UNIT 2 GRAVITY DAMS

Structure

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2.1 INTRODUCTION

In Unit 1 you learnt about the various types of Reservoir, their classification, and their selection. In this unit, you will learn about the stability analysis of a concrete gravity dam.

Objectives

After studying this unit, you will be able to

- discuss the design requirements of a gravity dam,
- design an arbitrary section,
- carry out the stability analysis of a gravity dam,
- explain the purpose of the various joints provided in a dam, and
- explain the need for temperature control of concrete.

2.2 DESIGN REQUIREMENTS

A gravity dam derives its stability from the forces of gravity of the materials within the body of the dam and hence the name. In order to withstand the forces and the overturning moments caused by the water stored in the reservoir, the gravity dam should have sufficient self weight. Good foundations are required for a gravity dam since it transfers the loads to the foundations by cantilever action. There are two types of forces:

1) Forces causing stability :
   a) Self weight of the dam, and
   b) Horizontal thrust of tail water.
2) Forces causing instability :
   a) Water pressure from reservoir,
   b) Uplift pressure,

c) Forces due to waves on the surface of reservoir,

d) Ice pressure,

e) Temperature stresses,

f) Silt pressure,

g) Wind pressure, and

h) Earthquake forces.

Classification of Loading for Design

For purposes of design the loads are taken in certain combinations, and the dam section should be designed so as to develop stresses within the permissible limits. The combinations are:

Normal Loads

They are those, under the combined action of which the dam section should have adequate stability, and the permissible stresses and factors of safety should not be exceeded. These loads are:

a) Weight of the dam and the structure over it,

b) Water pressure corresponding to full reservoir level, and

c) Uplift.

Abnormal Loads

These are loads which in combination with normal loads encroach upon the factor of safety and increase the allowable stresses yet remaining lower than the higher emergency stress limits. They include:

a) Higher water pressure during floods,

b) Wave pressure,

c) Silt pressure,

d) Ice pressure, and

e) Earthquake force.

Load Combinations

The designs should be based on the most adverse combination of "probable" load conditions, but should include only those loads having reasonable probability of simultaneous occurrence. The USBR specifies "Standard" and "Extreme" load combinations as under:

Standard Load Combinations

a) Normal reservoir level, ice and silt (if applicable), and normal uplift.

b) Normal water level, earthquake, silt (if applicable) and normal uplift.

c) Maximum flood water surface elevation, silt (if applicable), and normal uplift.

Extreme Load Combinations

a) Maximum flood water elevation, silt (if applicable), and extreme uplift (i.e. drains choked).

Reservoir Empty Condition

a) Empty reservoir (without earthquake) should be computed for design of reinforcement or grouting studies.

b) Construction stage reservoir empty, earthquake considered but no wind load.

SAQ 1

i) What are the design requirements of a gravity dam?

ii) What do you understand by combination of loads in the design of a gravity dam?

2.3 DESIGN OF AN ARBITRARY PROFILE

The stability conditions required to be met for a gravity dam, subjected to only its self weight, W, force due to water pressure, P, and uplift force, U, can be satisfied by a simple right-angled triangular section (Figure 2.1), with its vertex at the reservoir water level, and which is sufficiently wide at the base where the water pressure is maximum. Such a section is said to be an elementary section of a gravity dam. For the reservoir empty condition, the only force acting on the dam is its self weight whose line of action will meet the base at $b/3$ (b = base width) from the heel of the dam and thus satisfy the stability requirements of no tension. The base width of the arbitrary section is determined for satisfying no tension and no sliding criteria as given below, and the higher of the two base widths is selecteN for the arbitrary profile. For the section shown in Figure 2.1, (assuming width of the dam as one unit normal to the plane of the paper) if one considers that the resultant, R, of all the three

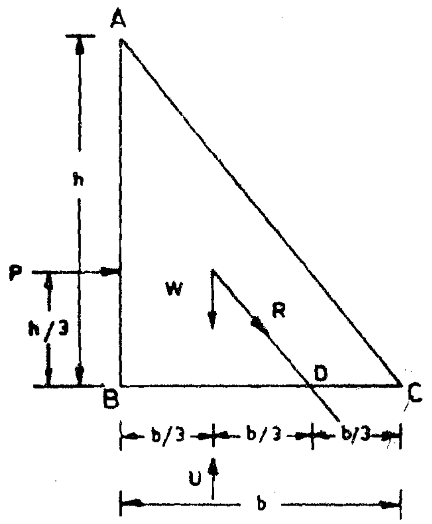

Figure 2.1 : Elementary Profile of a Gravity Dam

forces, W_C (= 0.5 $swbh$), W_1 (= 0.5 wh^2) and U (= 0.5 $wbhc'$) (here s = specific gravity of concrete and c' is a correction factor for uplift force) passes through the downstream middle third point (D), one gets

$$(0.5\ swbh) \times \left(\frac{b}{3}\right) - (0.5\ wh^2) \times \left(\frac{h}{3}\right) - (0.5\ wbhc') \times \left(\frac{b}{3}\right) = 0$$

or $\quad b^2\ (s - c') = h^2$

or $$b = \frac{h}{\sqrt{(s - c')}} \qquad \text{...(2.1)}$$

For $c' = 1$, $b = \dfrac{h}{\sqrt{(s-1)}}$ ...(2.2)

and if uplift is neglected, that is, $c' = 0$,

$$b = \frac{h}{\sqrt{s}} \qquad \text{...(2.3)}$$

For no sliding to occur, $\mu\ (W_C - U) = P$

or $$\mu\ (0.5\ swbh - 0.5\ whbc') = 0.5\ wh^2$$

or $$b = \frac{h}{\mu\ (s - c')} \qquad \text{...(2.4)}$$

For $c' = 1$, $b = \dfrac{h}{\mu\ (s-1)}$ ...(2.5)

and for no uplift, $c' = 0$, and $b = h / (\mu s)$ ...(2.6)

It is clear that for satisfying the requirement of stability, the arbitrary section of a gravity dam should have minimum base width equal to the higher of the base widths obtained from no-sliding and no-tension criteria.

