dam axis. For a traditional concrete gravity dam, the dam-foundation interface is usually the most critical section for stability evaluation. However, because of the potentially weaker horizontal joints between the layers, in addition to the dam-foundation interface, it is also necessary to perform stability analyses for other critical sections through the body of the RCC dam.

Seismic aspects:

The analysis of RCC dams for seismic loading conditions is identical to that for traditional concrete dams. In seismic design of concrete dams, there are certain "good practices", such as eliminating or minimising geometrical discontinuity in the dams and reducing dead load at the top of the dam. These practices are equally applicable to RCC dams. The tensile and shear strength of the horizontal lift joints required for seismic loading may be higher than those under static loading. Proper measures have to be taken during construction to accommodate these requirements.

Galleries

Galleries and adits serve the same purposes in RCC dams as they do in traditional concrete dams. For example, a foundation gallery can serve as access to the interior of the dam for inspection, as a collector of seepage, as access for instrumentation and other equipment, and as a terminal point for drain holes drilled from the crest or a gallery at a higher elevation. Design requirements for RCC galleries and adits are commensurate with those of traditional concrete dams. The paradox is that the inclusion of galleries in RCC dams interferes with clean, efficient placement and compaction of RCC. For that reason, some Designers of RCC dams would like to reduce the number of galleries and adits to a minimum, especially in low dams where the need for them may be questionable [24]. However galleries provide the only immediate interior access during operation for inspection, for safety, and to clean or re-drill drains to maintain stability as designed. Costs associated with failure or for additional stability can far outweigh the costs of construction. RCC productivity may drop 10 to 15 percent for those layers that cross a gallery.

Designers of RCC dams should weigh the advantages and disadvantages of galleries. For example in low dams (or at the ends of large dams), it is possible to place a porous-pipe drain in the foundation and drill from the top of the dam to intersect with the pipe. Where galleries are necessary, the layout of the gallery should be designed taking into account the effects on RCC placement operations. If possible, the gallery should be located a reasonable distance from the upstream face to allow construction equipment to operate in the area. The galleries can be stepped in a manner that when placing the RCC adjacent to the gallery, access to placement areas is not completely blocked. The gallery construction methods (see Section 5.9) should be consistent with the purpose of the gallery. A gallery that is only to provide access to the interior of the dam can be constructed by any method. A gallery that is intended to provide a means to inspect the RCC and to observe cracks should avoid the methods that mask the RCC, e.g., pre-cast concrete forms.

Spillways:

Spillway designs used for traditional concrete dams are also suitable for RCC dams. However, in RCC dams when gated spillways are provided, they are constructed in conventional concrete.

Appurtenant structures and inserts

Appurtenant structures and inserts can provide obstacles to RCC placement. The preferred practice for RCC dams is to locate any insert that has to pass through the dam in or along the rock foundation to minimise delays to RCC placement.

6 INSTRUMENTATION

The instrumentation in an RCC dam is similar to that in a traditional concrete dam. However more emphasis is usually placed on the thermal conditions in an RCC dam (because of the more rapid method of construction) and therefore frequently there are more thermocouples in an RCC dam than in a comparable traditional concrete dam.

Unless carefully planned, installation of embedded instruments such as strainmeters, thermocouples and piezometers can interfere with RCC construction and their installation should be carefully considered during design. It is noted that these instruments also interfered with the construction of traditional concrete dams but to a lesser degree.

7 CONSTRUCTION

The layout, planning and logistics for construction of RCC dams are somewhat different from those of traditional mass concrete dams. Instead of vertical construction with independent monoliths, RCC construction involves placing relatively thin lifts over a large area, essentially placing a series of roads one on top of the other in rapid succession. If a problem develops on a given layer, it has to be resolved before any subsequent layers can be placed. There are no alternate monoliths on which to work while a problem is being studied. It is therefore important that all related activities such as foundation clean-up, access and delivery of materials and embedded parts be planned and programmed well ahead of time. When problems of an engineering nature develop, responsibility and authority to act on those problems should ideally be at site level.

Traditional mass concrete placement usually requires a high ratio of man-hours to volume placed due to labour-intensive activities, such as raising formwork, joint preparation and consolidating concrete with immersion vibrators. RCC usually has a lower ratio of man-hours to volume placed because of the use of mechanised equipment for spreading and compacting the concrete, less formwork and reduced joint preparation.

Thickness of layers:

A number of factors effect the thickness of layer. Modern vibratory rollers have more than sufficient energy to obtain good densities with a well-designed workable RCC in layer thicknesses certainly up to 1000 mm. The more important factor is the need to have sufficient compactive energy at the bottom of the layer to obtain good bond between the new layer and that previously compacted. A further factor that influences lift thickness is the maximum allowed exposure time before covering one layer with the subsequent layer. Each project should be studied to optimise the benefits of various layer thicknesses. Thicker layers mean longer exposure times but fewer joints between those layers and thus a reduction in the number of potential weaknesses in the structure. Thinner layers result in more potential joints but allow those joints to be covered sooner, resulting in improved bond. The compacted thickness of any RCC lift should be at least three times the diameter of the maximum size of aggregate The majority of layers have been 300 mm thick.

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Compaction:

Traditional concrete is consolidated during vibration, whereas the density of an RCC is achieved by compaction. There are a great variety of parameters that can influence the compaction, such as the maximum size of aggregate, the quantity and type of cementitious material, the water content, the thickness of the layers, the equipment used, etc. Adequate compaction is an essential factor in order to obtain a good-quality RCC. RCC is roller compacted or tamped into a dense mass by external energy rather than by being internally (or externally) vibrated and densified by settlement under its own weight. Compaction should be performed as soon as practicable after the RCC is spread.

Manoeuvrability, compactive force, drum size, frequency, amplitude, operating speed, and required maintenance are all parameters to be considered during the selection of a vibratory roller. The compactive output in volume of concrete per hour obviously increases with size and speed (which should be limited in the Specification) of the roller. Project size, workability of the mix, layer thickness, the extent of consolidation due to dozer action and space limitations will usually dictate selection. Rollers larger than about five tonnes usually cannot operate closer than about 200 mm to vertical formwork or obstacles, so smaller hand-guided compaction equipment and thinner layers are usually needed to consolidate RCC in these areas.

The freshly-spread RCC surface should be smooth so that the roller drum produces a consistent compactive pressure under the entire width of the drum. If the spread surface of less workable RCCs is not smooth, the drum may over-compact high spots and under-compact low spots.

Generally four to eight passes of a 10-ton vibratory roller will achieve the desired density for RCC in 300-mm thick layers. This assumes compaction in a timely manner with appropriate equipment. Over-compaction or excessive rolling should be avoided as it may reduce the density in the upper portion of the layer.

Compaction should be accomplished as soon as practicable after the RCC is spread, especially in hot weather. Compaction is frequently specified to be completed within 15 minutes of spreading and 45 minutes from the time of initial mixing. Tests have shown substantial and rapid reductions in compressive, tensile and modulus values if low-cementitious RCC is compacted when it is more than about 30 to 45 minutes old and the mix temperature is 20 °C or higher

8 JOINTS BETWEEN LAYERS OF RCC.

The performance of an RCC dam will almost entirely be dictated by the performance of the horizontal joints between the layers. If there is no segregation when the RCC is placed and spread, if there is intimate contact between the two layers and if there is sufficient energy from the vibratory roller to turn that contact into good bond, the RCC will perform as a monolithic structure with a performance at least to that of a traditional concrete dam. In addition the joint surfaces must be scrupulously clean; this is generally accomplished by a vacuum truck or by air blowing. However, if any of these factors are not present, the performance of the joint may be less satisfactory.

The adhesion between layers of RCC is produced by two mechanisms; cementitious (chemical) bond and penetration of the aggregates from the new layer into the surface of the previously-placed layer. As the exposure time between the placement of the layers increases, the chemical bond becomes the predominate factor because the potential for penetration of the aggregates decreases faster than the chemical bond.
The treatment of the joints between the layers differs from that of traditionally placed mass concrete because there is no surface water gain during setting of the concrete. Consequently, there is no weak laitance film on the surface. Surface water gain (bleeding) is the result of subsidence during setting, when the excess water separates from the mixture and is displaced to the surface by the heavier materials. Bleeding does not occur in properly-proportioned RCC with a reasonable water/cementitious ratio. However, it is not uncommon for full consolidation of the RCC to bring paste to the surface. This paste is not weakened by subsequent water gain and, if properly cured, does not have to be removed prior to placement of the covering layer.

Following three classes of joint treatments are defined:

1. **Fresh (or "hot") joint** – this is a joint that occurs when the RCC layers are being placed in rapid succession and the RCC is still workable when the next layer is placed.

2. **Intermediate (or "warm" or "prepared") joint** – this is the condition that occurs between a fresh joint and a true "cold" joint.

3. **Cold joint** – at this stage the surface of the previously-placed layer is judged to be such that little or no penetration of the aggregate from the new layer will be possible into the previously-compacted layer.

Bedding mixes used on the surface of the layer will improve the shear and tensile strength of the joint for a given set of conditions. However, with a well designed workable RCC with an excess of paste an equivalent performance can be achieved. Indeed, the joints with the best performance found during the testing of cores have all been in RCC dams without bedding mixes.

There are two forms of bedding mix, mortar and bedding concrete (with a maximum size of aggregate greater than 5 mm). The mortar is generally 10 to 20 mm thick. The thickness of the bedding concrete has varied considerably and has been up to 75 mm thick.

9 CURING AND PROTECTION OF RCC

After the RCC has been placed and compacted, the surface of the layers should be kept continuously moist 24 hours a day and protected from drying or freezing prior to placement of the next layer as would concrete placed by traditional methods. The surface should be clean and at, or near, a saturated-surface-dry (SSD) condition just prior to placement of the next layer. The surface should also be protected from freezing by insulating with plastic thermal blankets (or some other means) until it gains sufficient maturity.

The final layer of RCC should be cured for an appropriate time, generally in excess of 14 days. Curing compound is generally considered unsuitable because of the difficulty in achieving 100 % coverage on the relatively rough surface, the probable damage from construction activity, the low initial moisture in the mixture, and the loss of beneficiary surface temperature control that is associated with moist curing when there is low relative humidity. An effective cure is covering with a layer of damp sand which will also provide beneficial thermal protection.

10 PERFORMANCE

Cores have been extracted from a significant number of RCC dams for testing to ascertain the performance of the dam. The diameter of the cores should be at least two and a half times that of the maximum size of aggregate and preferably three times. These results giving the in-situ performance are far more important that those obtained from testing of manufactured specimens as they give a representation of the concrete in the dam. In addition it is the performance at the horizontal joints between the layers that is of more importance than the performance of the parent (unjointed) material.

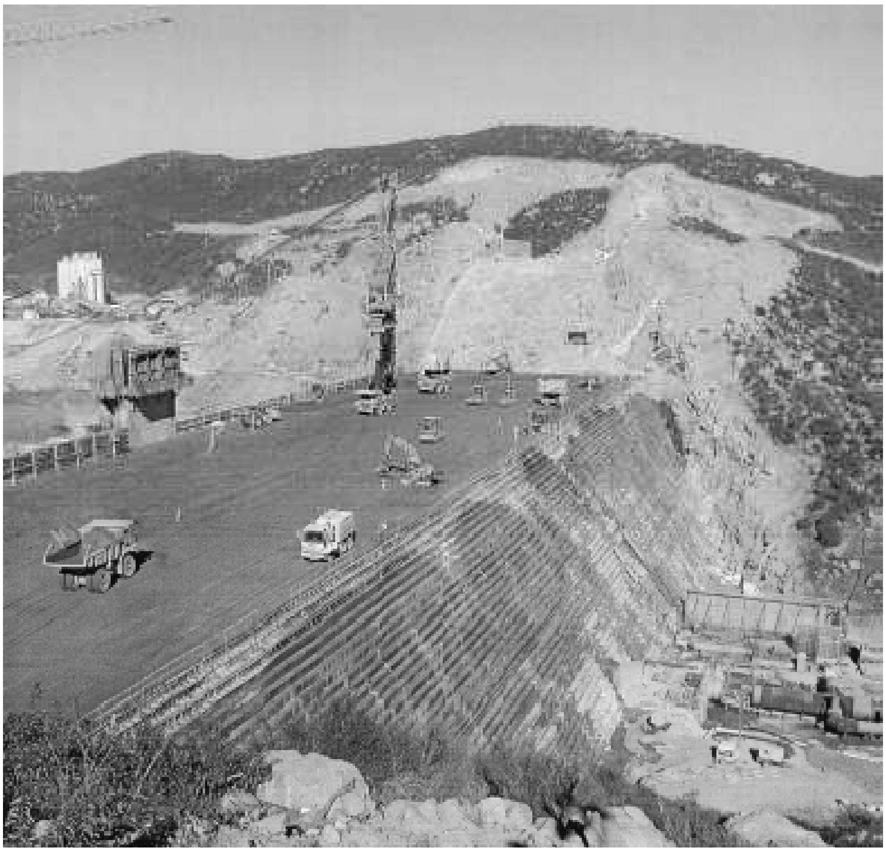
Results of various constructed RCC dams have shown that RCC dams can be designed for any reasonable impermeability and there are examples of dams in which the in-situ permeability has been measured at 1 $X10^{-12}$ m/s which is at least as good as, if not rather better than, the in-situ permeability of traditional concrete dams

It is also apparent that the performance at the joints increases significantly as the cementitious content increases, for example with low-cementitious RCC dams the average cohesion is approximately 6 % of the compressive strength, while with high-cementitious content RCC dams it is over 9 %.

