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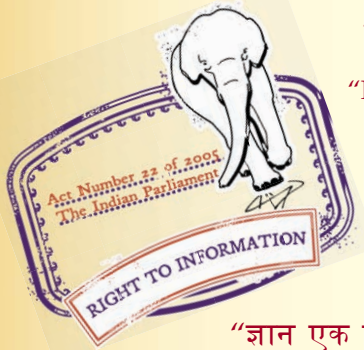
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IS 6512 (1984): Criteria for design of solid gravity dams
[WRD 9: Dams and Spillways]



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“Knowledge is such a treasure which cannot be stolen”

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(Reaffirmed 1990)

Indian Standard

CRITERIA FOR
DESIGN OF SOLID GRAVITY DAMS

(First Revision)

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MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
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Indian Standard

CRITERIA FOR DESIGN OF SOLID GRAVITY DAMS

(*First Revision*)

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Indian Standard

CRITERIA FOR DESIGN OF SOLID GRAVITY DAMS

(First Revision)

0. FOREWORD

0.1 This Indian Standard (First Revision) was adopted by the Indian Standards Institution on 10 September 1984, after the draft finalized by Dams (Overflow and Non-overflow) Sectional Committee, had been approved by the Civil Engineering Division Council.

0.2 Designs are made more rational by utilizing fully the available data from analytical procedures, laboratory and field investigations and measurements of the behaviour of structures in service. It is essential that all design loads are carefully chosen to represent, as nearly as can be determined, the actual loads that will act on the structure and that all the resistive forces used in design represent as accurate an evaluation as possible. It is, in addition, necessary that dams be frequently inspected (as in all cases, uncertainties exist regarding such factors as loads, resistive forces or characteristics of the foundation) and adequate observations and measurements be made of the structural behaviour of the dam and the foundation to ensure that the structure is safe at all times.

0.3 As no criterion may be applicable under all conditions, the criteria laid down in this standard are preceded by a discussion of the underlying considerations to explain the basis for the criterion specified and to serve as a guide in appraising the applicability of the criterion to any special condition or to the appropriateness of any deviation. This also helps in the frequent examination of design practices essential to ensure that the practices remain consistent with the best existing knowledge and concepts, by allowing revisions needed in the light of fresh data. At any time, the criteria are meant to be supplemented with the latest available knowledge.

0.4 This standard was first published in 1972. Since then more dams have been designed and constructed in the country and more experience has been gained. The standard is, therefore, proposed to be revised to reflect the latest practices. Important changes affected in the standard include the following:

- a) Methods and formula for computing wave height and free board have been modified;

- b) Requirement for minimum free board has been modified; and
- c) Permissible factor of safety has been related to the concept of partial factor of safety concerning friction and cohesion, and their values specified.

0.5 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the results of a test, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard lays down criteria for design of solid gravity dams made of masonry or concrete or both.

2. REQUIREMENTS FOR STABILITY

2.1 The design shall satisfy the following requirements of stability:

- a) The dam shall be safe against sliding on any plane or combination of planes within the dam, at the foundation or within the foundation;
- b) The dam shall be safe against overturning at any plane within the dam, at the base, or at any plane below the base; and
- c) The safe unit stresses in the concrete or masonry of the dam or in the foundation material shall not be exceeded.

3. FORCES CONSIDERED IN THE ANALYSIS OF STABILITY

3.1 The following forces may be considered as affecting the design:

- a) Dead load,
- b) Reservoir and tailwater loads,
- c) Uplift pressure,
- d) Earthquake forces,
- e) Earth and silt pressures,
- f) Ice pressure,
- g) Wind pressure,
- h) Wave pressure, and
- j) Thermal loads.

*Rules for rounding off numerical values (*revised*)

3.2 The forces to be resisted by a gravity dam fall into two categories as given below:

- a) Forces, such as weight of the dam and water pressure, which are directly calculable from the unit weights of the materials and properties of fluid pressures; and
- b) Forces, such as uplift, earthquake loads, silt pressure and ice pressure, which can only be assumed on the basis of assumption of varying degree of reliability.

NOTE — It is in the estimating of the second category of the forces that special care has to be taken and reliance placed on available data, experience, and judgement.

3.3 For consideration of stability the following assumptions are made:

- a) 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 (*see 7.1.3*).

NOTE — However, in the stability analysis of a gravity dam, it becomes frequently necessary to make an analysis of the whole block, wherever special features of foundation and large openings so indicate.

- b) That the vertical stress varies linearly from upstream face to downstream face on any horizontal section.

4. LOAD COMBINATIONS

4.1 Criteria — Gravity dam design should be based on the most adverse load combination A, B, C, D, E, F or G given below using the safety factors prescribed. Depending on the scope and details of the various project components, site conditions and construction programme one or more of the following loading combinations may not be applicable *ipso-facto* and may need suitable modifications:

- 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 normal dry weather tailwater, normal uplift; ice and silt (if applicable).
- c) *Load Combination C (Flood Discharge Condition)* — Reservoir at maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift, and silt (if applicable).
- d) *Load Combination D* — Combination A, with earthquake.
- e) *Load Combination E* — Combination B, with earthquake but no ice.
- f) *Load Combination F* — Combination C, but with extreme uplift (drains inoperative).
- g) *Load Combination G* — Combination E, but with extreme uplift (drains inoperative).

5. DESIGN CRITERIA

5.1 Dead Load

5.1.1 General — The dead load to be considered comprises the weight of the concrete or masonry or both plus the weight of such appurtenances as piers, gates and bridges. The unit weight of concrete and masonry varies considerably depending upon the various materials that go to make them. It is essential to make certain that the assumed unit weight for concrete/masonry or both can be obtained with the available aggregates/stones.