Again, for an arbitrary section,

$$\sum W = (W_c - U)$$

or, $$\sum W = 0.5\ wbh\,(s - c')$$

or, $$\sigma_{yx} = \left(\frac{\sum W}{b}\right)\left[1 \pm 12 \times \frac{ex}{b^2}\right] \qquad ...(2.7)$$

For no tension in the dam, $e = \frac{b}{6}$

Therefore, at the toe of the dam $\left(\text{i.e. } x = \frac{b}{2}\right)$

$$\sigma_{yD} = \frac{2\sum W}{b}$$

$$\sigma_{yD} = wh\,(s - c') \qquad ...(2.8)$$

and at the heel of the dam

$$\sigma_{yU} = 0$$

Accordingly, the principal stress, $\sigma_{1D} = \sigma_{yD} \sec^2 \phi_U$

and, $$\sigma_{1D} = wh\,(s - c')\left[1 + \left(\frac{b}{h}\right)^2\right]$$

$$= wh\,(s - c')\left[1 + \frac{1}{(s - c')}\right]$$

$$= wh\,(s - c' + 1) \qquad ...(2.9)$$

Similarly, $$(\tau_{yx})_D = \sigma_{yD}\ \tan \phi_D$$

$$= \frac{wh\,(s - c')\,b}{h}$$

$$= \frac{wh\,(s - c')}{\sqrt{s - c'}}$$

$$= wh\sqrt{s - c'} \qquad ...(2.10)$$

The principal and the shear stresses at the heel are, obviously, zero.

Similarly, when the reservoir is empty,

$$\sum W = 0.5\ wbhs$$

$$\sigma_{yD} = 0$$

$$\sigma_{1U} = \sigma_{yU} = \frac{2\sum W}{b} = whs \qquad ...(2.11)$$

Sometimes, depending upon whether or not the compressive stress at the toe σ_{1D} exceeds the maximum permissible stress, σ_m, for the material of the dam, a gravity dam is called a "high" or a "low" dam. On this basis, the limiting height, h_l, is obtained by equating the expression for σ_{1D} with σ_m. Thus,

$$\sigma_m = wh_1 (s - c' + 1)$$

or,

$$h_1 = \frac{\sigma_m}{wh (s - c' + 1)} \qquad \text{...(2.12)}$$

If the height of a gravity dam is less than h_1, it is a low dam, otherwise it is a high dam.

SAQ 2

i) What is an arbitrary section of a gravity dam, and how would you design one such section?

ii) When do you term a dam as "low" or "high"?

2.4 STABILITY ANALYSIS

Stress analysis of gravity dams can be carried out by using either of the three methods:

a) The gravity method,

b) The trial load method, and

c) The finite element method.

The first method alone will be discussed here as it is the simplest. The gravity method of stress analysis is applicable to the general case of a gravity dam when its blocks are not made monolithic by keying and grouting the joints between them. All these blocks of the gravity dam act independently and the load is transmitted to the foundation by cantilever action and is resisted by the weight of the cantilever. The following assumptions are made in the gravity method of analysis:

i) The concrete in the dam is a homogeneous, isotropic, and uniformly elastic material,

ii) No differential movements occur at the site of the dam due to the water loads on the walls and base of the reservoir,

iii) All loads are transmitted to the foundation by the gravity action of vertical, parallel cantilevers which receive no support from the adjacent cantilever elements on either side,

iv) Normal stresses on horizontal planes vary uniformly as a straight line from the upstream face to the downstream face, and

v) Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to the downstream face of the dam.

The assumptions at (iv) and (v) above are substantially correct, except for horizontal planes near the base of the dam where the effects of foundation yielding affect the stress distributions in the dam. Such effects are, however, usually small in dams of low or medium height. But these effects may be significant in high dams in which case stresses near the base should be checked by other suitable methods of stress analysis.

As shown in Figure 2.2, $\sum W$ and $\sum H$ represent, respectively, the sum of all the resultant vertical and horizontal forces acting on a horizontal plane (represented by the section PQ) of a gravity dam. The resultant R of $\sum W$ and $\sum H$ intersects the section PQ at O′ while O represents the centroid of the plane under consideration. The distance between O and O′ is called the eccentricity of loading, e. When e is not equal to zero, the loading on the plane is eccentric and the normal stress σ_{yx} at any point x on the section PQ away from the centroid O is given by

$$\sigma_{yx} = \frac{\sum W}{A} \pm \frac{\sum W_{ex}}{I} \qquad \text{...(2.13)}$$

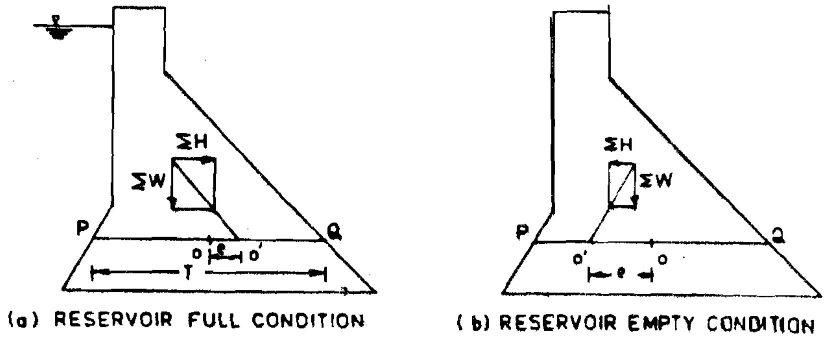

Figure 2.2 : Resultant Force on a Gravity Dam

where A is the area of the plane PQ and I is the moment of inertia of the plane PQ about an axis passing through its centroid and parallel to the length of the dam. It should be noted that whereas the direct stress ($= \sum W/A$) at every point of the section PQ is always compressive, the nature of the bending stress ($= \sum \frac{W_{ex}}{I}$) depends on the location of O′ with respect to O. If O′ lies between O and Q, there will be compressive bending stress for any point between O and Q and tensile stress for any point between O and P. Accordingly, when the reservoir is full, one should use the positive sign in Eq. (2.13) for all points between O and Q and the negative sign for all points between O and P. Similarly, when the reservoir is empty (in which case $\sum H$ may be an earthquake force acting in the upstream direction), and O′ lies between O and P, one should use the positive sign for all points between O and P and the negative sign for all points between O and Q.

Considering unit length of the dam and the horizontal distance between the upstream edge P and the downstream edge Q of the plane PQ as T, one can write $A = T$, and $I = \frac{T^3}{12}$. Thus, Eq. (2.13) reduces to

$$\sigma_{yx} = \left(\sum \frac{W}{T}\right)\left[1 \pm 12 \times \frac{ex}{T^2}\right] \qquad ...(2.14)$$

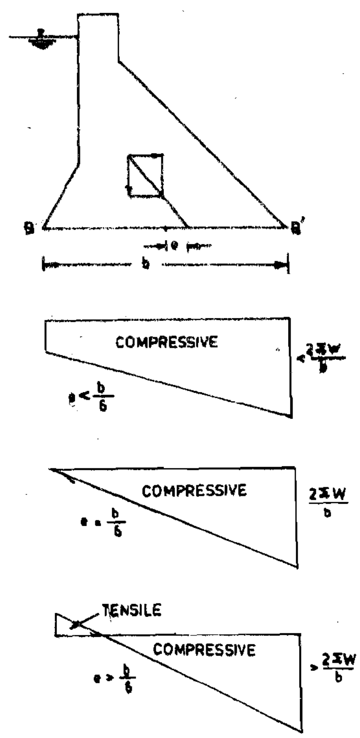

Figure 2.3 : Normal Stresses on the Base of a Gravity Dam

One can use this equation for determining the normal stress on the base of the dam BB′ (Figure 2.3). If the width of the base BB′ is b, Eq. (2.14) for the base of the dam reduces to

$$\sigma_{yx} = \left(\sum\frac{W}{b}\right)\left[1 \pm 12 \times \frac{ex}{b^2}\right] \quad ...(2.15)$$