11 ADVANTAGE OF RCC DAM

The advantages of RCC in dam construction as compared with traditional concrete dams include:

- More rapid construction (2.5 to 3 m vertical progress per week can be achieved in large dams – greater rates have been achieved in smaller dams);
- Effective use of conventional equipment (trucks, dozers, vibratory rollers, etc.);
- A reduced cost of construction as a consequence of the above



OVERVIEW OF GHATGHAR PSS- PLANNING, CONSTRUCTION AND SUPERVISION w.r.t. ROLLER COMPACTED CONCRETE DAM

BY

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ABSTRACT: Ghatghar Pumped Storage Scheme is conceived for enhancing hydropower capacity of Government of Maharashtra and thereby achieving balance of hydro-thermal ratio in the state. Ghatghar Pumped Storage Scheme is first ever largest scheme (250 MW) consists two units of 125 MW each. Ghatghar Pumped Storage Scheme comprises underground powerhouse complex. The construction of machine hall, transformer hall of powerhouse complex are in advanced stage. In order needs commission powerhouse, the construction of upper and lower dam needs to be completed prior to or concurrently to powerhouse. Upper dam is 15 m high while lower dam is 84 m high. Conventional masonry or concrete dam would require 5 to 10 years for construction. This has lead to search for latest technology to build dam fastest. The answer to this is Roller Compacted Concrete (RCC). Roller Compacted Concrete (RCC) dam is a key solution for number of hydro projects world over. India has also gone for its first ever RCC dam for Ghatghar Hydroelectric project. This project is unique of its kind because of use of reversible turbine pumps and adoption of RCC technology for three dams. Out of three dams Saddle dam (40000 m^3) has been completed and in next two seasons, it is planned to complete the Lower dam (600000 m^3). On successful completion of Lower dam the new trend will be set for adopting RCC technology in upcoming new dams for hydroelectric projects in the country.

This paper describes preliminary trials like Test sections, full scale trial and the construction methodology. Use of fly ash is also an attractive eco-friendly feature of the dam.

Introduction

Ghatghar Pumped Storage Scheme in Maharashtra state of India is on river Pravara, 15 km upstream of existing Bhandardara dam on same river, provider for an installation of two reversible pump turbines, each of 125 MW in an underground powerhouse.

Power will be generated during peak hours of demand using potential energy by releasing water from upper dam to lower dam and water will be pumped back from lower dam to upper dam during night hours utilizing surplus power.

The scheme envisages construction of two reservoirs, i.e., (i) Upper reservoir near village Ghatghar District Ahmednagar (ii) the lower reservoir near village Chondhe (Bk.) District Thane (iii) and a water conductor system with powerhouse in between the two dams. The scheme involves transferring of water alternately between the upper reservoir and the lower reservoir with a vertical distance of more than 400 m between them. The scheme is intended to be operated on a weekly cycle with generation for six hours daily except Sunday and pumping for seven hours during of peak hours from Monday to Saturday and the balance on Sunday.

To have a speedy construction of this hydroelectric project, the upper dam and lower dam are proposed to be constructed in Roller Compacted Concrete. The features of upper and lower dam are as given below:

Table No.1: The features of upper and lower dam

Dam	Height	Length	Qty. of RCC
Upper dam	14.5 m	451 m	42000 m^3
Saddle dam No.1	11.5 m	250 m	13000 m^3

Lower dam	84.0 m	415 m	600000 m^3

Necessity of RCC Dams

Ghatghar Pumped Storage Scheme is the first ever largest scheme (250 MW) each. The Government of Maharashtra (India) comprising two units of 125 MW each. The scheme was administratively approved in June-1988. A loan of Rs. 360 crores was granted in December-1988 from J.B.I.C., Japan. However forest clearance was accorded in 1992 and clearance under wild life protection act was obtained as late as in 1997. The actual work could start only in the year 1998. This loss of precious time about 10 years was to be negotiated by finding rapid methods of construction for three dams. The work of underground powerhouse was started in the year 2000 with construction period of four years and in advance stage of construction. The conventional concrete / masonry dam would require at least 5 to 10 years for such a large dam of 84 m high. In order to commission the powerhouse the construction of all three dam's needed to be completed prior to powerhouse. This has lead to adopt this new technique of RCC dam. Due to this decision the construction period could be reduce to three years. This is the main advantage to go for RCC dams. This has been proved world over for number of hydroelectric projects.

RCC Dams

Three RCC dams namely saddle dam, (11.5 m) 13000 m^3 upper dam (14.5 m) 42000 m^3 and the lower dam (84.0 m) 600,000 m^3 will be built in the same order for Ghatghar pumped Storage Scheme. The saddle and upper dam is located near the origin of river Pravara at Ghatghar close to the westerly natural slope of Sahyadri ranges at MSL 750 m. It provides storage of 5.87 m^3 of water required for the operation of pumped storage scheme. Tow saddles on right fringe of upper reservoir, open out on the westerly slope, out of which one saddle is already build in colgrout masonry. The Saddle.250 m, along with maximum height of 11.5 m, above the deepest foundation was proposed to be constructed in RCC and is completed in April 2003. The upper dam is gravity dam 451 m long with 14.5 m maximum height above the deepest foundation the dam has a vertical upstream face and a stepped downstream face with a slope 0.8:1 (H.:V.) beyond 10 m vertical from the top. Top width is 8 m. Central spillway in the river portion with 5 radial gates each of 12 m^3 m in size. River sluice located in one of piers of the non-overflow portion. The overflow portion is in conventional concrete while the non-overflow portion of the dam is being constructed in RCC.

The lower dam is located down the continental divide across Shahi river, near Village Chonde at MSL 350 m which is at the foot hill of Sahyadri ranges. It provides storage 3.80 Mm^3 of water. The lower dam is a gravity dam 415 m long with maximum height 84 m above deepest foundation. The dam has upstream batter of 0.14:1 (H: V), and downstream batter of 0.782:1 (H: V) and beyond 11 meter height, it is vertical and the top width of 8 m. An ungated stepped spillway 84 m long with height and width of each step at 0.6 m and 0.47 respectively is proposed in the gorge portion with 8 m wide bridge on the top. The discharge over the stepped spillway is 2 m^3/m which is quite low. The lower dam including overflow portion is proposed to be constructed in RCC.

Design of Dams

All the three dams are designed basically as gravity dam. The lower main dam having maximum height of 84 m is provided with foundation gallery, and foundation drainage holes, inspection gallery, ventilation shaft, lift shaft etc., as per the conventional gravity dam. The other two dams with low height are not provided with such openings.

All the three dams are proposed to be constructed using high paste RCC withl layer thickness of 300 mm. Further to make the dam impermeable 0.5 m-1 m wide facing both upstream and downstream is proposed to be formed in grout enriched vibrated RCC (GEVR). All the three dams are being constructed by one and the same contractor, Patel Engineering Ltd (India) in association with ASI (USA).The work of powerhouse complex is also with the same contractor. Thus coordination between two works has become easier. The design of the dam is done by Tata Consulting Engineers (India) as an Indian counter part of EDPC (Japan) who is the prime consultant. Malcolm Dustan and Associates (UK) is the consulting engineer for RCC dam.

Test Section 1

Based on MERI mix (**Table2**) Test Section 1 was carried out in March 2002. Test section 1 was successful in understanding methodology and sequence of operation and training the staff. Also new procedure for GEVR was decided in Test Section 1. However the results of test section were not up to the mark for impermeability and requirement for strength due to lapses in quality control aspects like lack of uniform gradation of aggregate, no temperature control etc. due to these discrepancies it was decided to place additional test section 2. It was realized from test section 1 that extensive laboratory work is required to finalize the mix proportion. The results of test section 1 were analyses in light of mix design parameter. Accordingly a detailed mix design exercise at field laboratory was carried out. The cementations material was kept same as per MERI design and changing other parameters like gradation of aggregate sand / aggregate ratio. The corrections for specific gravity and water absorption, were applied. The different mixes were evolved based on 28 days strength result.

Table No. 2: Details of mix by MERI

Cement (Kg)	Flyash (Kg)	% Flyash	Sand (Kg)	Course Aggregate (Kg)			Water (Kg)	Admixtures (Kg)
88	132	60	714	50-20 mm	20-10 mm	10-5 mm	115	0.88
				769	390	391		

Test Section 2

Test section 2 was carried out in October 2002 based on mix by field lab (**Table3**), though there was increase in strength of specimen as well as in-situ core strength. The test section 2 was not successful in getting a continuous core, due to segregation observed at lift joints. At this juncture, Malcolm Dunstan and Associate (MD&A) were invited for guidance. The objective of the trial mix programme carried out by MD&A was to optimize the gradation and workability of the RCC so that cohesive RCC could be obtained with little potential for segregation and one that could be used for GEVR. At this stage actual strength of RCC was considered to be less important because it was to be used initially in the Saddle dam in which the stresses required are less stringent than the upper dam and much less stringent than lower dam. Two main changes were made to the specified mixture proportion, first the gradation was changed and second mix was made much workable. The actual optimize gradation is compared to the suggested limit. This mix was designated as G-85.

Table No. 3: Details of mix by field lab

Cement (Kg)	Flyash (Kg)	% Flyash	Sand (Kg)	Course Aggregate (Kg)			Water (Kg)	Admixtures (Kg)
88	132	60	721	50-20 mm	20-10 mm	10-5 mm	146	0.88
				745	314	265		

A small trial of the proposed RCC was undertaken with the mix suggested by MD&A. A number of load of RCC were dump from truck on to the surface of test section 2. The RCC was spread in layer and compacted by 4 passes of the double drum vibratory roller (1 static and 3 vibratory). All the nuclear densitometer readings were in excess of 2500 kg/m3. Although the trial was only small it was sufficient to show that RCC with the gradation proposed and with the workability proposed could easily be handle and could be roller compacted to a high density. The surface of RCC after roller compaction was satisfactory. The same mix was decided to be adopted for placement of saddle dam as a full scale trail.

Table No.4: Details of mix by MD & A (G-85)

Cement (Kg)	Flyash (Kg)	% Flyash	Sand (Kg)	Course Aggregate (Kg)			Water (Kg)	Admixtures (Kg)
				50-20 mm	20-10 mm	10-5 mm		
88	132	692	558	558	446	406	135	2.2

Full – Scale Trial

The full-scale trial was done for layer 1-2-3 with mix proposed by MD&A as shown in **Table 4**for saddle dam in December 2002. it was found that both the cylinder density and in-situ densities are very much higher than those found during the first two test section. This was because the water contents of the mix was higher and thus theoretical air free density was lower than that of mixes used in two test sections. The reasons for higher density were that the RCC was much more compactable and thus there were lower air voids.

The objective of the full scale trial was to investigate all the construction procedure in particular the joint treatment between the layer and the training of personnel who were suppose to work at the Upper dam and lower dam. The first trial was constructed between 9 to 27 March 2003 and consisted of 19 layers of 300 mm compacted thickness with a total volume of 6780 m^3 in left hand side (LHS) portion of saddle dam. Similarly the second half of saddle right hand side (RHS) portion was started on 22nd April 2003 and completed on 9th May 2003. The RCC volume of 7431 m^3 was placed in RHS. Thus this full scale trial of RCC placement for complete saddle dam for the placement of RCC volume of 14200 m^3 gave good learning curve to all the supervisors of department and the contractor and the designer. During this learning, following activities were learned:

. Calibration of nuclear densitometers and measuring the in-situ densities
. Optimization of dosages of set-retarder
. The optimization of the exposure time between layers
. The insertion of joint crack inducers
. The use of interface against abutment: Upstream and downstream GEVR
. The optimization of water contents in different hours of day and night.
. The optimization of number of passes of vibratory rollers

Construction Plants and Equipments

The contractor has installed a crushing plant of 100 t/hr initially and subsequently added one more 80 t/hr for upper dam and Saddle. A batching plant of 64m^3/hr was installed for placement of Saddle and RHS portion (15000 m^3) of upper dam. The contractor subsequently installed one 120 m^3/hr. 'SCHWING' batching plant for LHS portion (25000 m^3) of upper dam. The minimum capacity for crushing plant was specified as 220 t/hr (draw out) Therefore aggregate productions was less resulting into delay of RCC placement for saddle and upper dam, so also was the case with batching plant. The specified capacity for batching plant was 100 m^3/hr (output) which has resulted in long exposure time. That was to be negotiated by using 'CONPLAST R' set-retarder.