5.1.2 Criteria for Design

5.1.2.1 The magnitude of dead load is considered as the mass of concrete or masonry or both plus that of the appurtenances, such as gates and bridges.

5.1.2.2 For preliminary designs, the unit mass of concrete and masonry may be taken as 2 400 kg/m³ and 2 300 kg/m³, respectively. For final designs the unit mass shall be based on actual test data.

5.2 Reservoir and Tailwater Loads

5.2.1 General — Although the weight of water varies slightly with temperature, the variation is usually ignored. In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and will deserve consideration. The mass of the water flowing over the top of the spillway is not considered in the analysis since the water usually approaches spouting velocity and exerts little pressure on the spillway crest. If gates or other control features are used on the crest they are treated as part of the dam so far as application of water pressure is concerned.

5.2.2 Criteria for Design — The mass of water is taken as 1 000 kg/m³. Linear distribution of the static water pressure acting normal to the face of the dam is assumed. Tailwater pressure adjusted for any retrogression should be taken at full value for non-overflow sections and at a reduced value for overflow sections depending on the type of energy dissipation arrangement adopted and anticipated water surface profile downstream. The full value of corresponding tailwater should, however, be used in the case of uplift.

5.3 Uplift Pressures

5.3.1 Uplift forces occur as internal pressures in pores, cracks and seams within the body of the dam, at the contact between the dam and its foundation and within the foundation. It is recognized that there are

two constituent elements in uplift pressure; the area factor or the percentage of the area on which uplift acts and the intensity factor or the ratio which the actual intensity of uplift pressure bears to the intensity gradient extending from head water to tailwater at various points.

5.3.1.1 Effective downstream drainage, whether natural or artificial, will generally limit the uplift at the toe of the dam to tailwater pressure. Formed drains in the body of the dam and drainage holes drilled subsequent to grouting in the foundation, where maintained in good repair, are effective in giving a partial relief to the uplift pressure intensities under and in the body of the dam. The degree of effectiveness of the system will depend upon the character of the foundation and the dependability of the effective maintenance of the drainage system. In case observations of the behaviour of the dam will indicate the uplift pressures actually acting on the structure and when the uplift pressures are seen to approach or exceed design pressures, prompt remedial measures should necessarily be taken to reduce the uplift pressures to values below the design pressures.

5.3.2 Criteria for Design — The following criteria are recommended for calculating uplift forces:

- a) Uplift pressure distribution in the body of the dam shall be assumed, in case of both preliminary and final designs, to have an intensity which at the line of the formed drains exceeds the tailwater head by one-third the differential between reservoir level and tailwater level. The pressure gradient shall then be extended linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 percent of the area.
- b) Uplift pressure distribution at the contact plane between the dam and its foundation, and within the foundation shall be assumed for preliminary designs to have an intensity which at the line of drains/drainage holes exceeds the tailwater head by one-third the differential between the reservoir and tailwater heads. The pressure gradient shall then be extended linearly to heads corresponding to reservoir level and tailwater level. The uplift shall be assumed to act over 100 percent of the area (Fig. 1). For final designs, the uplift criteria in case of dams founded on compact and unfissured rock shall be as specified above. In case of highly jointed and broken foundation, however, the pressure distribution may be required to be based on special methods of analysis taking into consideration the foundation condition after the treatment proposed. The uplift shall be assumed to act over 100 percent of the area.

- c) For the loading conditions F and G (see 4.1) the uplift shall be taken as varying linearly from the appropriate reservoir water pressure at the upstream face to the appropriate tailwater pressure at the downstream face. The uplift is assumed to act over 100 percent of the area (Fig. 2).

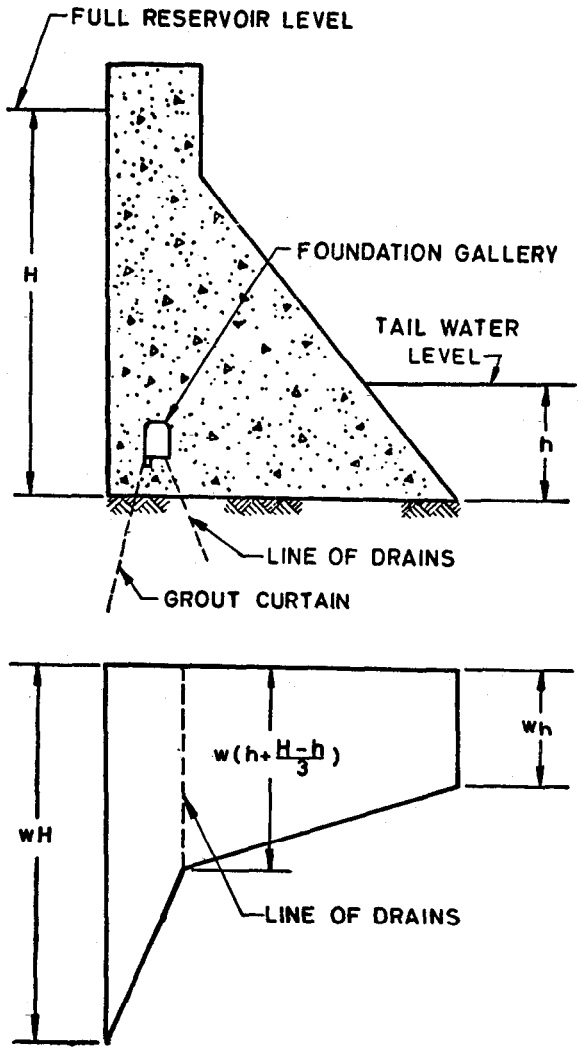


FIG. 1 UPLIFT DIAGRAM

- d) No reduction in uplift is assumed at the downstream toe of spillways on account of the reduced water surface elevation (relative to normal tailwater elevation) that may be expected immediately downstream of the structure.
- e) It is assumed that uplift pressures are not affected by earthquakes.

