Design of a Concrete Gravity Dam for Flood Control in Brahmani River Basin

Dr. Ratnakar Swain[1], Payal Maharathi[2]

[1, 2]Parala Maharaja Government Engineering College, Civil Engineering Department Berhampur, Odisha, India
Corresponding Email ID- *niki.maharathi1[at]gmail.com*

Abstract: ***Flooding occurs when an extreme volume of water is carried by rivers, creeks and many other geographical features into areas where the water cannot be drained adequately. Floods causes extremely large numbers of fatalities in every country, but due to India's extremely high population density and often under enforced development standards, a large amount of damages and many deaths which could be otherwise avoided, are allowed to happen. India witnesses flood due to excessive rain which then results in overflow of rivers, lakes which adds to cause large amounts of damage to people's lives and property. Orissa is a land of rivers. The Mahanadi, the Kathjodi, the Brahmani, the Baitarani, the Subarnarekha and the Budhabalanga flow on the land of Orissa. These rivers are feed by rain water. So, during the rainy season, floods come in them. These rivers are in spate generally in the month of September. Floods in Odisha cause a great loss to her people. Crops worth corers of rupees are destroyed by flood. The sufferings of people are beyond imagination and beyond description. There are many methods to controlling the floods. The techniques are construction of dams, diversion canals, flood plains and ground water replenishment, river defenses, self-closing flood barrier, temporary perimeter barriers. We are choosing method of construction of dam. Many dams are associated reservoirs are designed completely or partially to aid in flood protection and control. Many large dams have flood-control reservations in which the level of a reservoir must be kept below a certain elevation before the onset off the rainy seasons to allow a certain amount of space in which flood water scan fill. Dam is built to control water through placement of a blockage of earth, rock and concrete across a stream or river. Dams are usually constructed to store water in a reservoir, which is then used for a variety of applications such as Irrigation and municipal water supplies .Here we considered 30yrs of rainfall for data .From which maximum discharge is calculated. Stage calculation is done from rating curve. By analyzing the 30 years data the height of the dam is calculated. Calculation of geometry of dam is done. All the required forces acting on dam are calculated. Then the stability analysis of the dam is done. At last the profile calculation of the spillway is done.***

Keywords: Dam, Flood Control, Spillway

1. Introduction

Flood is overflow of water, a deluge of water that submerges land which as normally dries. A flood occurs when water overflows or inundates land that normally dries. This can happen in a multitude of ways. Flood causes extremely large numbers of fatalities in every country, but due to India's extremely high population density and often under enforced development standards, a large amount of damages and many deaths which could be otherwise avoided, are allowed to happen. Flood control methods are used to reduce or prevent the detrimental effects of flood waters. Flood relief methods are used to reduce the effects of flood waters or high water levels. Dams have been constructed for millennia, influencing the lives of humans and the ecosystems they inhabit. Remnants of one such man-made structure dating back 5,000 years are still standing in northeast Africa (UNESCO-WWAP, 2003). Around 2950-2750 B.C., the first dam known to exist was built by the ancient Egyptians, measuring 11.3 meters (37 feet) tall, with a crest length of 106 m (348 ft.) and foundation length of 80.7 m (265 ft.) (Yang, et al, 1999). The dam was composed of 100,000 tons of rubble, gravel, and stone, with an outer shell of limestone. The immense weight was enough to contain water in a reservoir estimated to have been 570,000 cubic meters in capacity. Many concrete gravity dams have been in service for over 50 years, and over this period important advances in the methodologies for evaluation of natural phenomena hazards have caused the design-basis events for these dams to be revised upwards. Older existing dams may fail to meet revised safety criteria and structural rehabilitation to meet such criteria may be costly and difficult. The identified causes of failure, based on a study of over 1600 dams are: foundation problems (40%), inadequate spillway (23%), poor construction (12%), uneven settlement (10%), and high pore pressure (5%), acts of war (3%), embankment slips (2%), defective materials (2%), incorrect operation (2%), and earthquakes (1%). Earthquake hazards had caused the collapse and damage to continual functioning of essential services such as communication and transportation facilities, buildings, dams, pipelines, water and waste water systems, electric and nuclear power plants with severe economic losses. However, the structural response of a material to different loads determines how it will be economically utilized in the design process. This necessitates the seismic analysis of concrete gravity dams. Finite element has been widely used in seismic analysis of concrete gravity dams utilizing the most natural method based on the Lagrangian–Eulerian formulation.