Aggregate cooling system

The specifications required that placement temperature of RCC shall not more than 27^o C. To achieve this aggregate were covered by weather sheds and were chilled by water to 10^o C. But during summer, hottest day, chilling plant had difficulties and the work of upper dam was to be stopped for few hours and to be taken during cooler hours.

Transportation of RCC

The batching plant is installed at upper dam location which is at a distance of 2 km from the saddle. The transportation of fresh RCC was done by the dump trucks with capacity of 4 m^3. A special arrangement was done to the rear of the trucks to reduce the potential for segregation when the concrete was dumped. The trucks were covered to avoid dust and to protect from solar radiation.

A stacker-conveyor was installed at the center of saddle to convey the RCC from specially raised platform to saddle.

A hopper was installed at a fixed point from where the RCC was again discharge into intra-dump trucks in the body of a dam. Pair intra dump trucks were used for conveying the RCC to the point the placement.

RCC Placement

The RCC discharged at the point of placement by the intra dump trucks was spread the required thickness (340 mm un-compacted, to achieve 300 mm after compaction) the thickness of layer was assured by automatic laser guided system connected to blades of dozers.

The RCC was placed in 8 m wide strip parallel to the axis of the dam. As width was less there was not possibility of a joint either parallel or perpendicular of dam axis. The placement was carried out continuously 7 days a week and 24 hrs a day.

Curing

The RCC was cured continuously by using sprinkling the water into in order to keep the surface of a layer always in moist condition. A saturated surface dry (SSD) condition was maintained till the placement of next layer. After completion of RCC placement of Saddle continuous curing was done for 40 days.

Horizontal Joint Treatment

As per different exposure time the horizontal joint were treated in a different way. For exposure time of less than 24 hrs, only loose aggregate were removed and pounded water was squeezed. For exposure time of more than 24 hrs, an exposed aggregate finish was achieved for cold joints. When the concrete had stopped for a long time high pressure water blaster (40 MPa) was used. Depending upon mean temperature of the month, joints were classified as hot, warm, cold and super cold, and were treated accordingly. In saddle there was only one planned cold joint after layer 1, 2, 3, in RHS portion. While in upper dam there is only one planned cold joint at the end of working season in June 2003.

Contraction Joints

The contraction joints are vertical joint perpendicular to axis of a dam. These joints have been specified at 50 m for upper and saddle dam. Central Water Power Research Station, Pune (CWPRS) has recommended placement temperature as 17^{o}C. However in these thermal studies joints spacing and cooling effect due to gallery have not been considered. In order to negotiate this contraction joints are further decided to be at closer spacing of 25 m, accordingly they have been provided. Considering higher ambient temperature at lower dam 15 m joints spacing is proposed. In saddle dam and upper dam at each layer, galvanized steel crack inducer was inserted at the location of each contraction joints. The crack inducers were 500 mm long and 200 mm high. These plates were inserted in a freshly laid layer with the help of jack hammer very easily.

Quality Control

The quality control was exercised effectively through well equipped Quality Control Laboratory installed at upper dam site. The personnel involved in this were trained and the supervisor had responsibility for evaluating and controlling the following on a daily basis:

- Proper aggregate gradation
- Quality of cement and fly ash
- Mixture proportion and variability at the mixing plant
- Temperature and water content of fresh concrete both at mixing plant and placement points
- Fresh density and workability of the RCC (in terms of the Ve-Be time) insitue density
- Compressive and direct tensile strength

Conclusions

The in-situ properties of RCC for saddle dam are now available based on this; suitable optimized RCC mix proportion would be adopted for the lower dam. The lower dam RCC placement will be started only after analysis of the results of full scale trial on saddle as well as of upper dam. At the moment micro planning for execution of lower dam is being done on the basis of experience gained during this long learning curve. In general, the successful completion of Saddle and upper dam has boosted the level of confidence for everyone who has participated in this process with a view to achieve an excellent RCC dam. This step by step approach will facilitate in building the confidence level and heading towards perfection. The lessons learnt from each earlier dam regarding the mix design, construction methodology, adequacy of plant and equipment, results of strength obtained will be useful for construction of 84 m high lower dam which is the main target.

The daily rate of placement of LHS of upper dam with the batch plant of 120 m^3/hr was 900 m^3. The two batch plants of 120 m^3/hr, are installed at lower dam and suitable augmentation in batch plant capacity will be done. So that about 600000 m^3 of RCC will be placed with average monthly rate of placement 40000 m^3 in 15 months. Due to mon soon, the placement will be done in two seasons. This will be fastest dam building activity in order to commensurate the commissioning of 250 MW powerhouses. This is possible only because of decision of going for RCC. Thus, on the successful completion of the three dams in general and lower dam in particular the new trend will be set for adopting RCC technology in upcoming new dams for hydroelectric projects in the country.

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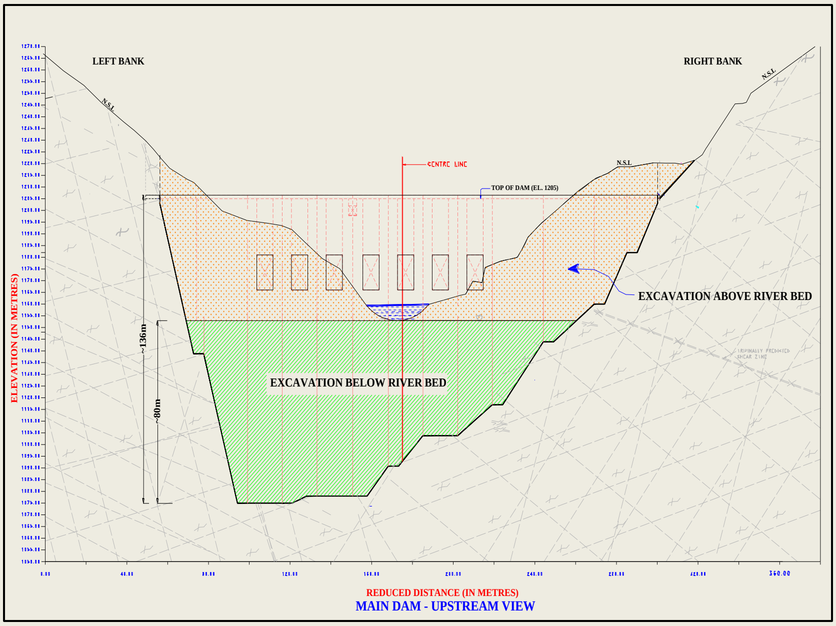
Dunstan, Malcolm – December 2002 and February 2003 Report on Value- Engineering proposal for the upper and lower dam.



River valley

- The hard rock foundation below the normal river overburden
- Sand, loose rocks and boulders

Concrete Dam Foundation

Rock foundation

- Strong to with stand stress
- Offer resistance to sliding
- No movement with in the rock
- Reasonably Impervious
- No differential settlement

Causes of Failure of Dams

• OVERTOPPING	29%
• **FOUNDATION**	**53%**
• OTHERS	18%

Failure of St. Francis dam

Major reasons:

Rock Formations

Conglomerate: expands with water

Schist: badly laminated, interspersed with talc

Fault line

Paleolithic landslide

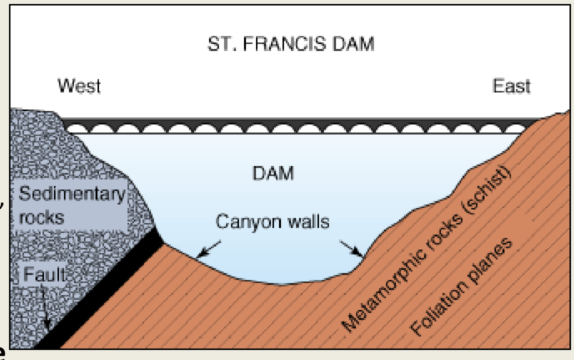
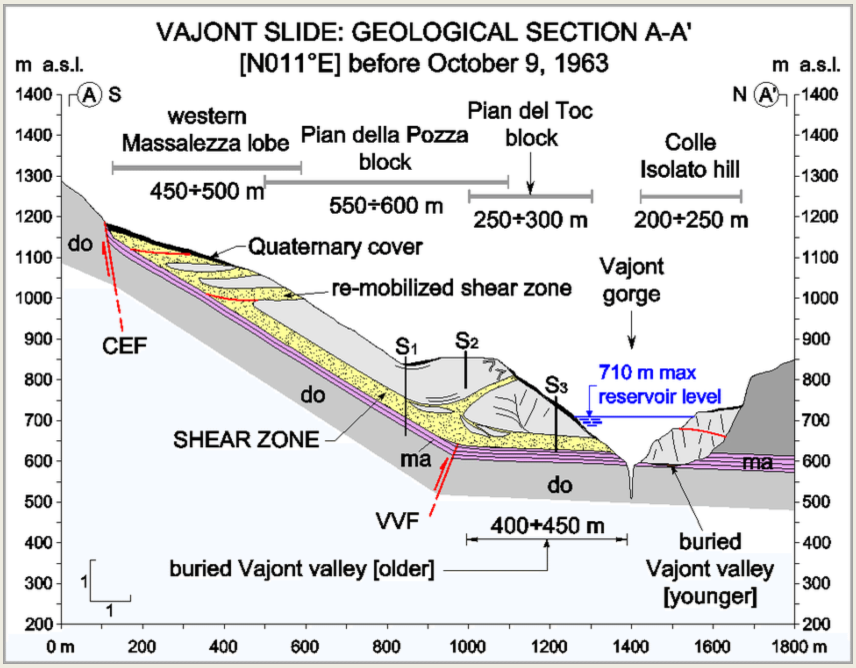


FIG. 3. Geological Section through the St. Francis dam-site.



Concrete Dam Foundation

- May not always be completely satisfactory all along the proposed foundation and abutment area.
- Cracks and joints (called seams) filed with poor quality crushed rock.
- In most cases strengthened artificially to

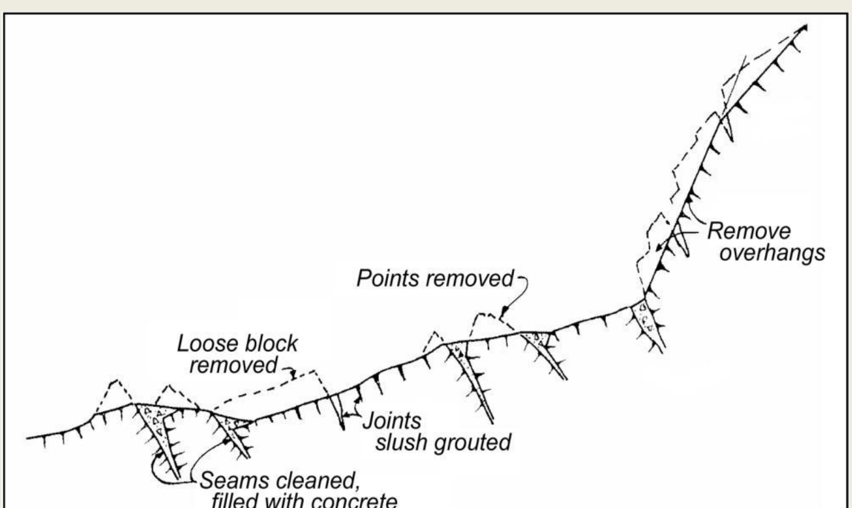
➢ Sustain the loads that would be imposed by the dam and the reservoir water.

➢ Check water seeping into the foundations under pressure from the reservoir.