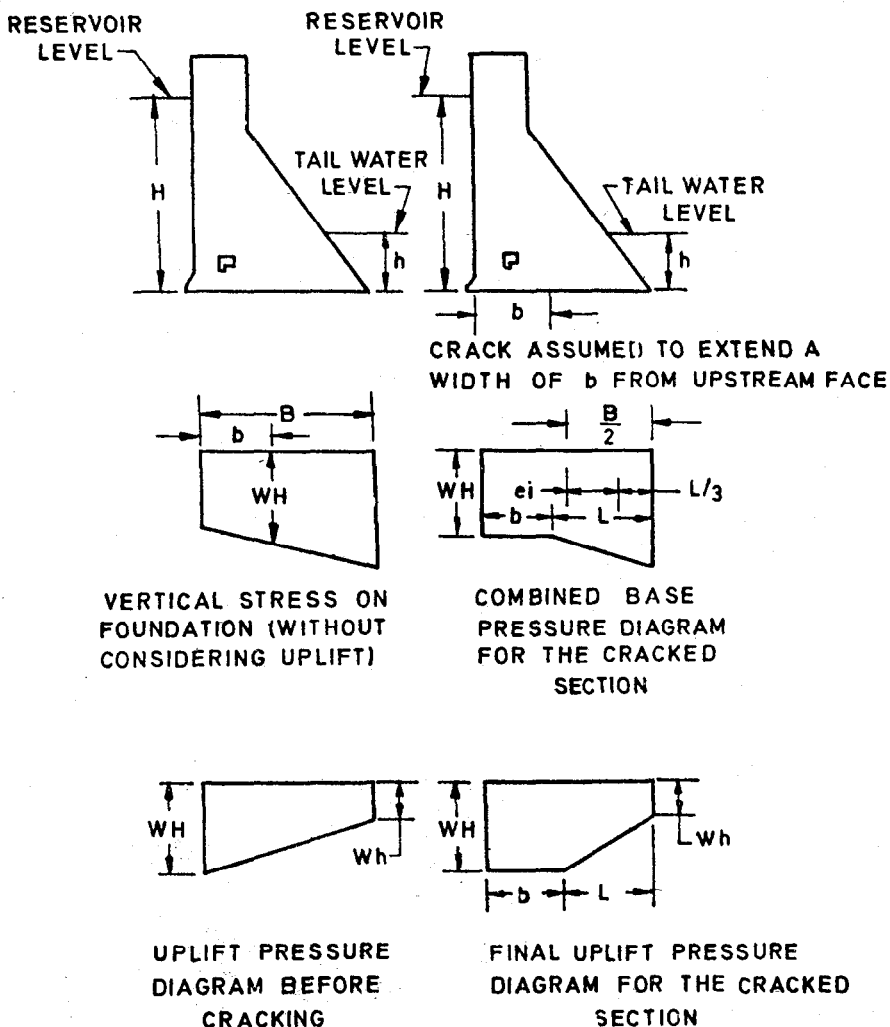


FIG. 2 PRESSURE DIAGRAM

5.4 Earthquake Forces — The criteria for seismic design shall be in accordance with IS : 1893-1984*.

5.5 Earth and Silt Pressures

5.5.1 General — Gravity dams are subjected to earth pressures on the downstream and upstream faces where the foundation trench is to be backfilled. Except in the abutment sections in specific cases and in the junctions of the dam with an earth or rockfill embankment, earth pressures have usually a minor effect on the stability of the structure and may be ignored.

5.5.1.1 The present procedure is to treat silt as a saturated cohesionless soil having full uplift and whose value of internal friction is not materially changed on account of submergence.

5.5.1.2 Experiments indicate that silt pressure and water pressure exist together in a submerged fill and that the silt pressure on the dam is reduced in the proportion that the weight of the fill is reduced by submergence.

5.5.2 Criteria for Design — The following criteria is recommended for calculating forces due to silt:

- a) Horizontal 'silt and water pressure' is assumed to be equivalent to that of a fluid with a mass of $1\,360\text{ kg/m}^3$, and
- b) Vertical 'silt and water pressure' is determined as if silt and water together have a density of $1\,925\text{ kg/m}^3$.

5.6 Ice Pressure

5.6.1 The problem of ice pressure in the design of dam is not encountered in India except, perhaps, in a few localities.

5.6.1.1 Ice expands and contracts with changes in temperature. In a reservoir completely frozen over, a drop in the air temperature or in the level of the reservoir water may cause the opening up of cracks which subsequently fill with water and freeze solid. When the next rise in temperature occurs, the ice expands and, if restrained, it exerts pressure on the dam. In some cases the ice exerts pressure on the dam when the water level rises. For ice sheets of wide extent this pressure is moderate but in smaller ice sheets the pressure may be of the same order of magnitude as in the case of extreme temperature variation.