For the toe (B′) and the heel (B) of the dam, $x = b/2$. Hence, the normal stresses at the toe (σ_{yD}) and the heel (σ_{yU}) of the dam are as follows:

When the reservoir is full,

$$\sigma_{yD} = \left(\sum\frac{W}{b}\right)\left[1 + \frac{6e}{b}\right] \quad ...(2.16)$$

$$\sigma_{yU} = \left(\sum\frac{W}{b}\right)\left[1 - \frac{6e}{b}\right] \quad ...(2.17)$$

When the reservoir is empty,

$$\sigma_{yD} = \left(\sum\frac{W}{b}\right)\left[1 - \frac{6e}{b}\right] \quad ...(2.18)$$

$$\sigma_{yU} = \left(\sum\frac{W}{b}\right)\left[1 + \frac{6e}{b}\right] \quad ...(2.19)$$

These equations indicate that if e is less than or equal to $b/6$, the stress is compressive all along the base and when e is greater than $b/6$ there can be tensile stresses on the base. The stress distribution for different values of e and when the reservoir is full have been shown in Figure 2.3. This means that if there has to be no tension at any point of the base of the dam, the resultant for all conditions of loading must meet the base within the middle-third of the base.

A plane on which only normal stresses act is known as a principal plane. Shear stresses are not present on such a plane. Accordingly, the upstream and the downstream faces of a gravity dam having tailwater are principal planes as the only force acting on these surfaces is on account of water pressure which acts normal to these surfaces. Further, at any point in a structure the principal planes are mutually perpendicular. Therefore, other principal planes would be at right angles to the upstream and the downstream faces of a gravity dam. In an infinitesimal triangular element PQR at the toe of a gravity dam (Figure 2.4), the plane QR

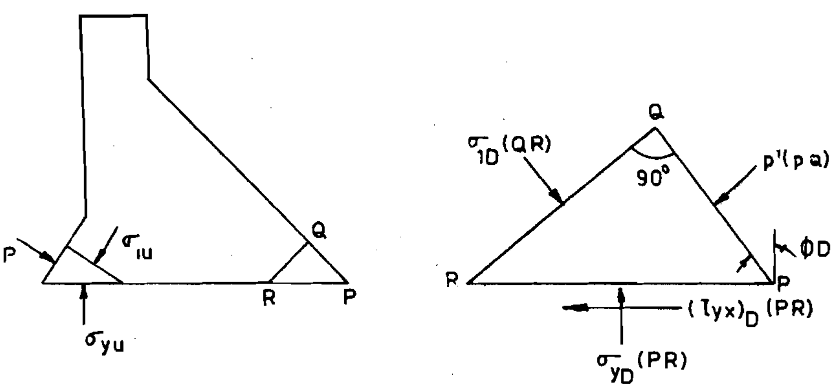

Figure 2.4 : Principal Stresses in a Gravity Dam

is at right angles to the downstream face PQ. Hence PQ and QR are the principal planes and PR is the part of the base of the dam. The stresses acting on the principal planes PQ and QR are, respectively, p' (tailwater pressure) and σ_{1D} as shown in Figure 2.4 and are the principal stresses. The normal and the tangential stresses acting on PR are σ_{yD} and $(\tau_{yx})_D$, respectively. Since the element is very small, the stresses can be considered to be acting at a point. Considering the equilibrium of the element PQR, the algebraic sum of all the forces in the vertical direction should be zero. If one considers the unit length of the dam, then

$$\sigma_{1D}\,(QR) \cos\phi_D + p'\,(PQ) \sin\phi_D - \sigma_{yD}\,(PR) = 0$$

or

$$\sigma_{1D}\,(PR) \cos^2\phi_D + p'\,(PR) \sin^2\phi_D - \sigma_{yD}\,(PR) = 0$$

Therefore,

$$\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D \qquad ...(2.20)$$

Thus, knowing p' and σ_{yD} from Eq. (2.16), one can obtain the principal stress σ_{1D} at the toe of the dam. Usually, p' is either zero (no tailwater) or very small in comparison to σ_{1D}. Therefore, σ_{1D} is the major principal stress and p' is the minor principal stress. When p' is zero, Eq. (2.20) reduces to

$$\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D \qquad ...(2.21)$$

Considering the hydrodynamic pressure p'_e, due to earthquake acceleration (towards the reservoir) the effective minor principal stress becomes $p' - p'_e$, and Eq. (2.20) becomes

$$\sigma_{1D} = \sigma_{yD} \sec^2 \phi_D - (p' - p'_e) \tan^2 \phi_D \qquad ...(2.22)$$

When there is no tailwater, both p' and p'_e are zero and Eq. (2.21) is used for the calculation of σ_{1D}.

Similarly, considering an infinitesimal element at the heel of the dam (Figure 2.4), one can obtain expression for σ_{1U} as follows:

$$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U - (p' + p'_e) \tan^2 \phi_U \qquad ...(2.23)$$

For the condition of empty reservoir, $p = p_e = 0$ and hence

$$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U \qquad ...(2.24)$$

When the reservoir is full, the intensity of water pressure p is usually higher than the normal stress σ_{1U}. Therefore, at the heel, p is the major principal stress and σ_{1U} is the minor principal stress. For vertical upstream face, $\phi_U = 0$ and therefore, σ_{1U} equals σ_{yU}.

Again, resolving the forces acting on the infinitesimal element PQR in the horizontal direction and equating their algebraic sum to zero for the equilibrium condition, one gets

$$(\tau_{yx})_D \ (PR) + p' \ (PQ) \cos \phi_D - \sigma_{1D} \ (QR) \sin \phi_D = 0$$

which gives

$$(\tau_{yx})_D = (\sigma_{1D} - p') \sin \phi_D \,.\, \cos \phi_D$$

$$= (\sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D - p') \sin \phi_D \,.\, \cos \phi_D$$

Therefore,

$$(\tau_{yx})_D = (\sigma_{yD} - p') \tan \phi_D \qquad ...(2.25)$$

Similarly, considering the equilibrium of the element at the heel of the dam,

$$(\tau_{yx})_U = -(\sigma_{yU} - p) \tan \phi_U \qquad ...(2.26)$$

Including the effects of earthquake acceleration, Eqs. (2.25) and (2.26) reduce to

$$(\tau_{yx})_D = [\sigma_{yD} - (p' - p'_e)] \tan \phi_D \qquad ...(2.27)$$

$$\text{and} \qquad (\tau_{yx})_U = -[\sigma_{yu} - (p + p_e)] \tan \phi_U \qquad ...(2.28)$$

In this way, one can calculate the principal stresses at the upstream and the downstream faces of the dam at any horizontal section by considering only the forces acting above the section.