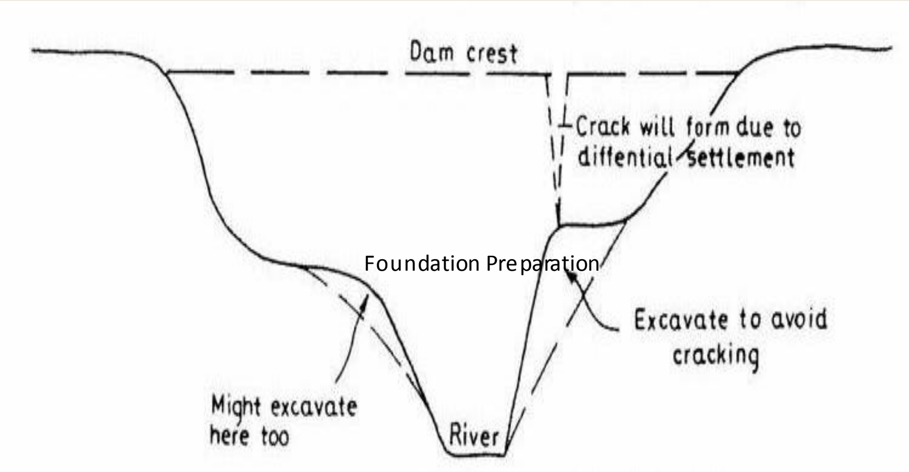
Concrete Dam Foundation

Improvement of the foundation for a dam by

- Grouting the foundation to increase its strength and to render it impervious.
- Excavation of weak rock zones and backfilling the entire excavated region with concrete.
- Making a concrete cutoff walls across leakage channels in the dam foundation.



Treatment of Rock Foundation



Foundation Preparation

Foundation Preparation

- The foundation surface should be shaped by excavation and filling
- Foundations such as shale, chalk, mudstone, and siltstone may require protection against air and water slaking.
- These excavations may be protected by leaving a temporary cover of unexcavated material, immediately applying a minimum of 12 inches of cement mortar to the exposed surfaces.

Foundation Problems

Most of the dams have to be built on complex foundations requiring special treatments. Various types of geological features encountered are:

1. Faults
2. Shear Zones
3. Shear Seams
4. Shattered/Highly jointed rock
5. Foundations with more than one type of rock with different properties
6. Folds
7. Buried Channels
8. Jointing pattern of the rock mass
9. Caverns/Cavities
10. Springs etc.

Faults and Shear Zone

- A **fault** or **fault line** is a planar fracture in rock in which the rock on one side of the fracture has moved with respect to the rock on the other side.
- A *shear zone* or *shear* is a wide zone of distributed shearing in rock. Typically this is a type of fault but it may be difficult to place a distinct fault plane into the shear zone. Shear zones may form zones of much more intense foliation, deformation, and folding. En echelon veins or fractures may be observed within shear zones.

Treatment of Fault and Weak Zones

- Faults and weak zones exist in most rock formations, their size, Continuity and orientations are important factors in determining the suitability of a foundation for any dam.

Type of Fault	Problem to Foundation	Treatment
Low- Angle Faults (Dip angle < 450)	Providing Inadequate Sliding Resistance	1. Excavating out the weak material 2. Providing Shear Keys 3. Use Rock Anchors
High – Angle Faults (Dip angle > 450)	The main problem is that of Bridging over of the Structure and the resulting Stress Concentrations	1. Dental treatment 2. Providing Seepage Cut-off on U/s 3. Use Rock Anchors

Shear Zone-Backfilled by Concrete

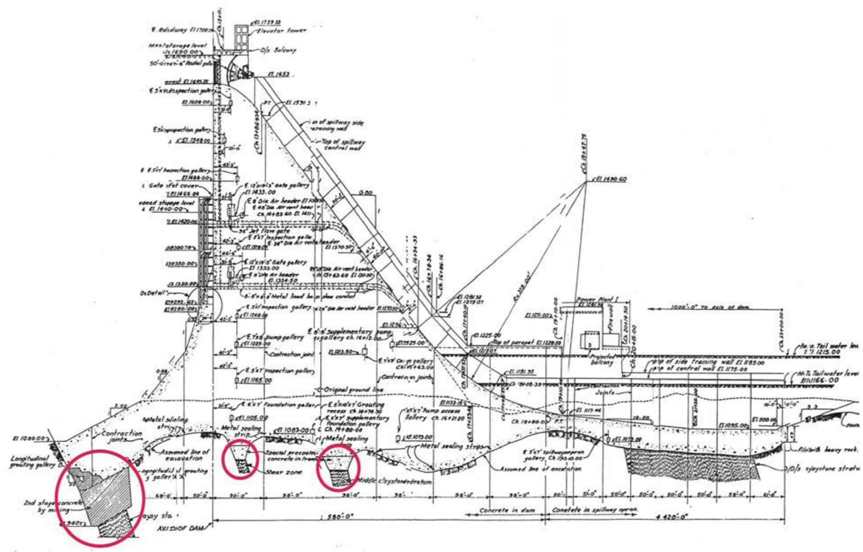


FIGURE 42. FOUNDATION FEATURE FOR BHAKRA DAM

Grouting

- Grouting is the process of injecting liquids, mixed suspensions, or semi-solid mixtures under pressure to achieve one or more desirable end results in terms of improving engineering properties.
- Permeation grouting is the injection of high-mobility grouts (HMGs) into small voids or low-(mobility grout (LMG) for effective filling of large voids.

Purpose of Grouting

Permeability Reduction:

- Necessary for reducing rates of seepage or leakage through or into structures and foundations
- Reducing hydrostatic forces acting on structures,
- Inhibiting internal erosion of foundation and embankment materials
- In any critical hydraulic application, grouting is normally one of several lines of defense.

Purpose of Grouting

Improvement of Mechanical Properties

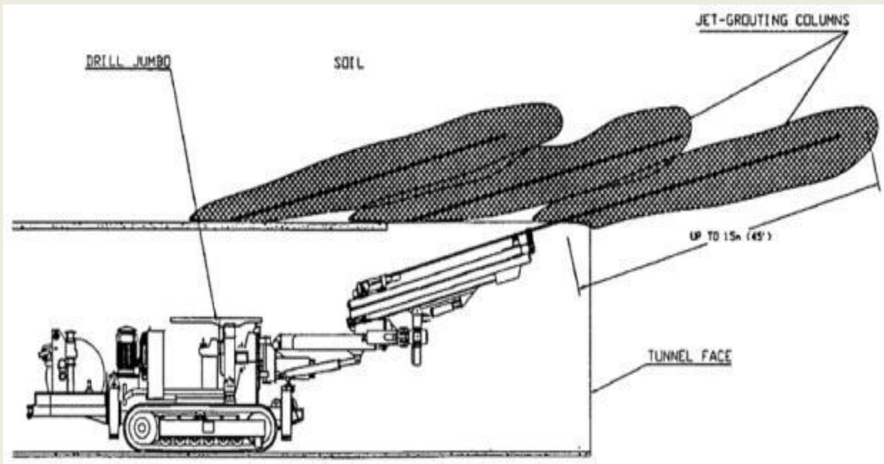
- Enhancement of bearing capacity
- Improvement in settlement-related properties such as elastic modulus and void ratio
- Improvement in shear strength, and elimination of voids that might adversely affect either loading conditions or the response to loads.

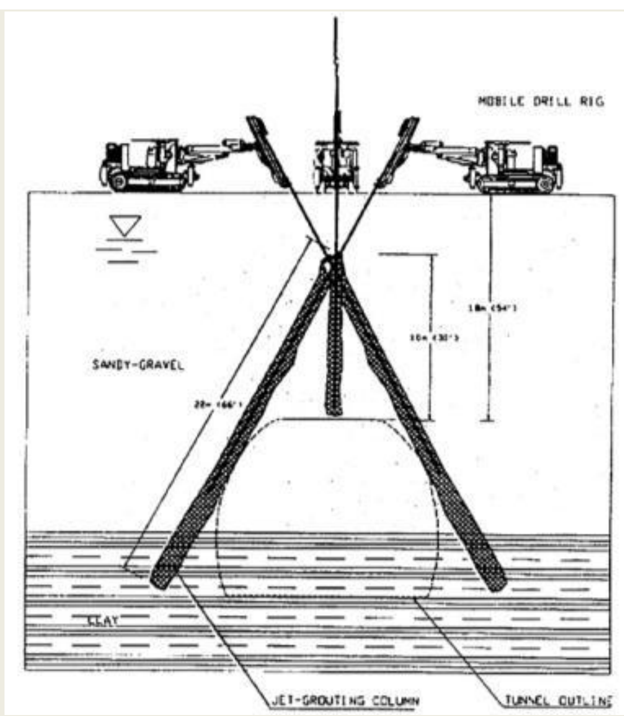
Application of Grouting in dams

- Curtain Grouting for Hydraulic barrier grouting to control leakage and pressure distributions.
- Foundation consolidation grouting to reduce foundation and structure deformations under load.
- Contact grouting of the interface between structures and foundations.
- Compaction grouting for densification of loose deposits or jet grouting to replace zones of loose materials.

Application of Grouting in tunnels

- Grouting in advance of tunneling to reduce water inflows during construction.
- Grouting in advance of tunneling to improve excavation stability and/or reduce or to prevent ground loss during tunneling
- Grouting between the tunnel lining and the tunnel excavation surfaces to reduce longterm tunnel loads, improve stress distributions, and reduce water inflows


JET-GROUTING COLUMNS
DRILL JUMBO
SOIL
UP TO 15m (45')
TUNNEL FACE


MOBILE DRILL RIG
18m (54')
10m (30')
SANDY-GRAVEL
20m (66')
CLAY
JET-GROUTING COLUMN
TUNNEL OUTLINE

Types of Grouting

- Suspension-type grouts:- The suspension-type grouts include clay, cement and lime
- Emulsion-type grouts:- The emulsion-type grouts include bitumen
- Solution type grouts:- The solution-type grouts include a wide variety of chemicals

Pure Cement Grout

- It is an unstable grout
- Bleeding can be avoided with water cement ratio less than 0.67
- Usual mix proportions are from water cement ratio 0.4 to 1 for grouting
- Very high mechanical strength can be attained with this type of grout
- The grain fineness is an important factor for fine fissures.

Bentonite Cement Grout

- In water stopping, grout will include a lot of bentonite and little cement. In consolidation works, grout will contain a lot of cement and little bentonite.
- Ideal mixes should be both stable and easy to pump.

Bentonite Cement Grout

When bentonite is added to a cement suspension, the effects are: -

- Obtain a homogeneous colloidal mix with a wide range of viscosity.
- Avoid cement sedimentation during grouting.
- Decrease the setting time index and separation filtering processes.
- Increase the cement binding time.
- Improve the penetration in compact type soils
- Obtain a wide range of mechanical strength values.

Foundation Grouting

In concrete dam foundation, two kinds of grouting programme are usually identified:-

- Consolidation grouting
- Curtain grouting

Consolidation Grouting

- The intent is to fill open fractures to improve the structural properties of the rock mass.
- To reduce deformations associated with closing of fractures under applied loads,
- To treat locally fractured zones and thereby homogenize the foundation,
- To fill fractures for the purpose of reducing movement of rock blocks that might otherwise loosen during excavation and/or tunneling operations.
- For foundations in karst areas to increase the level of assurance of adequate foundation support.
- The enhancement value of consolidation grouting depends on the rock mass conditions. Greatest benefit will occur in highly fractured rock masses with a predominant number of open joints.

Consolidation Grouting

- Low pressure grouting
- Shallow depth grouting
- Grout holes arranged in a pattern or grid

Purpose

- To consolidate the foundation rock and make more impermeable to so as to prevent erosion of infilling material in this zone of maximum seepage gradient.

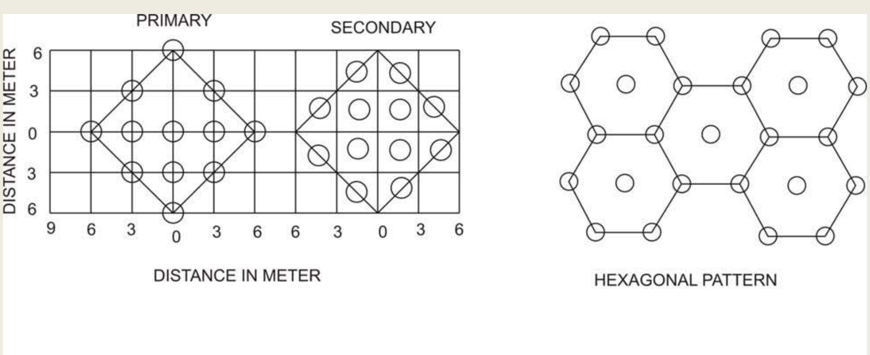
Consolidation Grouting

- Drilling and grouting is usually done from excavated surface
- Holes drilled normal to the foundation
- For the dams more than 30 m in height, it is usual to drill holes to a depth of 6 to 15 m.
- Spacing of 3 to 8 m.