5.6.1.2 Ice is plastic and flows under sustained pressure. The duration of rise in temperature is, therefore, as important as the magnitude of the rise in temperature in the determination of the pressure exerted by

*Criteria for earthquake resistant design of structures (*fourth revision*).

ice on the dam. Wind drag also contributes to the pressure exerted by ice to some extent. Wind drag is dependent on the size and shape of the exposed area, the roughness of the surface area and the direction of wind.

5.6.1.3 Existing design information on ice pressure is inadequate and somewhat approximate. Good analytical procedures exist for computing ice pressures, but the accuracy of results is dependent upon certain physical data which have not been adequately determined. These data should come from field and laboratory.

5.6.1.4 Till specific reliable procedures become available for the assessment of ice pressure it may be provided for at the rate of 250 kPa applied to the face of dam over the anticipated area of contact of ice with the face of dam.

5.7 Wind Pressure — Wind pressure does exist but is seldom a significant factor in the design of a dam. Wind loads may, therefore, be ignored.

5.8 Wave Pressure

5.8.1 In addition to the static water loads the upper portions of dams are subject to the impact of waves. Wave pressure against massive dams of appreciable height is usually of little consequence. The force and dimensions of waves depend mainly on the extent and configuration of the water surface, the velocity of wind and the depth of reservoir water. The height of wave is generally more important in the determination of the free board requirements of dams to prevent overtopping by wave splash. An empirical method based upon research studies on specific cases has been recommended by T. Saville for computation of wave height. It takes into account the effect of the shape of reservoir and also wind velocity over water surface rather than on land by applying necessary correction. It gives the value of different wave heights and the percentage of waves exceeding these heights so that design wave height for required exceedance can be selected.

5.8.2 Criteria for Design — The following formulae are made use of in determining wave height, wave pressure and wave force:

- a) For determining the height of waves the method recommended by T. Saville should be adopted. Details of the procedure are given in Appendix A.
- b) The maximum unit pressure p_w in kPa occurs at $0.125 h_w$ above the still water level and is given by the equation:

$$p_w = 24 h_w$$

where

h = height of wave in m.

- c) The wave pressure diagrams can be approximately represented by the triangle 1-2-3 in Fig. 3. The total wave force P_w (in kN) is given by the area of the triangle, or

$$P_w = 20 h_w^2$$

- d) The centre of application is at a height of $0.375 h_w$ above the still water level.

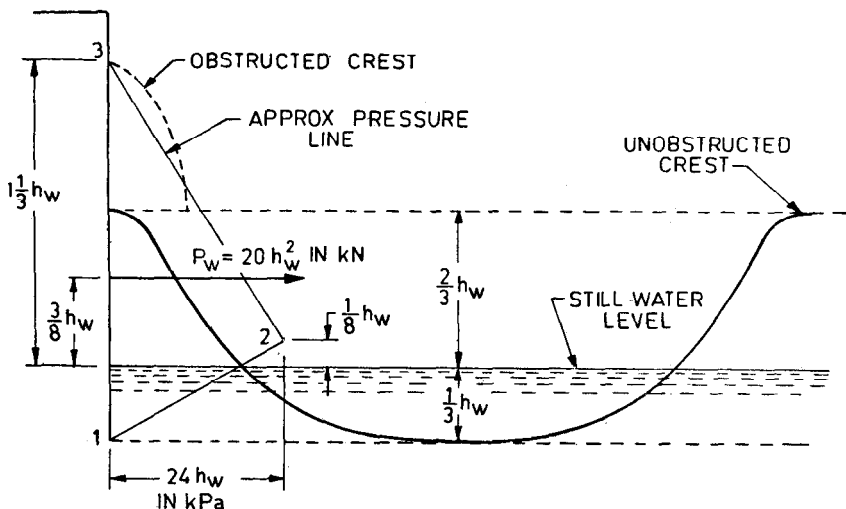


FIG. 3 WAVE DATA

5.8.3 Free Board — Free board is the vertical distance between the top of the dam and still water level. The free board shall be wind set-up plus $1\frac{1}{3}$ times wave height above normal pool elevation or above maximum reservoir level corresponding to the design flood, whichever gives higher crest elevation for the dam. The free board shall not, however, be less than 1.0 m above mean water level (MWL) corresponding to the design flood. If design flood is not same as probable maximum flood (PMF) than the top of dam shall not be lower than MWL corresponding to PMF. Notwithstanding the above requirement, 1.0 m high solid parapet shall be provided on the upstream side above the top of the dam in all cases.

5.8.3.1 The wave height and wind set up should be calculated according to the method recommended by T. Saville. The details of the procedure to be followed for computation of free board are given in Appendix A.

5.8.3.2 Wind velocity of 120 km/h over water in case of normal pool condition and of 80 km/h over water in case of maximum reservoir condition should generally be assumed for calculation of wave height if meteorological data is not available. When maximum wind velocity is known, the same shall be used for full reservoir level (FRL) condition and $2/3$ times that for MWL condition.

5.9 Thermal Loads — Measures for temperature control of concrete in solid gravity dams are adopted during construction. Yet it is noticed that stresses in the dam are affected due to temperature variation in the dam on the basis of data recorded from the thermometer embedded in the body of the dam. The cyclic variation of air temperature and the solar radiation on the downstream side and the reservoir temperature on the upstream side also affect the stresses in the dam. Even the deflection of the dam is maximum in the morning and it goes on reducing to a minimum value in the evening. The magnitude of deflection is also affected depending on whether the spillway is running or not. It is generally less when spillway is working than when it is not working. While considering the thermal load, temperature gradients are assumed depending on location, orientation, surrounding topography, etc.