During the recent years, the seismic behaviour of concrete gravity dams was in the centre of consideration of dam engineers. Numerous researches have been conducted in order to determine how the dams behave against the seismic loads. Many achievements were obtained in the process of analysis and design of concrete dams including dam-reservoir-foundation interaction during an earthquake. The earthquake response of concrete gravity dam-reservoir-foundation system has been addressed to study the effect of foundation flexibility and reservoir water body on the seismic response of concrete gravity dams. Safety evaluation of dynamic response of dams is important for most of researchers. When such system is subjected to an earthquake, hydrodynamic pressures are developed on upstream face of the dam due to the vibration of the dam and reservoir water. Consequently, the prediction of the dynamic response of dam to earthquake loadings is a complicated problem and depends on several factors, such as interaction of the dam with rock foundation and reservoir, the computer modelling and material properties used in the analysis.

Gravity dams are very important structures. The collapse of a gravity dam due to earthquake ground motion may cause an extensive damage to property and life losses. Therefore, the proper design of gravity dams is an important issue in dam engineering. An integral part of this procedure is to accurately estimate the dam earthquake response. The prediction of the actual response of a gravity dam subjected to earthquake is a very complicated problem. It depends on several factors such as dam-foundation interaction, dam-water interaction, material model used and the analytical model employed. In fluid-structure interaction one of the main problems is the identification of the hydrodynamic pressure applied on the dam body during earthquake excitation. The analysis of dam-reservoir system is complicated more than that of the dam itself due to the difference between the characteristics of fluid and dam's concrete on one side and the interaction between reservoir and dam on the other side.

Several studies related with different aspect of settlement rehabilitation and economic importance of dam have been carried out from various journals of India and abroad. A.Adediji, L.T.Ajibade (2008) focused on the change detection of major dam in Nigeria with the help of RS and GIS techniques. In this paper author gives ideas about the construction of dam, site of new dams, catchment area, and rehabilitation with the help of RS and GIS techniques. Acharya S., Acharya S. (1994) describe the settlement of hydraulic structure and its spatio – temporal analysis of North Bengal state. The Authors (Acharya and Acharya, 1994) stated that there is a variation in the structural forms of settlements also variation in patio – temporal distribution of settlements. Agba, A.M.Ogaboh, Akpanudoedehe, J.J. Ushie, E. M., (2010), In his paper “Socioeconomic and cultural impact of resettlement on Bakassi people of Cross river state, Nigeria”. Stated that the socio-economic and cultural impact of resettlement on Bakassi people of cross river state, Nigeria was examined. He concluded that the impact of resettlement on the accommodation, settlement layout and social networking were quit reviling. Badgujar A.A. (2004), In his dissertation, “Geographical aspect of tribal settlements in the western satpura region”, he study the tribal settlement with the view of its location, types, spacing, size and house types. Tribal settlements are different from other settlements. BasseyEbeno, Aminant Fashona, Mike Idah, Gebemiga Adeyemi, Shehu S. Fada, (2007) , In this paper author co-relate the leprosy and socio–economic impact of rehabilitation. In the process of rehabilitation it is observe the socio-economic setup is badly disturbed the related study clearly conclude the Nigeria. Bhaise S.D., Chaudhari S.R. (2003), In the of study rehabilitated villages due to Hatnur reservoir clearly shows the economically changes because rehabilitation of every settlement effect on social, cultural and economic factors. About this factor economy greatly change.