Consolidation Grouting

- Grout pressure is governed by rock condition.
- These pressure could be as low as 0.06 MPa to a high range of 0.5 to 0.7 MPa.
- Consolidation grouting is usually done through a 76 mm to 150 mm holes.
- Ascending Method
- Descending Method
- IS : 4999

Consolidation Grouting



Grout Mix

- The grout mixes used for consolidation grouting are normally either balanced stable grouts or neat cement grouts.
- Neat cement grouts generally develop a higher compressive strength and have a shorter set time, either type of mix would normally suffice, and the choice of one versus the other is predominantly based on the project size and type rather than on specific property differences.
- For example, on a dam project where balanced stable grouts are being used extensively in a hydraulic barrier application, it would be logical to use the same grouts for consolidation grouting. On a foundation improvement project where the only activity is consolidation grouting, it might be more logical to use neat cement grouts to limit execution complexity.
- Where neat cement grouts are used, the starting mix should not be thinner than a 2:1 water-cement ratio (by volume). If it is found that thickening is routinely required during the grouting, the starting mix should be changed to a 1:1 water-cement ratio

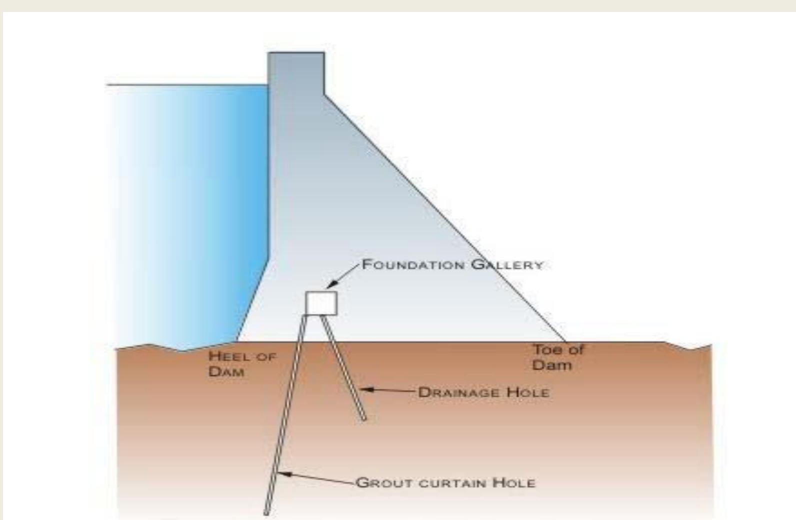
Refusal criteria

- Reaching a certain rate of grout take or reaching zero measurable take.
- Sometimes the definitions incorporate rates of take that vary with the applied pressure.
- Refusal be defined as the point at which there has been no measurable take (3-4 l/minute) at the desired pressure within a 15-minute period.
- It varied project to project based on various factors.

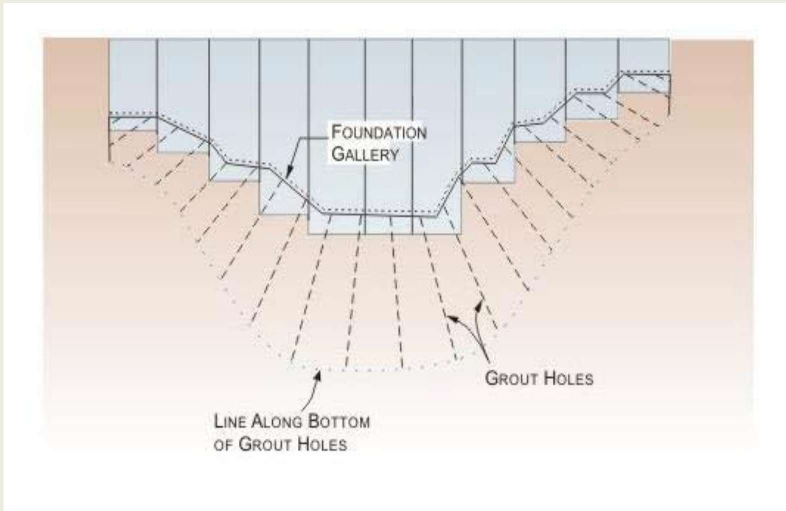
Curtain Grouting

- Grout curtain is provided near the heel of dam to impede the flow of water under or around the dam.
- Usually a single line of grout holes is provided below concrete dams
- High grout pressures without causing displacement in the rock or loss of grout through surface cracks, curtain grouting is carried out subsequent to consolidation grouting and after some of the concrete has been placed.

Curtain Grouting



Curtain Grouting



Curtain Grouting

- The grout holes are angled in the upstream direction or parallel to dam axis. The alignment of grout holes should be such that the base of the grout curtain will be located on the vertical projection of the heel of the dam.
- When the holes are drilled from upstream fillet, they are usually vertical or inclined downstream.

Curtain Grouting

- The depth of the grout curtain holes depends upon the nature of the rock in foundation and in general, it may range from 30 to 40 percent of the head of the water on good foundation and to 70 percent of head on poor foundations.
- According to IS: 11293 (Part2)-1993 “Guidelines for the design of grout curtains”, the following empirical criteria may be used as a guide:
- D= (2/3) H + 8

  Where ***D*** is the depth of the grout curtain in meters and ***H*** is the height of the reservoir water in meters.

Curtain Grouting

- Grout pressures are set to maximum permissible values for each segment and depth zone of the grout curtain on the basis of experience and geological evaluation supported by test grouting.
- Thumb rule for determining the maximum allowable pressure at the collar of the hole is that grout pressure should not exceed 0.23 Kg/sqcm per meter depth of the rock for average to weak rock.
- For sound rock with tightly interlocked joints, grout pressure several times of this value could be achieved.

Curtain Grouting

- Lugeon tests will determine the groutability of the rock mass and the type of grout to be used. Refer the following table for details
- The hydraulic conductivity is expressed in terms of the Lugeon value, which is empirically defined as the hydraulic conductivity required to achieve a flow rate of 1 litre/minute per meter of test interval under a reference water pressure equal to 1 MPa

Lugeon Units	Grout Usage
1 – 3	No grouting
3 – 10	Microfine cement or chemical grout
> 10	Ordinary Portland cement grout

Deere's Grout Take Classification System

Classification	Symbol	Original Criteria (maximum bags/ft)	Modified Criteria (Stable Grouts) (maximum gal/ft)
Very low	VL	0.09	1.0
low	L	0.18	2.5
Moderately Low	ML	0.36	5.0
Moderate	M	0.71	10.0
Moderately High	MH	1.43	25.0
High	H	2.85	50.0
Very High	VH	>2.85	>50.0

Grouting

- Grouting for ground engineering is a process for filling the voids, fissures or cavities existing in the soil and rock to improve water-tightness or mechanical characteristics
- The most important aim of the ground investigation for grouting is to identify if or not the ground is suitable for the intended grouting technique i.e. groutability of the ground.

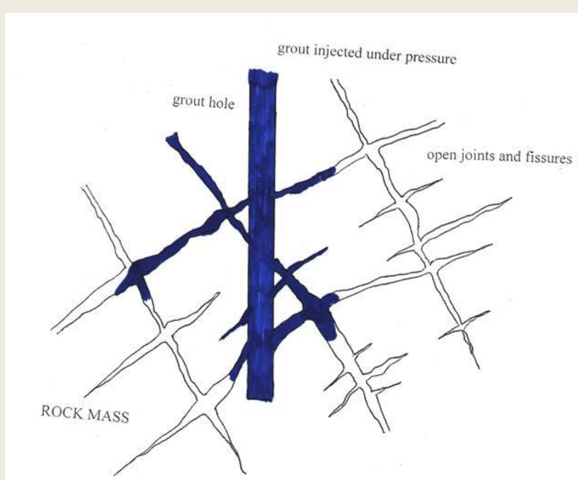
Grouting Techniques

- Rock Fissure Grouting,
- TAM Grouting,
- Compaction Grouting,
- Compensation Grouting and
- Jet Grouting

Rock Fissure Grouting

- Hole drilled through the fissures and joints of a rock mass to allow grout to be injected at close centers vertical
- The voids in the forms of rock fissures and open joints are filled with the grout injected under pressure with partial or complete displacement of infilling ground water

Rock Fissure Grouting



Rock Fissure Grouting

- Grouting pressure relates to the in-situ overburden pressure at the grouting depth.
- Maximum grouting pressure should be less than the overburden pressure. Above that pressure, joint / fissure will be opened and hydraulic fracture of rock may occur if the rock is poor and shattered

Rock Fissure Grouting

- Generally, it is assumed that the grout volume is equal to 5% to 10% of the rock mass volume to be grouted.
- Either the DTH Percussion Drilling Method or the Top Hammer Drilling Method is deployed to form the grout holes.
- The grout hole diameter is normally from 32mm to 150mm.

Tube-à- Manchettes (TAM) Grouting

- Soils or completely decomposed rock
- Sleeved perforated pipes in grout holes
- Partial or complete displacement of in-filling ground water.
- Often call Permeation Grouting
- The most obvious change in ground property by this treatment method is the reduction of permeability. Ground consolidation is also attained

Tube-à- Manchettes (TAM) Grouting

Criteria for design

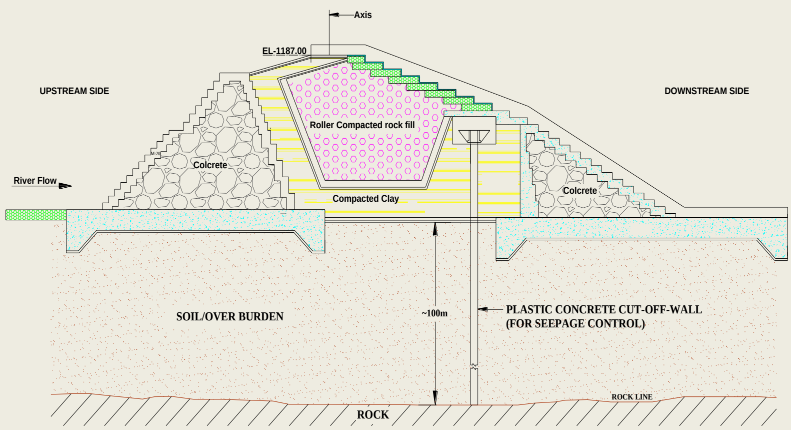
- The grout volume to be injected depends on ground porosity, geometry of the treated zone, grout hole spacing, stage length and total depth to be treated.
- The groutability of soil with particulate grouting has been evaluated based on the N value (Mitchell and Katti 1981)

Tube-à- Manchettes (TAM) Grouting

- N is defined as N = (D_{15})Soil / (d_{85})Grout.
- Grouting is considered feasible if N > 24 and not feasible if N < 11
- d_{85} - 85% finer size from grain size distribution curve of cement
- D_{15} -15% finer size from grain size distribution curve of soil
- IS 4999 :1991 Recommendation for Grouting of Pervious Soils.

Tube-à- Manchettes (TAM) Grouting

- Grout Pressure: Overburden pressure plus 100kPa or 20kPa per meter depth
- Drilling Method: ODEX system
- Sleeve grout is used to seal the gap between the grout pipe and the hole



U/S COFFER DAM - CROSS SECTION

Jet Grouting

- Jet grouting is a grout injection that cuts and mixes the soil to be treated with cement or cementitious grout.
- Soil particles are cut by the grout jetting under high pressure
- Soil particles are mixed with it to form a matrix
- matrix is impermeable and possesses some kind of strength.