5.10 Reaction of Foundations — The determination by statics of the reaction of the foundations of dams is covered in various texts. In general, the resultant of all horizontal and vertical forces including uplift should be balanced by an equal and opposite reaction at the foundation consisting of the total vertical reaction and the total horizontal shear and friction at the base and the resisting shear and friction of the passive wedge, if any. For the dam to be in static equilibrium the location of this force is such that the summation of moments is equal to zero. The distribution of the vertical reaction is assumed as trapezoidal for convenience only, with a knowledge that the elastic and plastic properties of both the foundation material and the concrete do affect the actual distribution.

5.10.1 The problem of determining the actual distribution is complicated by the horizontal reaction, internal stress relations and other theoretical considerations. Moreover, variation of foundation materials with depth, cracks, and fissures which affect the resistance of the foundation also make the problem more complex.

5.10.2 For overflow sections, the base width is generally determined by projecting the spillway slope to the foundation line and all concrete downstream from this line is disregarded. If a vertical longitudinal joint is not provided at this point, the mass of concrete downstream from the theoretical toe must be investigated for internal stresses.

5.10.3 Internal stresses and foundation pressures should be computed both with and without uplift to determine the worst condition.

5.11 Resistance Against Overturning — Before a gravity dam overturns bodily, other types of failures may occur, such as cracking of the upstream material due to tension, increase in uplift, crushing of toe material and sliding. A gravity dam is, therefore, considered safe against overturning if the criteria of no tension on the upstream face, the resistance against sliding as well as the quality and strength of concrete/masonry of the dam and its foundation is satisfied assuming the dam and foundation as a continuous body.

5.12 Sliding Resistance

5.12.1 General — Many of the loads on the dam are horizontal or have horizontal components which are resisted by frictional or shearing forces along horizontal or nearly horizontal planes in the body of the dam, on the foundation or on horizontal or nearly horizontal seams in the foundation. The stability of a dam against sliding is evaluated by comparing the minimum total available resistance along the critical path of sliding (that is, along that plane or combination of planes which mobilizes the least resistance to sliding) to the total magnitude of the forces tending to induce sliding.

5.12.1.1 Sliding resistance is a function of the cohesion inherent in the materials and at their contact and the angle of internal friction of the material at the surface of sliding. The junction plane between the dam and rock is rarely smooth. In fact, special efforts are made to avoid this condition. There may, however, be some lower plane in the foundation where sliding is resisted by friction alone especially if the rock is markedly stratified and horizontally bedded.

5.12.2 Criteria for Design — The factor of safety against sliding may be calculated as indicated in 5.12.2.1 on the basis of partial factor of safety in respect of friction ($F\phi$) and partial factor of safety in respect of cohesion (F_c) as given in Table 1.

TABLE 1 PARTIAL FACTORS OF SAFETY AGAINST SLIDING

SL. No.	LOADING CONDITION	$F\phi$	F_c		
			For Dams and the Contact Plane with Foundation	For Foundation	
				Thoroughly Investigated	Others
i)	A, B, C	1.5	3.6	4.0	4.5
ii)	D, E	1.2	2.4	2.7	3.0
iii)	F, G	1.0	1.2	1.35	1.5

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

$$F = \frac{\frac{(w - u) \tan \phi}{F\phi} + \frac{CA}{F_0}}{P}$$

where

- F = factor of safety against sliding,
- w = total mass of the dam,
- u = total uplift force,
- $\tan \phi$ = coefficient of internal friction of the material,
- C = cohesion of the material at the plane considered.
- A = area under consideration for cohesion,
- $F\phi$ = partial factor of safety in respect of friction,
- F_0 = partial factor of safety in respect of cohesion, and
- P = total horizontal force.

5.12.2.2 The value of cohesion and internal friction may be estimated for the purpose of preliminary designs on the basis of available data on similar or comparable materials. For final designs, however, the value of cohesion and internal friction shall be determined by actual laboratory and field tests (see IS : 7746-1975*).

5.13 Quality and Strength of Concrete/Masonry

5.13.1 General — The strength of concrete/masonry shall exceed the stresses anticipated in the structure by a safe margin. The maximum compressive stresses occur at the heel or toe and on planes normal to the faces of the dam. The strength of concrete and masonry varies with age, the kind of cement and other ingredients and their proportions in the work can be determined only by experiment.

5.13.2 Criteria for Design

5.13.2.1 General — Mix proportions are determined from the results of laboratory tests made with the materials that will be used in the structures. The proportions are selected to produce concrete/masonry of sufficient strength to meet the design requirements multiplied by an appropriate safety factor. In addition to meeting the requirements of strength described in 5.13.2.2 and 5.13.2.3, the concrete/masonry/mortar should be adequate in regard to placing characteristics, weathering resistance, impermeability and resistance to alkali-aggregate attack.

*Code of practice for *in-situ* shear test on rocks.

5.13.2.2 Compressive strength — The compressive strength of concrete and masonry shall conform to the following requirements:

- a) *Concrete* — Concrete strength is determined by compressing to failure 150 mm cubes. The strength of concrete should satisfy early load and construction requirements and at the age of one year it should be four times the maximum computed stress in the dam or 14 N/mm^2 , whichever is more. The allowable working stress in any part of the structure shall not also exceed 7 N/mm^2 .
- b) *Masonry* — The compressive strength of masonry is determined by compressing to failure 75-cm cubes of the masonry fabricated and cured at temperatures approximating to those expected in the structures (or 45×90 -cm cylinders cored out of the structures or blocks made for the purpose). This strength should satisfy early load and construction requirements and at one year it should be five times the maximum computed stress on the dam or 12.5 N/mm^2 whichever is more.