Bhaise Sanjay Devidas, Dr. S. R. Chaudhari, Lalit P. Sandanshiv (2011),In the present paper authors try to focused on social morphology of rehabilitated villages of Jalgaon district due to Hatnur irrigation project. It was accepted that in the rehabilitated villages the old social morphology pattern would not be observed. But it has been observed during the work that expectation was not true. In almost all the surveyed villages the same morphological pattern which was prevailed in their old settlements is still retained. Dixit K.K. (1991), these paper authors explain the structural characterises sticks of village shapes. Every village has it won shape there are many types of shapes. Every shape has won history. There is impact of physical as well as cultural factors on shapes of settlement. Dusane Y.B. (2009), In his dissertation “The study of interlinking of river and phad system in dhule district (M.S.)” he systematically studies the interlinking of river and phad system in Dhule district. He correlates the settlements in phad area and another area. There is a clear-cut difference between both settlements. Duvali S., Hamerlynck O (2003) ,“Mitigation of negative ecological and socio-economic impacts of the Diama dam on the Senegal River Delta wetland (Mauritania),using a model based decision support system”. This project provides an example of implementation of the recommendation of the world commission on dams through ecologically and sociologically benefits operation of a dam-based infrastructure within the basin, agreed through stakeholder participation. Florentina-Cristina IANCU (2007),The paper synthesizes the multiple consequences induced by the restructuring process of mining industry within personae depression. The analyzed area is anof the mining sector for the economy of the area.

Gatade D.G. (1994), The present paper focuses on spatial distribution on rural settlement in Karad tehsil Maharashtra state. The spatial distribution of settlement .this uneven distribution is the result of topography socio –economic condition culture and superstitious. Gatkul B.I., Bhore J.B. (2011), In this paper Conservation and management of Soil, Resources for sustainable Agriculture Development in Indapurtaluka, Dist. Pune: An Environment Approach. This paper gives idea on soil and resources management for the sustainable development of agriculture in particular tehsil. Hope E. Ogbeide, EtiosaUyigue, Solomon Oshodin, (2003), in his paper he clearly concluded that dams have very far reaching impacts on the environment and people of Nigeria. The two dams under studyare commissioned for water supply purposes. They are small dams measuring them against the standard definition of large dams by ICOLD (International Commission on Large Dams), which is a high of 15m and beyond from the ground level.

2. Methodology

There are two main types of dams, embankment dams and concrete dams. Embankment dams can either be rock fill or earth fill dams. The method of construction is similar in both cases, with just the main type of material differentiating. Concrete dams are superior in constructing massive overflow discharge sections, and are therefore often used in areas where floods are common. A lot less material is used compared to an embankment dam but concrete is usually more costly. It is also easier to connect a hydropower station to a concrete dam. Three different height spans exist when concrete dams are considered and they are defined as: low dams (up to 30 meters), medium height dams (30-90 meters) and high dams (90 meters and above). This is a measurement of the difference in elevation between the lowest constructed part of the dam foundation and the walkway at the dam crest.