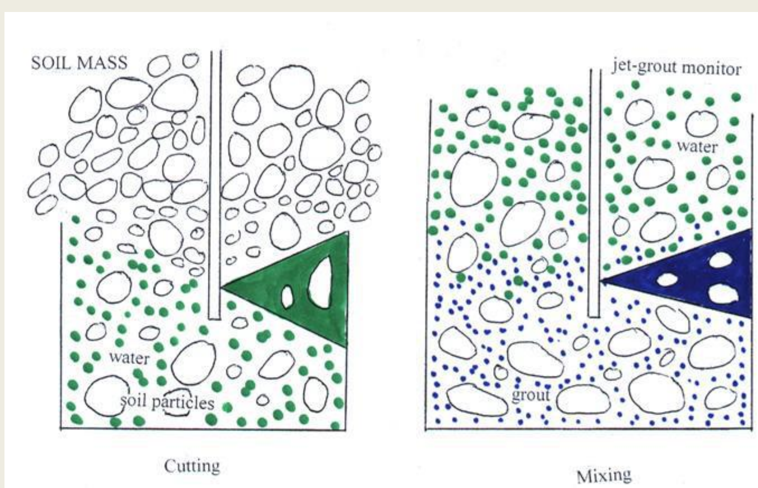
Jet Grouting

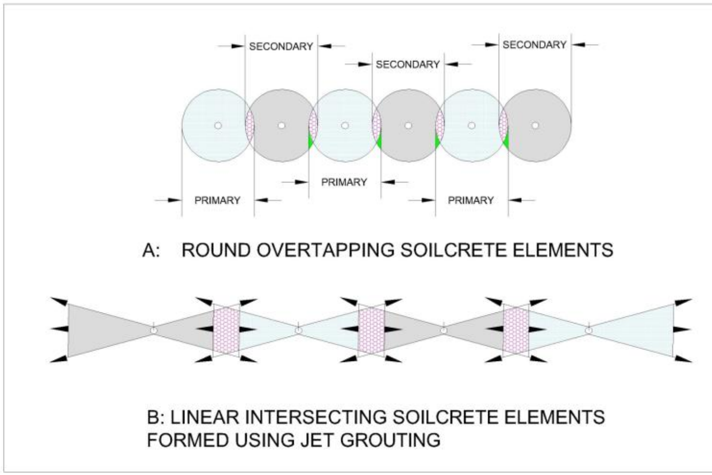
- **Mono-fluid system**: Cement grout is used as desegregating and consolidation fluid. In this the erosion and soil cementation are achieved with the same fluid (Cement gout) forming a jet stream when directed at the ground.

- **Bi-fluid system**: Cement grout plus air are used as desegregating and Consolidation fluid. In this and air jet envelopes the grout jet for improving the grout jet stream erosive efficiency (two fluid employed, grout and air).

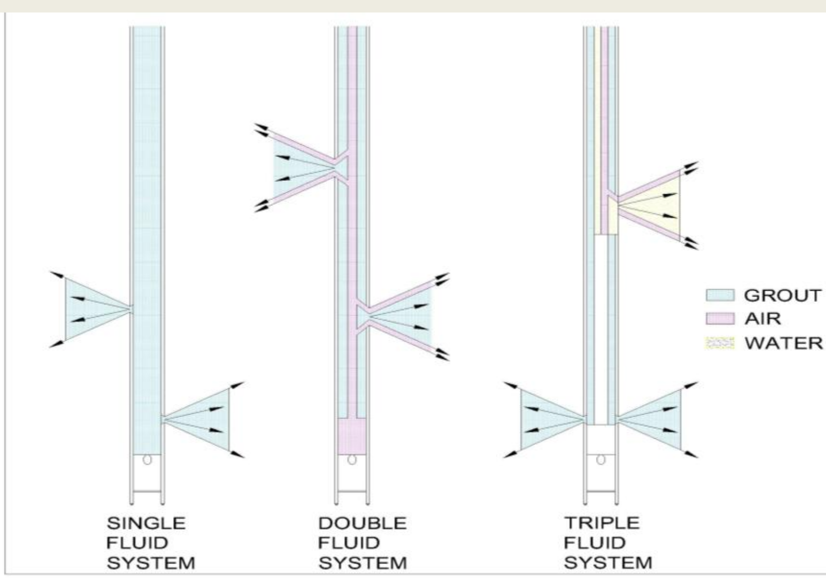
- **Triple-fluid system**: Water plus air used as desegregating fluid while cement grout is used as consolidating fluid. In this erosion is achieved with a double jet of water surrounded by an air jet and cementation is simultaneously obtained by a separate grout jet (three fluid employed, water, air and grout).

Jet Grouting





TYPES OF JET GROUT CUTOFF WALLS-GENERAL PLAN VIEW



SCHEMATIC OF GENERAL JET GROUT SYSTEM

Jet Grouting

Jet Grouting

Jet Grouting

Row of interlocking columns of jet grouted material within the alluvium.

Jet Grouting

- A jet grouted element corresponds to the volume of soil eroded and cemented from a single borehole. When rotating and simultaneously translating the erosive jet stream in the borehole with constant speeds, a column is formed having the shape of a cylinder in homogenous soil. In heterogeneous ground, with obstacles, the jet grouted column obtained has an irregular shape. The column diameter varies all along the borehole as a function of the resistance opposed by the soil to the jet erosion and as a function of the parameters of execution.
- The jet grouted materials is the soil- cement mixture left in the ground by the process, and constituting the body of a jet grouted element after setting. When jetting in sandy soils, the jet grouted material is like a cement mortar.

Jet Grouting

Grouting Scheme Design Considerations

- When the SPT N value is greater than 50, the soil mass is normally classified as not suitable for jet grouting.
- Presence of cobble / boulder will cause ‘Shadowing ‘ effect on cutting and mixing .
- The final strength of the mixed soil mass relates to the purpose of the grouting

Jet Grouting

- Pure cement grout is used for jet grouting to strengthen the ground.
- Compressive strength of soil cement mix ranges from 2 MPa to 25 MPa
- For water sealing purpose, bentonite is usually added to the cement grout
- The longer the cutting time is, the more the original soil mass be displaced and the larger the grout column diameter

Jet Grouting

- Mixing time dictates the extent of mixing of the grout and the soil mass.
- Prolonged mixing will eventually displace all the soil mass and replace it with the grout.
- it is very important to monitor the verticality of the jet columns

Jet Grouting

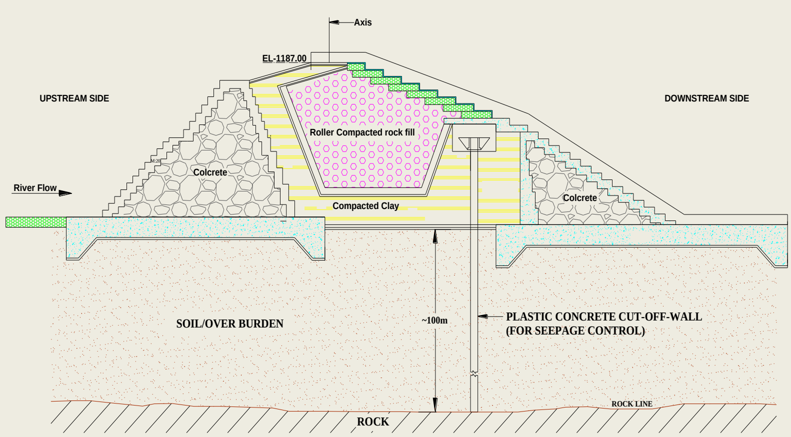
MAIN ADVANTAGES:

- Wide range of soils that can be successfully treated
- Capability to obtain columns of consolidating soil with diameter ranging from 60 to more than 200cm by using small diameter drilled holes, in general not larger than 100 to 140 cm
- Capability to overpass pre-existing foundations boulders, rocky layers
- Use of light weighing and small sized drilling rigs in small working areas

Diaphragm Walls

- A concrete cutoff wall is used as a remedial measure for seepage control.
- Positive cutoff is created by excavating the soil beneath the dam in form of a trench and the resulting void backfilled with a barrier material of well defined properties.
- Plastic concrete consists of aggregate, cement, water, and bentonite clay mixed at a high water cement ratio to produce a ductile material

Diaphragm Walls



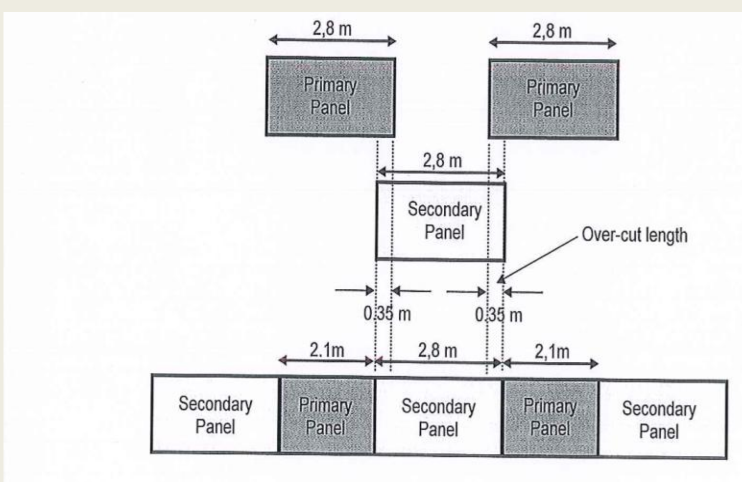
U/S COFFER DAM - CROSS SECTION

Diaphragm Walls

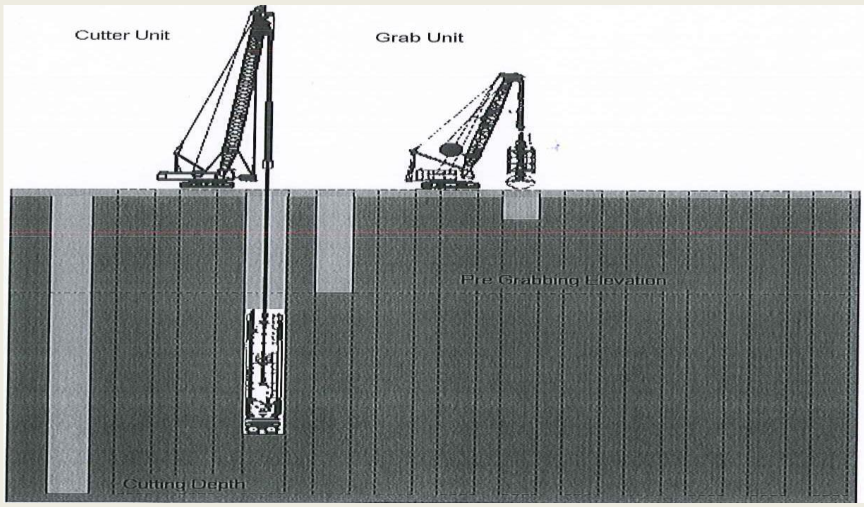
The diaphragm wall is constructed as a series of alternate panels:

- Primary panels
- Secondary panels

Diaphragm Walls

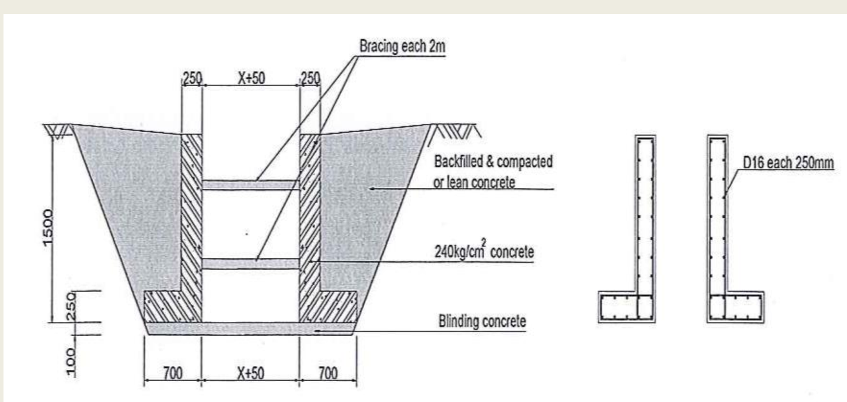


Diaphragm Walls



Diaphragm Walls

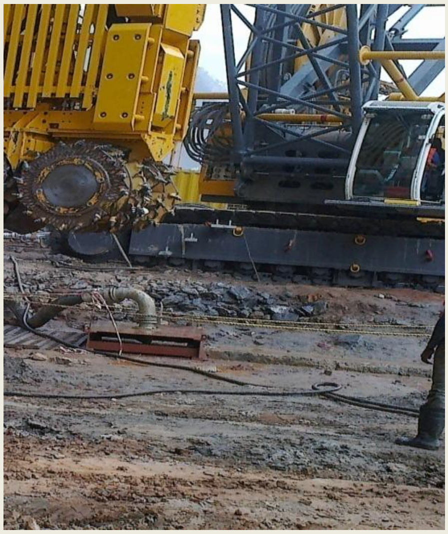
Cut-Off Wall Construction



Diaphragm Walls

Diaphragm Walls

Diaphragm Walls

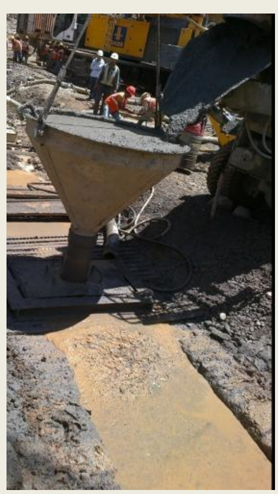


Diaphragm Walls

Plastic Concrete: Cement, Bentonite (1 to 2%), Sand and 10 mm size Aggregates

Diaphragm Walls

Diaphragm Walls



Diaphragm Walls

Dental Treatment

- Process of remove the weak material and reinforcing and stabilizing the weak zone by backfilling with concrete is known as dental treatment.