An arbitrary section is only an ideal profile which has to be modified for adoption in actual practice. Modification would mean providing a finite crest width, adequate freeboard, batter in the lower part of the water face and a flatter downstream face. The design of a gravity dam involves assuming its tentative section and then dividing it into a number of zones by horizontal planes for stability analysis at the level of each dividing horizontal plane. The analysis can be either two-dimensional or three-dimensional. The following example shows the two-dimensional method of stability analysis of gravity dams. The three-dimensional analysis being complicated is best done with the help of computers.

Example 2.1

For the section of the gravity dam shown in Figure 2.5, compute principal stresses for normal loading and vertical stresses for extreme loading at the heel and toe of the base of the dam. Also determine factors of safety against overturning and sliding as well as shear-friction factors for safety for "drains operating" and "drains not operating conditions". Other data are as follows:

Sediment deposited to a height of 15 m in the reservoir.

Coefficient of shear friction, $\mu = 0.7$ (normal loading)

$= 0.85$ (extreme loading)

Shear strength at concrete-rock interface, $C = 150$ t/ m^2

Weight density of concrete = 2.4 t/m^3

Weight density of water = 1 t/m^3

Coefficient of horizontal acceleration due to earthquake, $\alpha_h = 0.1$

Coefficient of vertical acceleration due to earthquake, $\alpha_v = 0.05$.

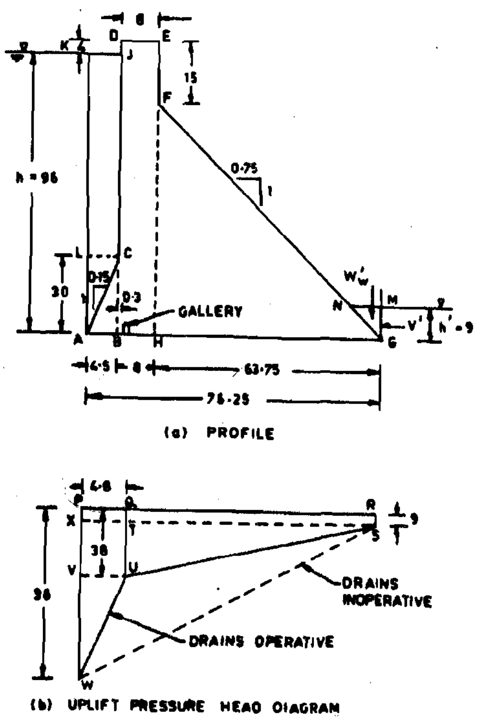

Figure 2.5 : Profile and Uplift Pressure Diagram for the Gravity Dam (Example 2.1)

Solution

Consider a 1 m wide strip of the dam. The computations are shown in Table 2.1.

Computation of Stresses

a) Normal loading combination (normal design reservoir level with appropriate dead loads, uplift (with drains operative), silt, ice, tailwater and thermal loads corresponding to normal temperature):

Resultant vertical force = $\sum W$ = sum of vertical forces at Sl. Nos. 1, 2(a), 3(a) and 4(a) of Table 4.1.

$= (8584.50 + 394.88 - 2000.68 + 15.6) = 6994.31$ t.

Resultant horizontal force = $\sum H$ = sum of horizontal forces at Sl. Nos. 2 (ii) and 2(a) of Table 2.1.

$= (-4567.5 - 40.5) = -4608.00$ t.

Moment about toe of the dam at the base = $\sum M$ = sum of moments at Sl. Nos. 1, 2, 3(a) and 4 of Table 2.1.

$= (418302.75 + 27091.99 - 147334.50 - 96183.57 + 1952.11)$

$= 203828.78$ tm

$$\text{Distance of the resultant from the toe, } y_t = \frac{\sum M}{\sum W}$$

$$= \frac{203828.78}{6994.31} = 29.14 \text{ m}$$

$$\text{Therefore, eccentricity, } e = \frac{76.25}{2} - 29.14 = 8.985 \text{ m} \approx 8.99 \text{ m.}$$

(The resultant passes through the downstream of the centre of the base.)

Using Eqs. (2.16) and (2.17),

$$\sigma_{yD} = \left(\frac{\sum W}{b}\right)\left[1 + \frac{6e}{b}\right] = \left(\frac{6994.31}{76.25}\right)\left[1 + \frac{6 \times 8.99}{76.25}\right]$$

$$= 156.62 \text{ t/m}^2$$

$$\sigma_{yU} = \left(\frac{\sum W}{b}\right)\left[\frac{1 - 6e}{b}\right] = \left(\frac{6994.31}{76.25}\right)\left[1 - \frac{6 \times 8.99}{76.25}\right]$$

$$= 26.84 \text{ t/m}^2$$

Using Eq. (2.20), the major principal stress at the toe, σ_{1D}

$$= \sigma_{yD} \sec^2 \phi_D - p' \tan^2 \phi_D$$

$$= 156.62 \times 1.5625 - 9 \times 0.5625 = 239.66 \text{ t/m}^2$$

Using Eq. (2.25), shear stress at the toe, $(\tau_{yx})_D = (\sigma_{yD} - p') \tan \phi_D$

$= (156.62 - 9) \times 0.75 = 110.72$ t/m^2}

Using Eq. (2.23), with $p_e = 0$, the minor principal stress at the heel,

$\sigma_{1U} = \sigma_{yU} \sec^2 \phi_U - p \tan^2 \phi_U$

$= 26.84 \times 1.0225 - 96 \times 0.0225 = 25.28$ t/m^2

Using Eq. (2.26), shear stress at the heel,

$(\tau_{yx})_U = -(\sigma_{yU} - p) \,.\, \tan \phi_u = -(26.84 - 96) \times 0.15 = 10.37$ t/m^2.

Further, Major principal stress at the heel = p = 96 t/m^2

and Minor principal stress at the toe = p' = 9 t/m^2.

b) Extreme loading combination (normal loading combination and the loading due to earthquake):

The inertial and hydrodynamic forces and corresponding moments due to horizontal earthquake have been computed as shown in Table 2.1. The effect of vertical earthquake can be included in stability computations by multiplying the forces and moments by $(1 - \alpha_V)$ and $(1 + \alpha_V)$ for upward and downward accelerations, respectively. Since the computation of hydrodynamic force involved the use of unit weight of water, the hydrodynamic force will also be modified by vertical acceleration due to earthquake. Further, the effect of earthquake on uplift forces is considered negligible.