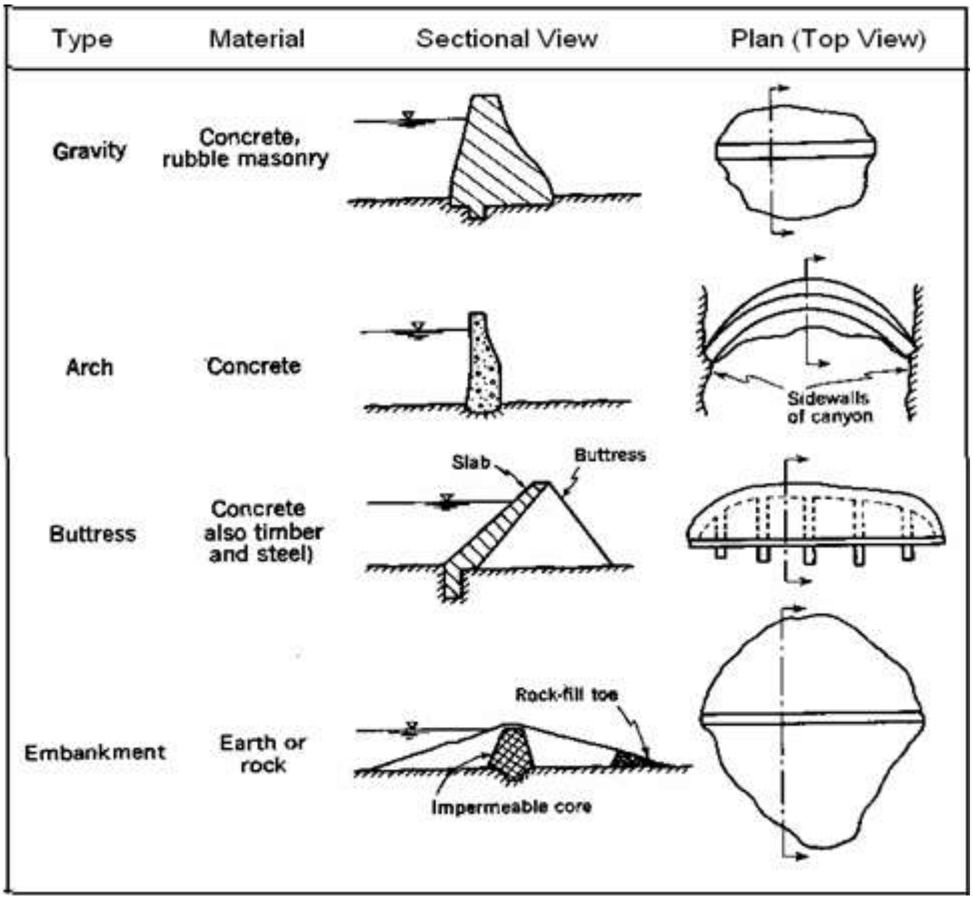

Figure 1: Different types of concrete dams

A gravity dam is a solid structure, made of concrete or masonry, constructed across a river to create a reservoir on its upstream. The section of the gravity dam is approximately triangular in shape, with its apex at its top and maximum width at bottom. The section is so proportioned that it resists the various forces acting on it by its own weight. Most of the gravity dams are solid, so that no bending stress is introduced at any point and hence, they are sometimes known as solid gravity dams to distinguish them from hollow gravity dams in those hollow spaces are kept to reduce the weight. Early gravity dams were built of masonry, but nowadays with improved methods of construction, quality control and curing, concrete is most commonly used for the construction of modern gravity dams. A gravity dam (Figure.1) is generally straight in plan and, therefore, it is also called straight gravity dam. The upstream face is vertical or slightly inclined. The slope of the downstream face usually varies between "0.7: 1‖ to ―0.8: 1‖. There are different types of concrete dams based on the principal for the transfer of the hydrostatic pressure. The theory behind gravity dams is that their own weight should be sufficient to withstand the hydrostatic pressures affecting them. This means that gravity dams are usually massive and therefore require a lot of construction material. With the amount of concrete required, this dam type may be somewhat expensive but on the other hand, it is very versatile. Another advantage is that it can possess substantial overflow discharge capacity.

Buttress dams, shown in figure 8, are similar to gravity dams with the distinction that they also use the gravity of the reservoir water instead of only the gravity of the dam itself. Because of this, the dam body does not need to be as massive and use buttresses instead of a solid downstream part of the dam. Being less solid on the downstream side, buttress dams have the advantage of being a lot less affected by the water uplift force.

Arch dams, shown in figure 9, are curved around a vertical cord to resist the hydrostatic pressure by arching thus transferring the pressure into the canyon walls. For this transfer to be possible and cost effective the width to height ratio should not exceed 5:1, although in some cases arch dams has been built with a ratio as high as 10:1. Another criteria which is important for arch dams is the shape of the canyon, if it is symmetrical an arch dam is often very suitable. If the canyon is a little less symmetrical, an arch dam with influences of a gravity dam may be constructed. If the canyon is extremely asymmetric, another dam type may be preferred.