Dental Treatment

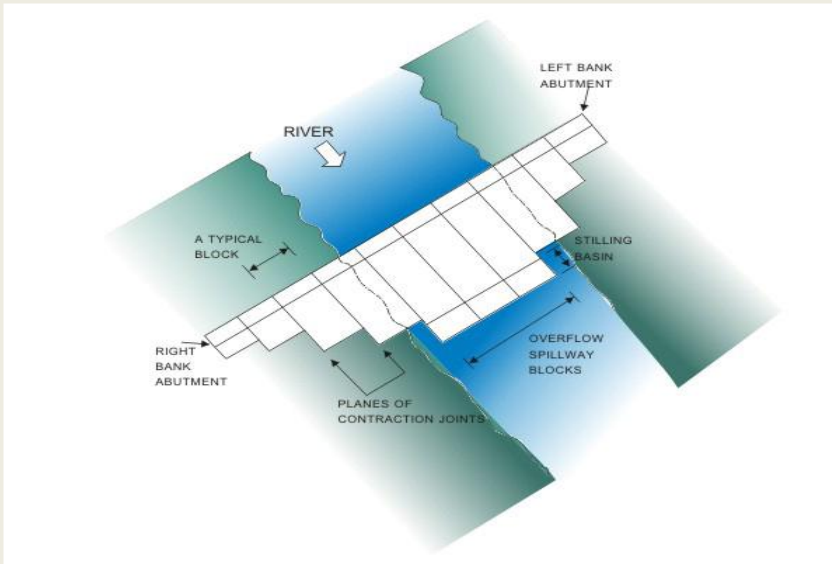
- Final excavation uncovers the faults, seams or inferior rock extending to such depths that it is impracticable to clear such areas.
- It requires to excavate such areas to certain depths and backfill with concrete.
- This procedure of reinforcing and backfilling with concrete is called "Dental Treatment".

Dental Treatment

Shasta and Friant dams Formula

- d = 0.00656 b H + 1.526 (for H> 46 m)
- d = 0.3 b+1.524 (for H < 46 m)
- where,
-
- H = height of dam above general foundation level
- b = width of the weak zone in m
- d = depth of excavation of weak zone below surface of adjoining sound rock

Abutment



Abutment

Abutment slope failure

- Circular Failure
- Planer Failure
- Wedge failure
- Toppling Failure

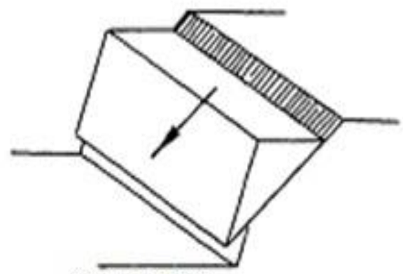
Circular Failure

- Circular failure occurs in waste rock heavily fractured rock and weak rock with no identifiable structural pattern. The failure surface is free to find a line of least resistance through the slope. The slide is controlled by shear strength i.e. cohesion and friction angle

Circular Failure

Plane Failure

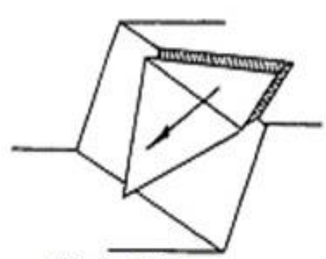
Occurs in rocks with plane discontinuities, e.g., bedding planes.



Plane Failure

Wedge Failure

Occurs in rocks with intersecting discontinuities forming wedges

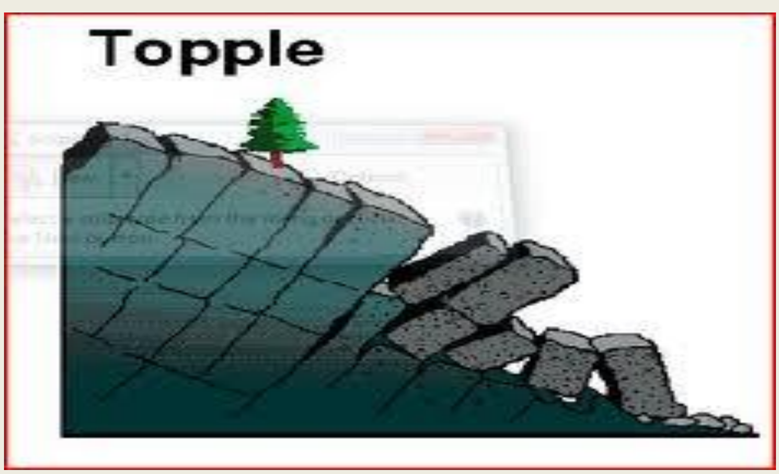


Wedge Failure

Toppling Failure

- Occurs in rocks with columnar or block structures separated by steeply dipping joints. In such failures rock block width and height ratio is less than the gradient of the toppling plane

Toppling Failure



Analysis

- Stability analysis of abutments is usually carried out limit equilibrium methods or by FEM analysis
- PHASE-2’, ‘ABACUS’ and ‘SLIDE’, GEO SLOPES.

Data usually required for analysis are

- Unit weight of rock/soil mass
- Modulus of Elasticity
- Modulus of deformation
- Rock mass classification
- Cohesion
- Friction angle
- Tensile strength
- Poisson ratio
- Slope geometry
- Water table information

Reservoir Rim Stability

- Slopes near the MWL/FRL, are inspected and those slopes which appears to be unstable are marked
- Rock slopes are usually strengthen by shotcreting, cable anchoring and rock bolting while slopes made of river born material (RBM) or colluviums mass are stabilized by geo gridding, turfing, retaining walls , timber piling etc.

Reservoir Rim

Following aspects of the reservoirs have to be properly investigated:-

(a) water tightness of the basins
(b) stability of the reservoir rim
(c) availability of construction material in the reservoir area
(d) silting
(e) direct and indirect submergence of economic mineral wealth and
(f) Seismo-techtonics.

These investigations are carried out by surface and sub-surface exploration of proposed basin.

Investigation Stage

A topographical index map on 1:50 000 scale should be used at this stage to delineate the areas which would require detailed study, subsequently.

Investigation Stage

To prevent an undesirable amount of leakage from the reservoir, the likely zones of such leakage, such as:-

- Major dislocations and
- Previous or cavernous formations running across the divide of the reservoir should be identified

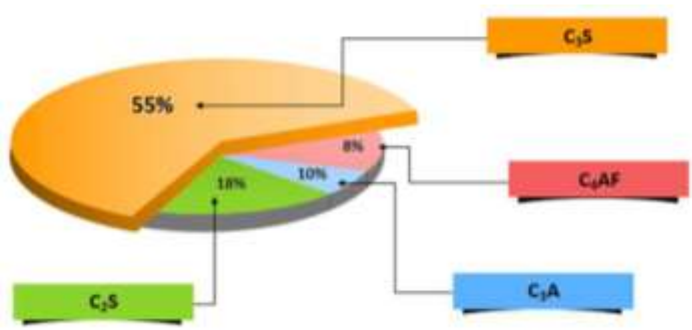
Mass Concrete Definition

Mass concrete is defined by ACI "Any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking."

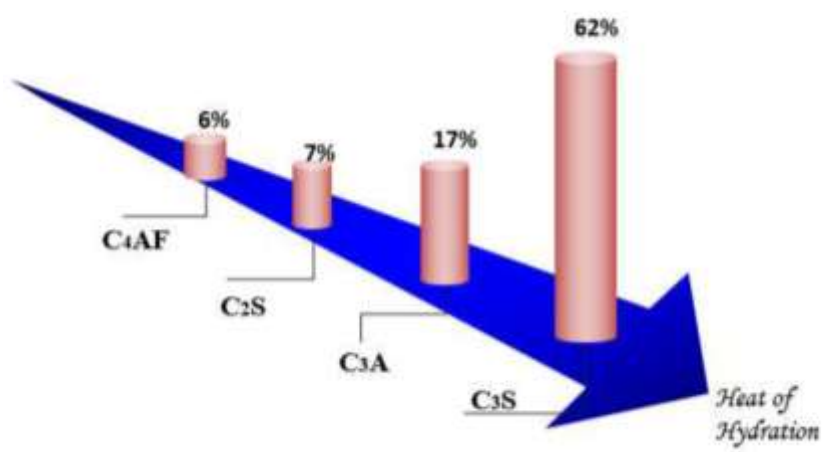
Examples:
- Dam
- Raft Foundation
- Pile Cap.
- Thick Wall.
- Thick column.
- Deep Slap.

- ❖ All concretes generate heat as the cementious materials hydrate.
- ❖ Most of this heat generation occurs in the first days after placement.
- ❖ For thin items such as pavements, heat dissipates almost as quickly as it is generated.
- ❖ For thicker concrete sections (mass concrete), heat dissipates more slowly than it is generated.
- ❖ The net result is that mass concrete can get hot.
- ❖ Management of these temperatures is necessary to prevent damage, minimize delays, and meet project specifications.

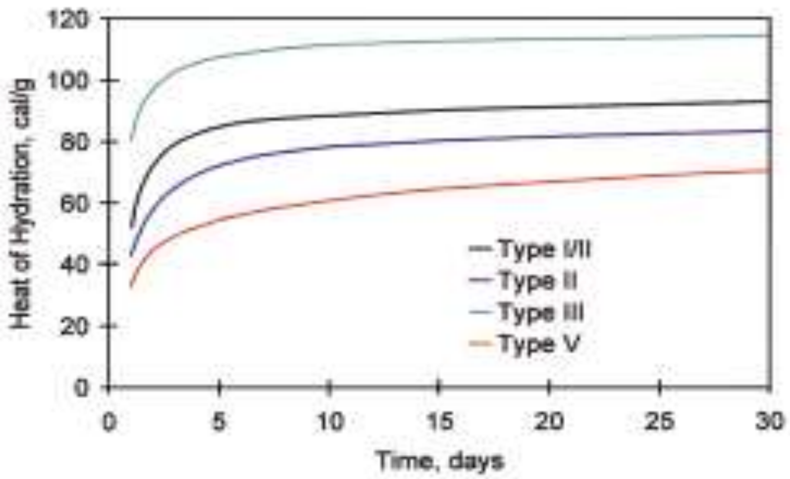
Type-I cement composition



Heat of Hydration of different components



Typical Values of Heat of Hydration



HOH Main Concerns

Differential Temperature:

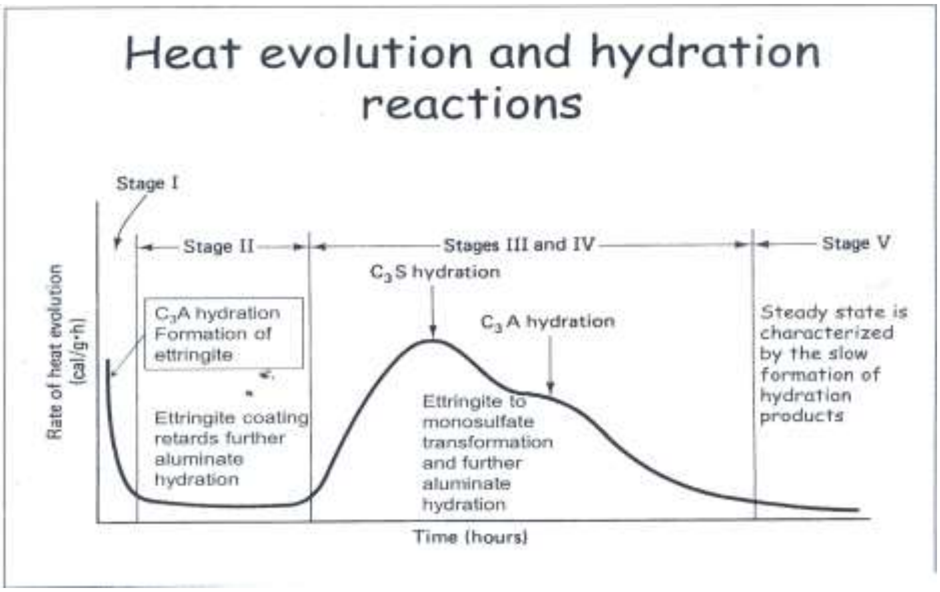
- Crack Concern: Thermal Cracking

Max Temperature:

- Durability Concern: DEF Cracking

"DEF = Delayed Ettringite Formation"