NOTE — For the purpose of quality control, correlation between the strength of mortar and that of masonry may be established in suitable smaller size specimen.

5.13.2.3 Tensile strength

- a) No tensile stress shall be permitted at the upstream face of the dam for load combination B (see 4.1). Nominal tensile stresses, however, may be permitted in other load combinations and their permissible values shall not exceed the values given in Table 2.

TABLE 2 VALUES OF PERMISSIBLE TENSILE STRESS IN CONCRETE AND MASONRY

LOAD COMBINATION	PERMISSIBLE TENSILE STRESS	
	Concrete	Masonry
C	$0.01 f_c$	$0.005 f_c$
E	$0.02 f_c$	$0.01 f_c$
F	$0.02 f_c$	$0.01 f_c$
G	$0.04 f_c$	$0.02 f_c$

where f_c is the cube compressive strength of concrete/mortar for masonry.

- b) Small values of tension on the downstream face may be permitted since it is very improbable that a fully constructed dam is kept empty and downstream cracks which are not extensive and for limited depths from the surface may not be detrimental to the safety of the structure.

6. CONFIGURATION OF DAM AND FOUNDATION

6.1 The shape of a dam and curvature in its layout are pertinent in regard to the stability and more favourable stress conditions. Wherever possible

dam and foundation designs should take advantage of the favourable conditions accruing from curved shapes, gradual transitions and fillets.

7. CONTRACTION JOINTS

7.1 Longitudinal and Transverse Contraction Joints—A contraction joint is formed vertical or inclined surface between masses of concrete/masonry placed at different times. They divide the dam into convenient sized monoliths to permit convenient and systematic construction and to prevent the formation, owing to volume changes that cannot be prevented, of haphazard ragged cracks.

7.1.1 Longitudinal Contraction Joints — One of the measures used to control cracks parallel to the length of the dam in the case of relatively high dams is to subdivide the monolith into several blocks by longitudinal contraction joints and subsequently grout these joints to ensure monolithic action. The spacing of the joints is largely dictated by convenience of construction and the foundation conditions. A spacing of 20-30 m is generally adopted. There is also now a school of thought which believes that the longitudinal joints need to be at very close spacing (about 15 m) if they are to achieve their purpose.

7.1.1.1 However, it is recognized that the practice of dividing a monolith into two or more blocks by introducing joints parallel to the axis is basically unsound unless a high degree of perfection is accomplished in ensuring monolithicity by provision of suitable shear keys and successfully grouting at the appropriate time. It is now being increasingly accepted that better alternative is to achieve necessary temperature control by precooling of the concrete supplemented where necessary, by post-cooling and avoid longitudinal contraction joints altogether, even in case of high dams. No longitudinal joints are considered necessary in dams built of rubble masonry with the construction methods in vogue in India, as of now.

7.1.2 Transverse Contraction Joints — The spacing of transverse contraction joints shall be such as to suit the methods of construction materials of the dam, the foundation conditions and the convenience of the location of control gates outlet, etc. A spacing of 15 to 25 m may be adopted for concrete dams; larger spacing may be adopted for masonry dams. The general requirement is that each joint extends entirely through the structure.

7.1.3 Grouting Transverse Contraction Joints — The characteristics of a dam and its profile determine the magnitude of the load transferred horizontally through the joints to the abutments. If the stream bed is wide and flat, the vertical cantilever blocks from the centre of the dam towards end canyon walls are approximately of the same length. Under

load each will deflect downstream very nearly the same amount and the load transferred horizontally across the joints (provided it is capable of transferring the load) to the abutments will, therefore, be negligible except near the abutment. In a narrow canyon with steep sloping walls, each cantilever block from the centre of the dam towards the canyon wall will be shorter than the preceding one. In this case the load will cause each succeeding block to deflect less than the preceding one and more than the succeeding one. If the transverse joints are keyed and grouted, or keyed and ungrouted the intervening cantilever block will be affected by adjacent ones. This interaction between blocks causes torsional moments, or twist in the blocks, which materially affect the way in which the loads are distributed to the foundation and abutments. If the joints are keyed and grouted, part of the load will be transferred horizontally to the abutments by both bending and shear in the horizontal beam elements. If the joints are keyed but ungrouted, the load will be transferred horizontally to the abutments by shear across the keys. If the joints are neither keyed nor grouted, the entire load on the dam is transferred to the foundation independently by each block.

APPENDIX A

(*Clauses 5.8.2 and 5.8.3.1*)

PROCEDURE FOR COMPUTING FREE BOARD FOR GRAVITY DAM

A-1. The following procedure shall be followed.

A-1.1 Select design FRL as reference level for which, free board is to be computed and enter at Step 1 in enclosed proforma (146.3 m).

A-1.2 Use reservoir water spread or submergence contour plan showing dam centreline and FRL contour and work out maximum fetch in km as shown in Fig. 4. Enter at Step 2 in *pro forma* (11.0 km).

A-1.3 Select a line AB, with A on dam axis and B on FRL contour in Fig. 4 so as to cover the maximum reservoir water spread area within 42° angle on either side of line AB. If necessary, do two or three trials, so that the computed effective fetch is maximum. Assume maximum wind velocity to act along BA. Draw 7 rays at 6° interval on either side of AB and compute effective fetch as shown in Fig. 4. Enter in Step 3 of *pro forma* (5.0 km).

A-1.4 Find out maximum wind velocity on land at dam site from 50 years return period chart*. Enter in Step 4 of *pro forma* (140 km/h).