Resultant vertical force with downward acceleration

$$= (8584.50 + 394.88 + 15.61) \times 1.05 - 2000.68$$

$$= 7444.06 \text{ t.}$$

Resultant vertical force with upward acceleration

$$= (8584.50 + 394.88 + 15.61) \times 0.95 - 2000.68$$

$$= 6544.56 \text{ t.}$$

Resultant horizontal force with downward acceleration

$$= (4567.50 + 40.50 + 858.45 + 491.19) \times 1.05$$

$$= 6255.52 \text{ t.}$$

Resultant horizontal force with upward acceleration

$$= (4567.50 + 40.50 + 858.45 + 491.19) \times 0.95$$

$$= 5659.76 \text{ t.}$$

Resultant moment about the toe with downward acceleration

$$= (418302.75 + 27091.99 - 147334.50 + 1952.11 - 28183.58 - 19321.38) \times 1.05 - 96183.57$$

$$= 168844.19 \text{ t.}$$

Resultant moment about the toe with upward acceleration

$$= (418302.75 + 27091.99 - 147334.50 + 1952.11 - 28183.58 - 19321.38) \times 0.95 - 96183.57$$

$$= 143603.45 \text{ t.}$$

Now, $y_t = \dfrac{\sum M}{\sum W} = \dfrac{168844.19}{7444.06} = 22.68$ m with downward acceleration

$$= \frac{143603.45}{6544.56}$$

$= 21.94$ m with upward acceleration.

Therefore, Eccentricity, $e = 76.25/2 - 22.68 = 15.45$ m with downward acceleration

$= \dfrac{76.25}{2} - 21.94 = 16.19$ m with upward acceleration.

The resultant in both cases, passes through the downstream side of the centre of the base.

Therefore, the vertical stresses at the toe and the heel with downward acceleration,

$$\sigma_{yD} = \left(\sum W/b\right)\left[1 + \frac{6 \times e}{b}\right] = \left(\frac{7444.06}{76.25}\right)\left(1 + \frac{6 \times 15.45}{76.25}\right)$$

$$= 216.32 \text{ t/m}^2$$

$$\text{and } \sigma_{yU} = \left(\sum W/b\right)\left[1 - \frac{6e}{b}\right] = \left(\frac{7444.06}{76.25}\right)\left[1 - \frac{6 \times 15.45}{76.25}\right]$$

$$= -21.06 \text{ t/m}^2.$$

Similarly, the vertical stresses at the toe and the heel with upward acceleration,

$$\sigma_{yD} = \left(\sum W/b\right)\left[1 + \frac{6\,e}{b}\right] = \left(\frac{6544.56}{76.25}\right)\left[1 + \frac{6 \times 16.19}{76.25}\right]$$

$$= 304.52 \text{ t/m}^2$$

and

$$\sigma_{yU} = \left(\sum W/b\right)\left[1 - \frac{6\,e}{b}\right] = \left(\frac{6544.56}{76.25}\right)\left[1 - \frac{6 \times 16.19}{76.25}\right]$$

$$= -23.52 \text{ t/m}^2$$

It should be noted that the upward acceleration causes higher tensile stresses at the heel and is, therefore, more critical.

Factors of Safety under Normal Loading Combination

a) Factor of safety against overturning $= \dfrac{\text{Stabilising moment}}{\text{Overturning moment}}$

$$= \frac{(418302.75 + 27091.99 + 1952.11)}{(147334.50 + 96183.57)} = 1.84$$

b) Sliding factor $= \dfrac{\sum H}{\sum W} = \dfrac{4608.00}{6994.31} = 0.66.$

c) i) Shear-friction factor of safety (with drains operative),

$$F_S = \frac{(Cb \times 1 + \mu \sum W)}{\sum H}$$

$$= \frac{(150 \times 76.25 + 0.7 \times 6994.31)}{4608.00} = 3.55.$$

ii) Shear-friction factor of safety (with drains inoperative),

$$F_S = \frac{(Cb \times 1 + \mu \sum W)}{\sum H}$$

$$= \frac{(150 \times 76.25 + 0.7 \times 4991.86)}{4608.00} = 3.24.$$

Factors of Safety under Extreme Loading Combination

a) Sliding factor (downward acceleration) $= \dfrac{6255.52}{7444.06} = 0.84.$

b) Sliding factor (upward acceleration) $= \dfrac{5659.76}{6544.56} = 0.865.$

c) Shear-friction factor of safety (downward acceleration),

$$F_S = \frac{(Cb \times 1 + \mu \sum W)}{\sum H}$$

$$= \frac{(150 \times 76.25 + 0.85 \times 7444.06)}{6255.52} = 2.84$$

d) Shear-friction factor of safety (upward acceleration),

$$F_S = \frac{(Cb \times 1 + \mu \sum W)}{\sum H}$$

$$= \frac{(150 \times 76.25 + 0.85 \times 6544.56)}{5659.75} = 3.00.$$

Table 2.1 : Computations of Forces and Moments

Sl. No.	Type of Load	Force (t)	Magnitude of Forces: Vertical (Downward + ve) (t)	Magnitude of Forces: Horizontal (Upstream + ve) (t)	Lever Arm (m)	Moment about Toe (Clockwise –ve) (tm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	Dead load (W_c)					
	ABC	1 × 30× 4.5 × 0.5× 2.4	+ 162.0		73.25	+ 11866.50
	BDEH	1 × 100 × 8 × 2.4	+ 1920.0		67.75	+ 130080.00
	FGH	1 × 85 × 63.75 × 0.5 × 2.4	+ 6502.5		42.50	+ 276356.25
			+ 8584.5			+ 418302.75
2.	Water load					
(i)	Vertical					
	Reservoir (W_w)					
(a)	KJCL	1 × 66 × 4.5 × 1.0	+ 297.00		74.00	+ 21978.00
(b)	ACL	1 × 0.5 × 30 × 4.5 × 1.0	+ 67.50		74.75	+ 5045.63
	Tailwater (W'_w)					
(c)	GMN	1 × 0.5 × 9 × 6.75 × 1.0	+ 30.38		2.25	+ 68.36
			+ 394.88			+ 27091.99
(ii)	Horizontal					
(a)	Reservoir (W_1)	1 × 0.5 × 96 × 96 × 1.0		– 4608.00	32.00	– 147456.00
(b)	Tailwater (W'_1)	1 × 0.5 × 9 × 9 × 1.0		+ 40.50	3.00	+ 121.50
3.	Uplift force, U					
(i)	Drains operative					
(a)	PQUV	1 × 38 × 4.8 × 1.0	– 182.40		73.85	– 13470.24
(b)	UVW	1 × 58 × 4.8 × 0.5 × 1.0	– 139.20		74.65	– 10391.28
(c)	QRST	1 × 9 × 71.45 × 1.0	– 643.05		35.73	– 22976.18
(d)	STU	1 × 29 × 71.45 × 0.5 × 1.0	– 1036.03		47.63	– 49345.87
			– 2000.68			– 96183.57
(ii)	Drains inoperative					
(a)	PRSX	1 × 9 × 76.25 × 1.0	– 686.25		38.13	– 26166.71
(b)	SWX	1 × 0.5 × 87 × 76.25 × 1.0	– 3316.88		50.83	– 168597.01
			– 4003.13			– 194763.72
4.	Silt load (W_s)					
(a)	Excess vertical pressure	1 × 0.5 × 15 × 2.25 × (1.925 – 1.00)	+ 15.61			+ 2357.11
(b)	Excess horizontal pressure	1 × 0.5 × 15 × 15 × (1.36 – 1.00)		– 40.50		– 405.00
			+ 15.61	– 40.50		+ 1952.11