- *Strength Concern : Effects on Ultimate Strength*



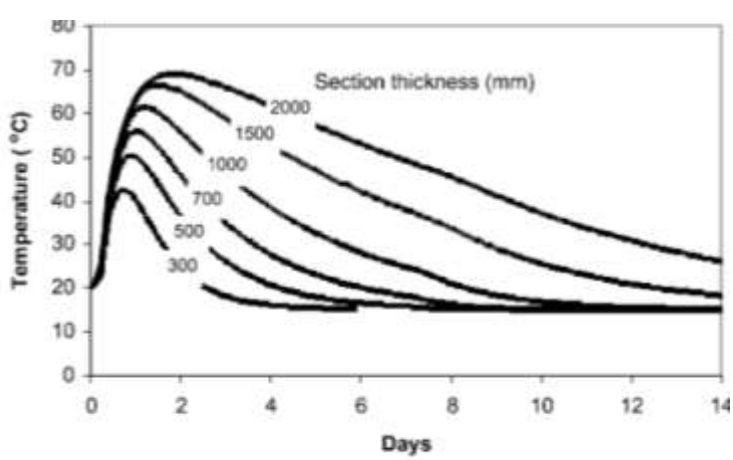
Maximum Temperature (Tmax)

Maximum Temperature "Tmax" is <u>sum</u> of Placement Temperature of Concrete and Temperature Rise due to Heat of Hydration

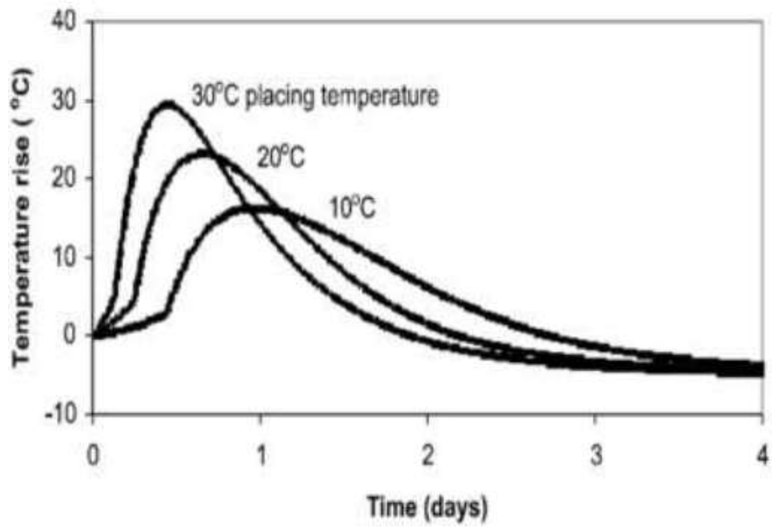
Factors Affecting Maximum Temperature “Tmax” of Concrete

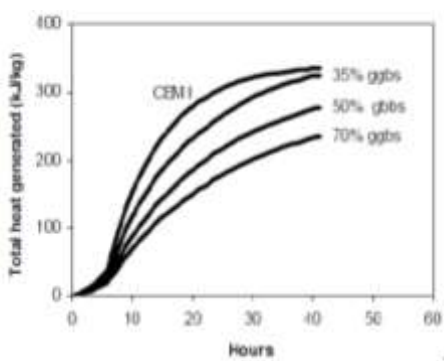
- Cement Content.
- Type and source of cementitious materials.
- Section Thickness.
- Concrete Placing Temperature.
- Formwork and insulation.
- Ambient Temperature.

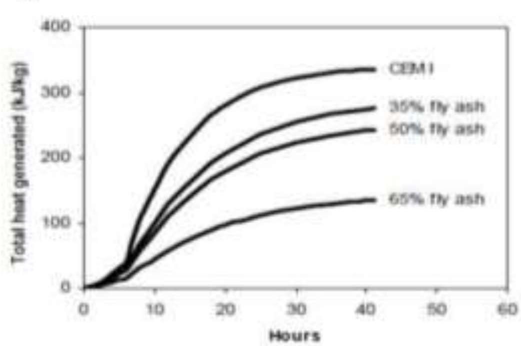
Temperature v/s Section thickness



Temperature v/s placing Temperature







Thermal Cracking

Thermal cracks occur when:

- The tensile stress due to thermal stress is greater than the tensile strength of concrete

OR

Thermal cracks occur when:

- The thermal strain is greater than the tensile strain capacity of the concrete

Thermal Stress Due to Temperature Changes

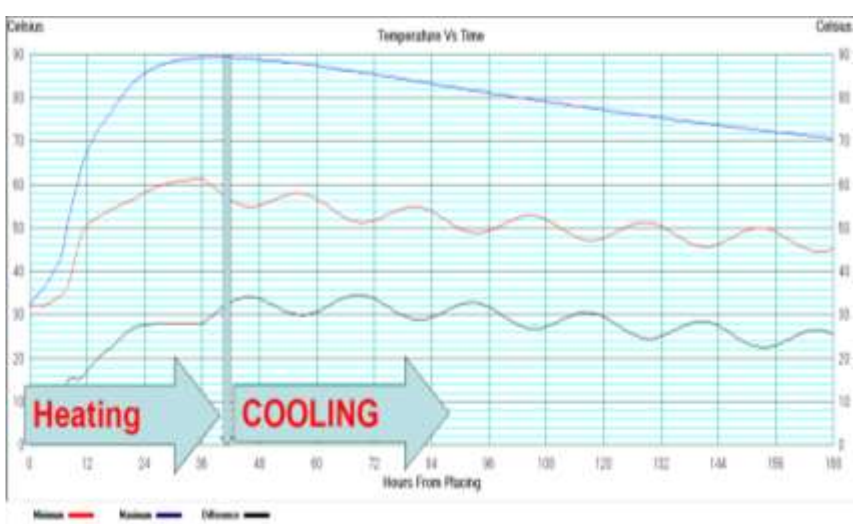
- $f = \alpha E_P R (T_P + T_0 - T_L - T_S)$ ΔT

where

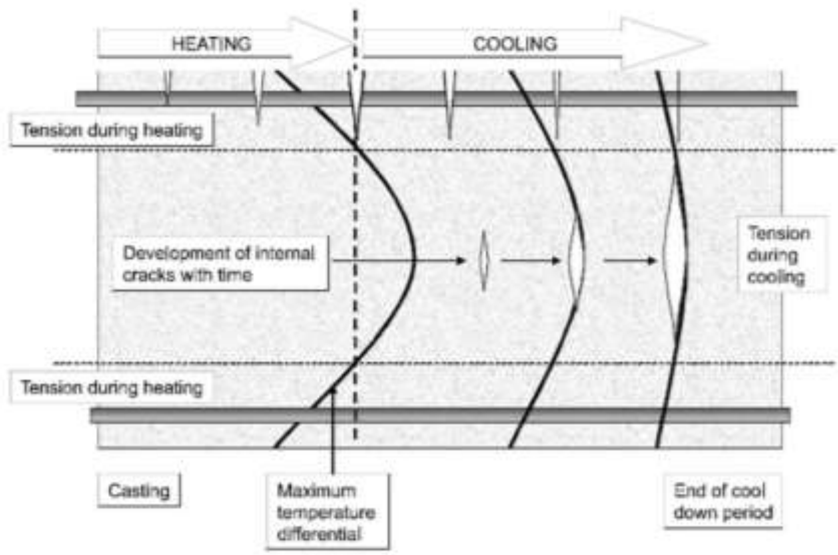
- T_P = Placement temperature of concrete in ^0C
- T_0 = Ultimate adiabatic rise in temperature of concrete in ^0C
- T_L = Temperature loss, i.e. $T_3 + T_2 + T_1$ in ^0C
- T_S = Final stable temperature of dam in ^0C
- α = Coeff of thermal expansion of concrete
- E_P = Sustained modulus of elasticity of concrete in MPa

- This temperature difference is the difference between the temperature at the hottest portion of the concrete and that at the surface.
- Thermal cracking will occur when contraction due to cooling at the surface causes tensile stresses that exceed the tensile strength of the concrete.

Concrete Temp. V/S Time



Development of surface cracks with time



Aggregate Type	Thermal expansion coefficient (microstrain/°C)
Quartzite	14
Gravel	13
Granite	10
Basalt	10
Limestone	9
Marble	7
Lytag (lightweight)	7

External Restraint

- Caused by the foundation rock (or the hardened surface of already cooled concrete) beneath the freshly placed concrete.
- The external restraint is of concern during temperature drop in concrete from the maximum temperature.
- Ratio of the height H to the length L of the concrete block

Internal Restraint

- When different areas of concrete have different temperature change and therefore volume change
- The thermal stress due to internal restraint is induced by differential temperature changes when the surface is cooled by ambient air temperature

Internal Restraint

- Internal restraint is the main cause of transverse vertical cracks on the upstream and downstream faces of dam due to severe ambient conditions. Provision of the transverse joints at suitable spacing serves to control or mitigate this tendency.

Allowable ΔT with Agg. Type

Aggregate type	Allowable ΔT in ° Celsius
Quartz	20
Granite	28
Lime Stone	35
Light weight coarse Agg with natural Sand	53

Typical Temperature rise in concrete (°C/100 kg Cement)

Thickness (m)	OPC (No additive)	GGBS (50%)	Fly Ash (50%)
1.0	12.5	9.5	7.5
1.5	13.5	11.0	8.3
2.0	14.0	11.8	9.0
2.5	14.5	12.1	9.5
3.0	14.8	12.3	9.7

Typical limits of max temp in concrete

- The max temperature at any point within the pour shat not exceed 65-70 °C

WHY......?

To avoid DEF and Negative affect on ultimate strength.

- Use of GGBFS and Fly Ash reduce the negative effects of high temperature on the ultimate strength ofconcrete

- ❖ The concrete temperature at the time of placement has a great impact on the maximum concrete temperature.
- ❖ Typically, for every 1°F (0.6° C) reduction or increase in the initial concrete temperature, the maximum concrete temperature is changed by approximately 1° F (0.6° C).
- ❖ As an example, to reduce the maximum concrete temperature by approximately 10° F (6° C), the concrete temperature at the time of placement should generally be reduced by 10° F (6° C).

According to previous knowledge, general conclusions can be made when using normal hardening cement:

- The 50 °C value is not generally exceeded when:

❖the initial temperature of concrete is < 20 °C,

❖The amount of cement < 350 kg/m^3 and

❖thickness of the structure 0.9 m

Important definitions

- Adiabatic condition: adiabatic environment is the environment perfectly thermally insulated
- Heat is Energy, Heat Quantity is measured by (Kj/Kg), it is Quantity dependent variable
- Heat Flow: Heat flows in the direction of decreasing temperature. (for concrete, generally from the interior to exterior, since the interior tend to be hotter

IQ-Drum

- IQ-Drum is a semi-Adiabatic Calorimeter, a plastic cylinder 150×300mm filled by concrete then place in its place in IQ-drum then connected to thermo-sensor attached to the IQ-drum by a thermocouple.
- IQ-drum measures: (every 15 minutes):

  Sample temperature (measured in °C) and the rate of heat loss from the calorimeter (measured in millivolt) .

IQ Drum

Temperature Control Measures

- Mix design
- Pre-cooling of concrete ingredients
- Artificially cooling the concrete by means of embedded pipe systems.

Precooling

- Keep temperature of the mix water to approximately 0^oC
- Replacing a portion of mix water with ice
- Cooling the Aggregates
- These treatments resulted in a drop not over 10^oC when concrete is placed.
- A drop less than 10^oC have been obtained in colder months only

Mix Design

- Heat generated within mass concrete is directly proportional to the amount of cement
- Mix selected is that which will provide the required strength and durability with the lowest cement content
- Heat of hydration requirements at ages 7 and 28 days are 70 and 80 calories per gram respectively for OPC

Mix Design

- Pozzolana are used in concrete along with cement.
- In place of OPC, PPC is used for production of mass concrete.
- Fly Ash is the most common Pozzolona used in concrete.
- About 40 to 50 percent cement is replaced by fly ash.

Placement Temperature

- The thermal study is carried as per the Indian Standard, “Temperature Control of Concrete for Dams – Guidelines”, IS:14591-1999.
- The loss of heat is calculated separately for the following three idealized conditions and then the total loss is obtained by summation phase losses

Placement Temperature

- The first idealized condition assumes that the placement temperature of the concrete and the atmospheric temperature are the same that is 0°C and the lift is cast upon an inert lift initially at 0°C. The loss of heat as it is generated as given by the equation

  $T_3 = \eta T_0$

η = Ratio of heat lost to the total heat generated.

Placement Temperature

- Under the second idealized condition, the heat loss due to the difference between the placement temperature and atmospheric temperature θ_o
- T_2= calculated as equation given in code

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 ।