*Being included in the revision of IS : 875-1964 Code of practice for structural safety of buildings: Loading standards (*revised*).

A-1.5 Convert wind velocity on land (UL) to wind velocity on water (UW) using table given below:

Effective Fetch in km	1	2	4	6	8	10	12
$\frac{UW}{UL}$	1.10	1.16	1.24	1.27	1.30	1.31	1.31

Enter wind velocity on water (UW) in Step 5 of *pro forma* (175 km/h).

A-1.6 Use Fig. 5 and compute significant wave height (H_s) for the computed wind velocity on water of 175 km/h and effective fetch of 5 km.

Enter in Step 6 of *pro forma* (2.8 m).

A-1.7 Using table below, select design wave height (H) as that specific height which is exceeded by only 4 percent waves ($1.27 H_s$).

Enter in Step 7 of *pro forma* (3.6 m).

Percentage of waves exceeding specific wave height H .	0.4	2	4	8	13
Ratio = $\frac{\text{Sp. height}}{\text{Significant height}} = \left(\frac{H}{H_s}\right)$	1.67	1.40	1.27	1.12	1.00

NOTE — Higher percentages of wave exceeding H may be considered for small dams, not likely to cause damage to life and property in case of breach. Lower percentage may be considered in case of large dams.

A-1.8 Compute wind set up from formula:

$$\text{Wind set up} = \frac{V^2 F}{62\,000 D}$$

where V = wind velocity over water in km/h;

F = maximum fetch in km; and

D = average depth of reservoir in m along maximum fetch, computed as shown in Fig. 4 (18.3 m).

Enter in Step 8 of *pro forma* (0.30 m).

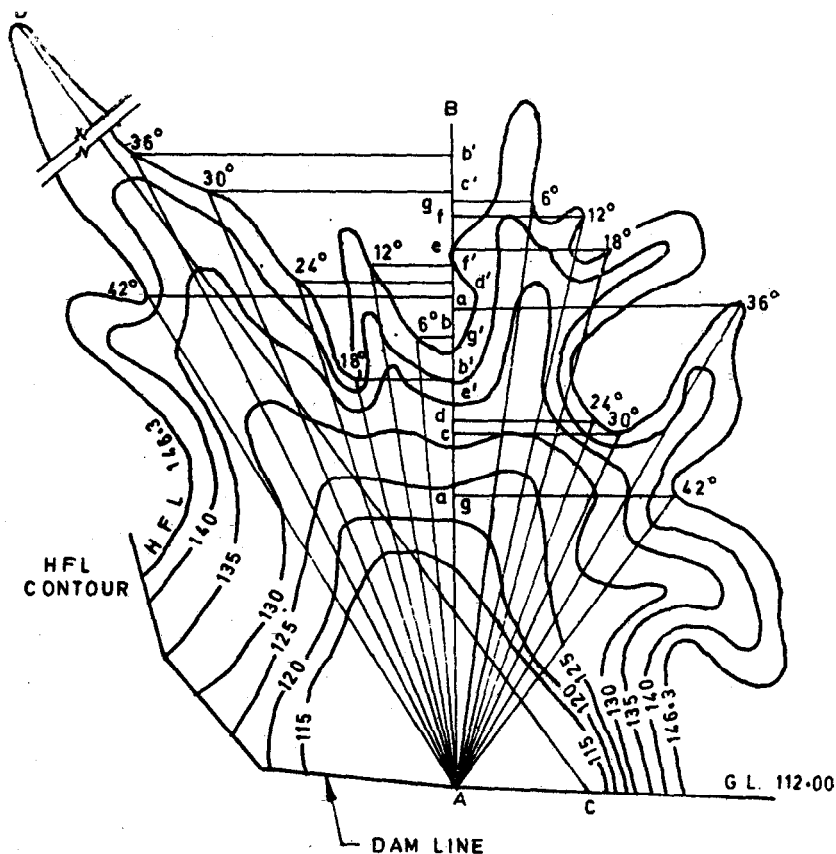
A-1.9 Work out computed free board as sum of $1\frac{1}{3}$ times the value in Step 7 and value in Step 8 and enter in Step 9 of *pro forma* (5.1 m).

A-1.10 *Pro forma* for computation of free board for gravity dams is given below:

- Step 1 Full reservoir level (FRL) = 146.3 m
- Step 2 Reservoir maximum fetch = 11.0 km
- Step 3 Reservoir effective fetch = 5.0 km
(from Fig. 4)
- Step 4 Maximum wind velocity over land = 140 km/h
- Step 5 Wind velocity over water (refer table under **A-1.5**) = Velocity over land \times Wind ratio
= 140×1.27
= 175 km/h
- Step 6 Significant wave height (H_s) = 2.8 m
(from Fig. 5)
- Step 7 Design wave height (H) = $1.27 \times H_s$
(refer table under **A-1.7**)
= $(1.27 \times 2.8) = 3.6$ m
- Step 8 Wind set up = $\frac{V^2 F}{62\,000 D} = \frac{(175)^2 \times 11}{62\,000 \times 18.3} = 0.3$ m
- Step 9 Computed free board = $1\frac{1}{3}$ (Step 7) + (Step 8)
= $4.8 + 0.3 = 5.1$ m
= $3.60 + 0.30 = 3.9$ m
- Step 10 Similar computation should be done with reference to MWL for $2/3$ wind velocity
- Step 11 Minimum free board to be provided above MWL = 1.0 m
- Step 12 Top of dam should be highest of
(FRL + Step 9), (MWL + Step 10) and
(MWL + Step 11)

NOTE 1 — Out of the free board computed 1 m may be provided as solid parapet.