Sl. No.	Type of Load	Force (t)	Magnitude of Forces: Vertical (Downward + ve) (t)	Magnitude of Forces: Horizontal (Upstream + ve) (t)	Lever Arm (m)	Moment about Toe (Clockwise –ve) (tm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
5.	Earthquake forces					
(i)	Inertial horizontal force due to weight of the dam					
(a)	ABC	162×0.1		– 16.20	10.00	– 162.00
(b)	BDEH	1920×0.1		– 192.00	50.00	– 9600.00
(c)	FGH	6502.5×0.1		– 650.25	28.33	– 18421.58
				– 858.45		– 28183.58
(ii)	Hydrodynamic force	At the base, $c = c_m = 0.73$ (for $\phi_U = 0$)				
(a)	Reservoir	$V_{pe} = 0.726\,(0.73 \times 0.1 \times 1.0 \times 96) \times 96$		– 488.43		
		$M_{pe} = 0.299\,(0.73 \times 0.1 \times 1.0 \times 96) \times 96^2$				– 19311.13
(b)	Tailwater	At the base, $c = c_m = 0.47$ (for $\phi_D = \tan^{-1}(0.75)$)				
		$V_{pe} = 0.726\,(0.47 \times 0.1 \times 1.0 \times 9) \times 9$		– 2.76		
		$M_{pe} = 0.299\,(0.47 \times 0.1 \times 1.0 \times 9) \times 9^2$				– 10.25
				– 491.19		– 19321.38

SAQ 3

What are the assumptions made in the gravity method of analysis?

2.5 TOP WIDTH

The top width of a gravity dam is generally fixed by the requirement of a roadway or for access to gate-operating mechanism and the parapet walls. The economical top width of a dam is around 14 per cent of the height of the dam. The usual widths provided vary from 6 - 10 m.

SAQ 4

What is the economical top width of a gravity dam?

2.6 FREEBOARD

The freeboard for the dam should be adequate to avoid overtopping of the dam during maximum flood coupled with waves. Generally, a freeboard allowance of 1.5 times h_w (where h_w is the height of waves), is made. The economical freeboard is around 5 per cent of the height of the dam.

SAQ 5

What is the economical freeboard of a gravity dam? How is the freeboard related to the wave height?

2.7 CONSTRUCTION JOINTS

Concrete in a dam is placed in lifts which are generally 1.5 m high. To develop a proper bond between the lifts, the top surface of the lower surface is freed of all coatings, laitance, stains, defective concrete and all foreign material and the surface is roughened. The construction joint is the surface of the previously placed concrete upon or against which new concrete is to be placed. Besides permitting subsequent placing of concrete, these joints facilitate construction, reduce shrinkage stresses and permit installation of embedments. Suitable measures are adopted to ensure proper bond between the previously placed concrete and the new concrete. They are done by applying a high-velocity sand blast and a water jet. A thin mortar layer is sometimes placed on the surface before placing the new concrete. Construction joints do not require water stops but the top surface is provided with a key while the concrete is still fresh.

2.8 TRANSVERSE AND LONGITUDINAL JOINTS

Most concrete dams are sub-divided into a number of blocks to relieve the thermal stresses and subsequent cracking in the body of the dam. The blocks are formed by transverse and longitudinal joints.

Transverse Joints

For dams transverse joints normal to the dam axis are provided. These joints are 12-18 m apart, usually spacing being 15 m. These joints are introduced to allow the concrete to contract on either side of the joint to relieve thermal stresses. Figure 2.6 shows the transverse and longitudinal joints in Bhakra dam. The transverse joints are vertical and normally extend from the foundation to the top of the dam. Reinforcement bars should not extend across these joints. The edges of the transverse or contraction joints at the face are chamfered to give a pleasing appearance and to avoid spalling. Such chamfers are 4 cm × 4 cm on the non-overflow blocks and 2 cm × 2 cm on the downstream face of the overflow blocks.

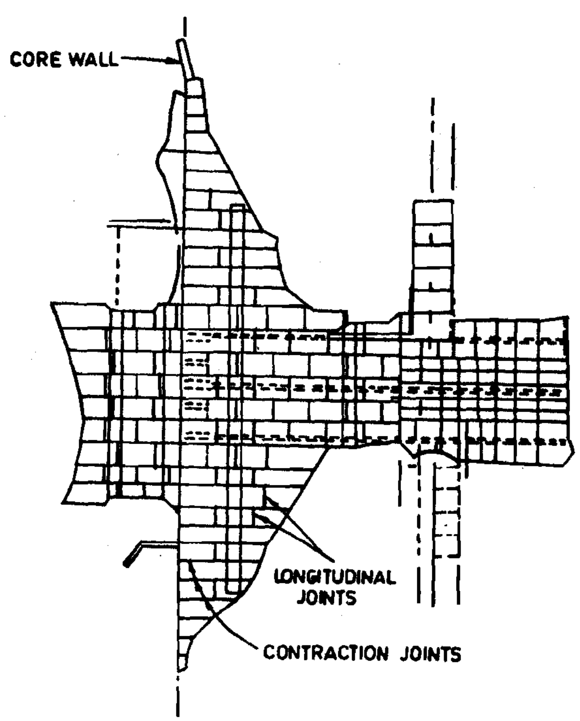

Figure 2.6 : Transverse and Longitudinal Joints in Bhakra Dam

Longitudinal Joints

As the height of the dam increases, the base width approaches a limiting dimension beyond which conditions favouring vertical cracking parallel to the dam axis are created. To prevent uncontrolled cracks, longitudinal joints are provided. They serve the same purpose in one block of the dam as the transverse joints in the dam as a whole. Spacings of these joints vary from 15 - 50 m (Figure 2.6). Where the longitudinal joints approach the downstream face of the dam, the joint is turned normal to the face to avoid feather-edging of concrete. A gap is often provided at the inclined portion of the joint which is later dry packed. Extension of longitudinal joints in the upstream face is undesirable and should be terminated at a minimum distance of 4 - 5 m from the face. These joints are staggered in adjacent blocks.

SAQ 6

i) What is a construction joint in a gravity dam?

ii) What is a transverse joint in a gravity dam?

iii) What is a longitudinal joint in a gravity dam?

2.9 KEY-WAYS

Figure 2.7 shows typical keys for joints in gravity dam. Vertical keys in transverse joints and horizontal keys in longitudinal joints are provided in a dam to increase the shearing resistance between the adjacent concrete blocks. The resulting structure has better stability due to the transfer of load from one block to another through the keys. The keys also increase the percolation path through the joints and thus reduce water leakage. They also hasten the sealing of the joints with sediment deposits. Shear keys provided in longitudinal joints improve the stability of the dam by increasing the resistance to vertical shear.

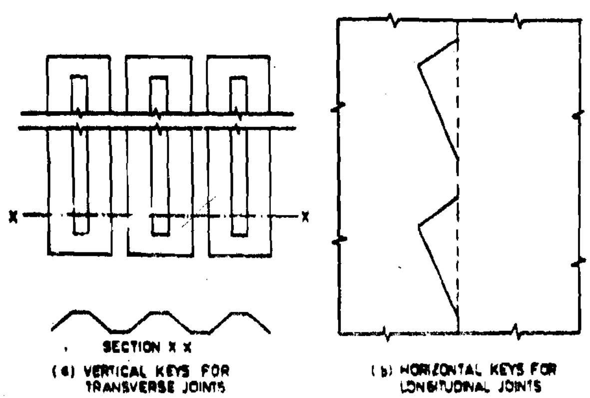

Figure 2.7 : Typical Keys for Joints in Gravity Dam

SAQ 7

What are the various types of key ways? What purpose do they serve?

2.10 WATERSTOPS AT JOINTS

Figure 2.8 shows waterstop installations in a dam. The waterstops are provided in transverse joints for stopping the flow of water into the joint and for stopping the flow of grout outside it. In longitudinal joints the waterstop only retain the grout.

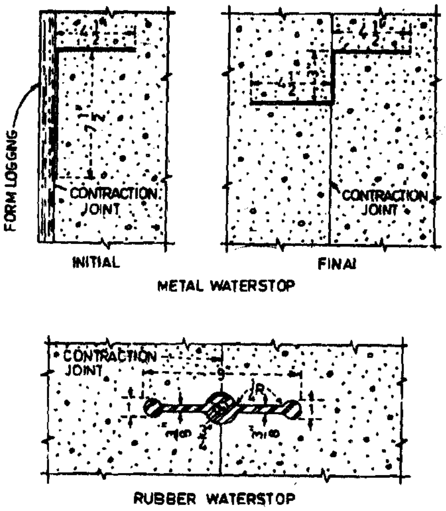

Figure 2.8 : Waterstop Installations

The usual practice is to provide two waterstops of copper or monel (an alloy of nickel and copper) with an asphalt seal in between. In some cases polyvinyl chloride (PVC) and rubber waterstops have been used. The longitudinal joints are provided with Z type while the transverse joints are provided with U or M type seals. Construction joints are sometimes provided with A or Z type seal to prevent seepage along the joint when some opening or gallery is located close to the face. The distance of the first seal from the upstream face in ungrouted contraction joints is about 60 cm.

The pipes inside the asphalt seal are installed for melting the asphalt and adding more asphalt at a later date. Another metal seal is provided downstream of the asphalt seal to limit the travel of asphalt along the joint between the two seals. Further downstream, open drains (called formed drains) about 15 – 20 cm in diameter at 3 m centre to centre are provided. Figure 4.8 shows some of the waterstops.

SAQ 8

What are the various waterstops provided in a gravity dam?

2.11 TEMPERATURE CONTROL

A decrease in temperature of concrete causes volumetric changes resulting in the development of tensile stresses and consequent cracking in the concrete mass. Such cracking in concrete dams is undesirable as it adversely affects their water tightness, internal stresses, durability and appearance. Temperature cracking can be reduced by adopting suitable design and construction procedure. Measures of temperature control facilitate construction and minimise cracks in the concrete. With temperature control measures, it is possible to use large construction blocks which result in rapid and economical construction. The methods of temperature control of concrete include precooling of the concrete, employing an embedded pipe cooling system, use of lesser amounts of normal cement, use of special low heat cement, and use of pozzolanas (such as calcined clays, flyash and pumicites) to replace part of the cement in the concrete. Besides, shallow construction lifts, water curing, use of retarding agents to reduce the early rate of heat generation of the cement and surface treatment are also useful methods of reducing temperature cracking.

SAQ 9

What is the necessity of temperature control in a gravity dam? How does thermal cracking occur?

2.12 CAUSES OF FAILURE OF A GRAVITY DAM

For a gravity dam to be safe against overturning, the dimensions of the dam should be such that the resultant of all forces intersects the base of the dam within its middle-third. Consider the base of a gravity dam or any horizontal section and the resultant of all the forces acting on the dam above the section. If the line of action of this resultant passes outside the toe, the dam would overturn. But, if the section of a gravity dam is such that the line of action of the resultant force is within the upstream and downstream edges of the section, overturning would never occur. However, if the line of action of the resultant passes sufficiently outside the middle third of the horizontal section, it may cause crushing of the toe. This would reduce the effective width and hence the sliding resistance of the section and may cause the resultant to pass outside the dam section. Also when the resultant passes downstream of the middle-third of the horizontal section, it induces tensile stresses at the heel of the section. These tensile stresses may cause cracks in the dam section which would result in increased uplift pressure. The stabilising forces would, thus, be reduced. Thus, it follows that before a gravity dam overturns bodily, other types of failures, such as crushing of the toe, sliding, cracking due to tension, and increase in uplift may occur. A gravity dam is safe against overturning if

a) no tension occurs at the heel,

b) adequate resistance to sliding is available, and

c) quality and strength of concrete and the foundation is satisfactory.

Concrete and masonry are weak in tension. So the design of a gravity section should ensure that no tensile stresses occur anywhere in the dam section. In very high dams, small tensile stresses not more than 50 N/ cm^2 may be permitted.