NOTE 2 — If the design flood is different from probable maximum flood (PMF), top of dam shall not be lower than MWL corresponding to PMF.



COMPUTATION OF FETCH OF WATER ALONG FETCH LINE C-D

SL No.	FRL IN m	CL IN m	DEPTH IN m
1	146.3	146.3	0
2	146.3	140.0	6.3
3	146.3	135.0	11.3
4	146.3	130.0	16.3
5	146.3	125.0	21.3
6	146.3	120.0	26.3
7	146.3	115.0	31.3
8	At the dam	112.0	34.3

Total 147.1

$$\text{Average Depth} = \frac{147.1}{8}$$

$$= 18.3 \text{ m}$$

CALCULATION OF EFFECTIVE FETCH

α°	$\text{Cos } \alpha$	x_1 IN km	$x_1 \text{ Cos } \alpha$
42	0.743	Aa = 3.00	2.23
36	0.809	Ab = 4.85	3.92
30	0.866	Ac = 3.65	3.16
24	0.914	Ad = 4.80	4.39
18	0.951	Ae = 5.50	5.23
12	0.978	Af = 5.90	5.77
6	0.995	Ag = 6.00	5.97
0	0.00	AB = 4.50	4.50
6	0.995	Ag' = 4.70	4.68
12	0.978	Af' = 5.3	5.18
18	0.951	Ae' = 4.2	3.99
24	0.914	Ad' = 5.25	4.80
30	0.866	Ac' = 6.20	5.37
36	0.809	Ab' = 6.55	5.30
42	0.743	Aa' = 5.10	3.79

$$\Sigma \text{Cos } \alpha = 13.512 \quad \Sigma x_1 \text{ Cos } \alpha = 68.28$$

i) Effective Fetch

$$= \frac{\Sigma x_1 \text{ Cos } \alpha}{\Sigma \text{Cos } \alpha}$$

$$= \frac{68.28}{13.512}$$

$$= 5.00 \text{ km}$$

ii) Maximum Fetch (C-D)

$$= 11.00 \text{ km}$$

FIG. 4 COMPUTATION OF EFFECTIVE FETCH (F) AND AVERAGE DEPTH (D)

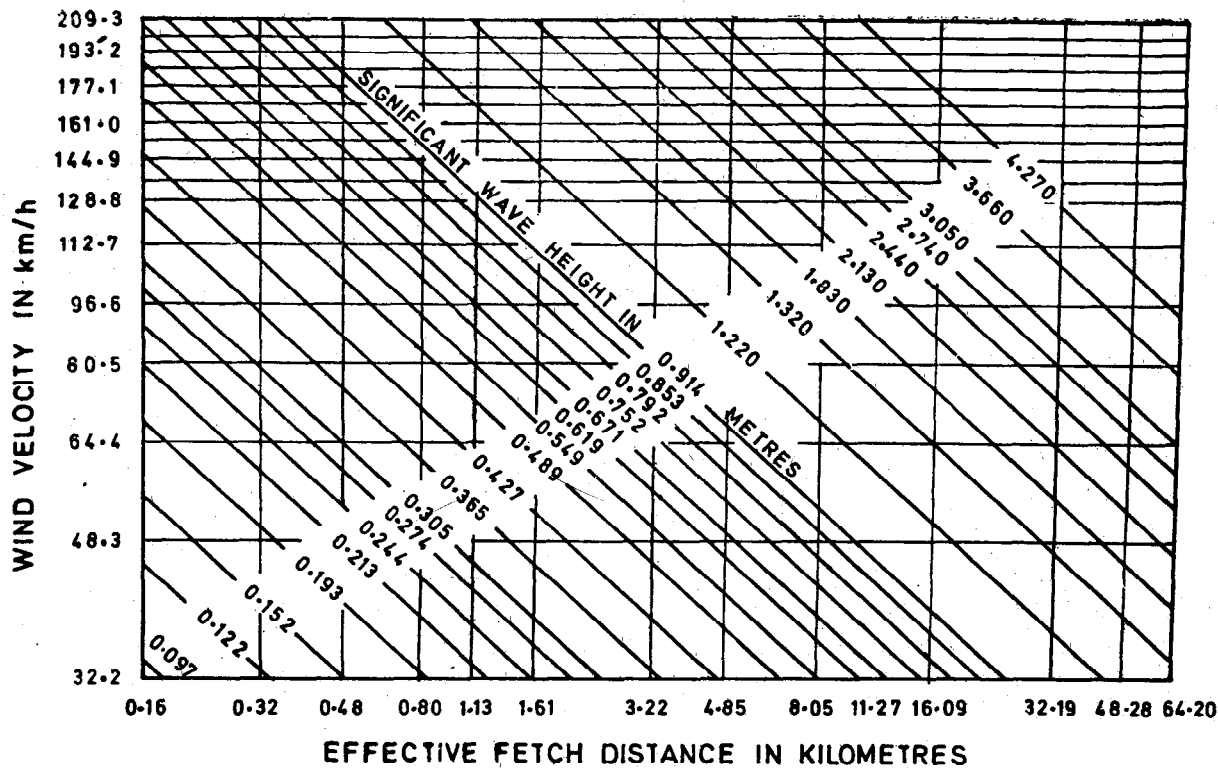


FIG. 5 CORRELATION OF SIGNIFICANT WAVE HEIGHTS (H_s) WITH RELATED FACTORS

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