The horizontal forces acting on a dam above any horizontal plane cause failure of the dam due to sliding if these actuating forces are more than the resistance to sliding on the plane. The resistance to sliding is due to frictional resistance and the shearing strength of the material along the plane under consideration. The shear-friction factor of safety, F_S, which is a measure of stability against sliding or shearing, is given by

$$F_S = \frac{[CA + \sum W\mu]}{\sum H} \qquad \text{...(2.29)}$$

where,

C = unit cohesion,

A = area of the plane considered,

$\sum W$ = sum of all vertical forces acting on the plane,

μ = coefficient of internal friction, and

$\sum H$ = sum of driving shear forces.

The shear-friction factor of safety can be used to determine the stability against sliding or shearing at any horizontal section within the dam, its contact with the foundation or through the foundation along any plane of weakness. The minimum allowable factor of safety, F_S, for gravity dams are 3.0, 2.0 and 1.0 for the usual, unusual and extreme loading conditions, respectively. The value of F_S for any plane of weakness within the foundation should not be less than 4.0, 2.7 and 1.3 for the usual, unusual and extreme loading conditions, respectively.

The maximum allowable compressive stress for concrete in a gravity dam should be less than the specified compressive strength of the concrete divided by 3.0, 2.0 and 1.0 for the usual, unusual and extreme loading conditions, respectively. The compressive stress should not exceed 1035 N/sq cm and 1550 N/sq cm for the usual, and unusual loading conditions, respectively.

The maximum allowable compressive stress in the foundation should be less than the specified compressive strength of the foundation divided by 4.0, 2.7 and 1.3 for the usual, unusual and extreme loading conditions, respectively. These values of factor of safety are higher than those for concrete so as to provide for uncertainties in estimating the foundation properties.

What are the causes of failure of a gravity dam?

What are the allowable factors for the dam and the foundation for different [illegible]

2.13 SUMMARY

The design of gravity dams involves the determination of the normal and principal stresses at the heel and the toe with combination of forces considered for normal loading and for earthquake conditions. Factors of safety against sliding and the shear friction factors are also to be determined taking the arbitrary profile as the preliminary section. The provision of construction joints, transverse and longitudinal joints and waterstops at the joints are essential for the safety of the dam and to reduce seepage losses. Temperature stresses are detrimental to the dam and measures have to be adopted to minimise these stresses. These aspects have been covered in this unit.

2.14 KEY WORDS

Abnormal Loads : They include higher water pressure due to floods, wave pressure, silt pressure, ice pressure and earthquake force.

Construction Joints : Concrete in a dam is laid in lifts. These lifts have a nearly flat surface on the top. The subsequent lift of concrete is poured on the previously laid concrete layer. The joint formed at the junction of the two lifts is called a construction joint.

Earthquake Forces : When an earthquake occurs the structure is subject to additional forces for which the dam section should be designed to withstand these earthquake forces.

Extreme Load Combination : It includes extreme uplift corresponding to drains choked, water surface at the maximum flood level and silt pressure.

Forces Causing Instability : These include uplift pressure, water pressure from the reservoir, earthquake forces, forces due to waves on the water surface, ice pressure, temperature stresses, silt pressure and wind pressure.

Forces due to Waves on the Surface of Reservoir : Wind causes waves to rise on the water surface of the reservoir. These waves exert a force on the upstream face of the dam.

Freeboard : It is the extra height provided in a dam above the maximum flood level together with waves as a protection against overtopping.

Horizontal Thrust of Tail Water : Water on the downstream face of the dam exerts a horizontal forces in the upstream direction.

Ice Pressure : In cold climates the top surface of the reservoir freezes to form sheets of ice. As the rim of the reservoir and the upstream face of the dam restrict the ice from expanding, a pressure is exerted on the dam which is called ice pressure..

Key-ways : Adjacent concrete blocks of a dam are likely to slide due to shear. Keys increase the shearing resistance between the dam blocks. Transverse joints are provided with vertical keys and longitudinal joints with horizontal keys.

Longitudinal Joints	:	To prevent uncontrolled cracking of a dam block, longitudinal joints are provided in each block normal to the direction of flow. The joints are staggered in adjacent blocks.
Normal Loads	:	They include the weight of the dam and the structure at its top, water pressure with full reservoir and uplift pressure.
Self Weight of the Dam	:	The weight of the concrete dam and the gate and bridge structure constitute the self weight of the dam.
Silt Pressure	:	The sediment trapped in the reservoir is confined to the lowest portions of the reservoir. This silt and sediment exert a pressure on the face of the dam called the silt pressure.
Stability Analysis	:	The checking of the dam section for safety against all destabilising forces is stability analysis.
Standard Load Combinations	:	This is the load combination recommended by USBR as occurring simultaneously with a reasonable probability of occurrence.
Temperature Control	:	Tensile stresses occur in a dam section when the temperature falls leading to the cracking of concrete. Temperature control is necessary to reduce the chances of such cracking.
Temperature Stresses	:	Stresses in concrete caused as a result of temperature falling or rising are termed as temperature stresses.
Top Width	:	It is the width of the dam at the top required for a roadway, gate operating mechanism and the parapet walls.
Transverse Joints	:	The joints provided normal to the dam axis to allow the concrete to contract on either side of the joint to relieve the thermal stresses are transverse joints. These joints are provided between two adjacent dam blocks. Differential settlement of adjacent blocks can occur independent of each other due to the transverse joints.
Uplift Pressure	:	Water seeping through the foundation of a dam will exert an upward pressure on the base of the dam. Such pressure causes instability of the dam structure.
Water Pressure from Reservoir	:	Water in the reservoir will exert a horizontal pressure on the upstream face of the dam. The pressure varies from zero at the top to a maximum at the bottom and equal to the product of the depth of water and the unit weight of water.
Waterstops at Joints	:	In order to prevent the loss of water through the joint between adjacent blocks, water stops are necessary. These are provided on the upstream face of the dam.
Wind Pressure	:	Wind pressure is neglecting in most cases.

2.15 ANSWERS TO SAQs

SAQ 2

ii) Low when dam height is less than $h_1 = \dfrac{\sigma_m}{[wh\,(s - c' + 1)]}$ and high when dam height is more than $h_1 = \dfrac{\sigma_m}{[wh\,(s - c' + 1)]}$.

SAQ 3

i) The concrete in the dam is a homogeneous, isotropic, and uniformly elastic material,

ii) No differential movements occur at the site of the dam due to the water loads on the walls and base of the reservoir,

iii) All loads are transmitted to the foundation by the gravity action of vertical, parallel cantilevers which receive no support from the adjacent cantilever elements on either side,

iv) Normal stresses on horizontal planes vary uniformly as a straight line from the upstream face to the downstream face, and

v) Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to the downstream face of the dam.

SAQ 4

The economical top width of a dam is around 14 per cent of the height of the dam. The usual widths provided vary from 6 – 10 m.

SAQ 5

The economical freeboard is around 5 per cent of the height of the dam. Generally, a freeboard allowance of 1.5 times h_w (where h_w is the height of waves) is made.