3. Basic Definition

The basic definitions of dam are given below:

(a) Axis of the dam

The axis of the gravity dam is the line of the upstream edge of the top (or crown) of the dam. If the upstream face of the dam is vertical, the axis of the dam coincides with the plan of the upstream edge. In plan, the axis of the dam indicates the horizontal trace of the upstream edge of the top of the dam. The axis of the dam in plan is also called the base line of the dam. The axis of the dam in plan is usually straight. However, in some special cases, it may be slightly curved upstream, or it may consist of a combination of slightly curved right portions at ends and a central abutment straight portion to take the best advantages of the topography of the site.

(b) Length of the dam

The length of the dam is the distance from one abutment to the other, measured along the axis of the dam at the level of the top of the dam. It is the usual practice to mark the distance from the left abutment to the right abutment. The left abutment is one which is to the left of the person moving along with the current of water.

(c) Structural height of the dam

The structural height of the dam is the difference in elevations of the top of the dam and the lowest point in the excavated foundation. It, however, does not include the depth of special geological features of foundations such as narrow fault zones below the foundation. In general, the height of the dam means its structural height.

(d) Maximum base width of the dam

The maximum base width of the dam is the maximum horizontal distance between the heel and the toe of the maximum section of the dam in the middle of the valley.

(e) Toe and Heel

The toe of the dam is the downstream edge of the base, and the heel is the upstream edge of the base. When a person moves along with water current, his toe comes first and heel comes later.

(f) Hydraulic height of the dam

The hydraulic height of the dam is equal to the difference in elevations of the highest controlled water surface on the upstream of the dam (i.e. FRL) and the lowest point in the river bed.

3.1 Static properties of concrete dam

(a) Strength

A gravity dam should be constructed of concrete that will meet the design criteria for strength, durability, permeability, and other required properties. Because of the sustained loading generally associated with them, the concrete properties used for the analyses of static loading conditions should include the effect of creep. Properties of concrete vary with age, the type of cement, aggregates, and other ingredients as well as their proportions in the mix. Since different concretes gain strength at different rates, measurements must be made of specimens of sufficient age to permit evaluation of ultimate strengths. Although the concrete mix is usually designed for only compressive strength, appropriate tests should be made to determine the tensile and shear strength values.

(b) Elastic Properties

Poisson's ratio, the sustained modulus of elasticity of the concrete, and the latter's ratio to the deformation modulus of the foundation have significant effects on stress distribution in the structure. Values of the modulus of elasticity, although not directly proportional to concrete strength, do follow the same trend, with the higher strength concretes having a higher value for modulus of elasticity. As with the strength properties, the elastic modulus is influenced by mix proportions, cement, aggregate, admixtures, and age. The deformation that occurs immediately with application of load depends on the instantaneous elastic modulus. The increase in deformation which occurs over a period of time with a constant load is the result of creep or plastic flow in the concrete. The effects of creep are generally accounted for by determining a sustained modulus of elasticity of the concrete for use in the analyses of static loadings. Instantaneous moduli of elasticity and Poisson's ratios should be determined for the different ages of concrete when the cylinders are initially loaded. The sustained modulus of elasticity should be determined from these cylinders after specific periods of time under constant sustained load. These periods of loading are often 365 and 730 days. The cylinders to be tested should be of the same size and cured in the same manner as those used for the compressive strength tests. The values of instantaneous modulus of elasticity, Poisson's ratio, and sustained modulus of elasticity used in the analyses should be the average of all test cylinder values.

(c) Thermal Properties

The effects of temperature change in gravity dams are not as important in the design as those in arch dams. However, during construction, the temperature change of the concrete in the dam should be controlled to avoid undesirable cracking. Thermal properties necessary for the evaluation of temperature changes are the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity. The coefficient of thermal expansion is the length change per unit length for 1 degree temperature change. Thermal conductivity is the rate of heat conduction through a unit thickness over a unit area of the material subjected to a unit temperature difference between faces. The specific heat is defined as the amount of heat required to raise the temperature of a unit mass of the material 1 degree. Diffusivity of concrete is an index of the facility with which concrete will undergo temperature change. The diffusivity is calculated from the values of specific heat, thermal conductivity, and density. Appropriate laboratory tests should be made of the design mix to determine all concrete properties.

3.2 Dynamic Properties of concrete dam

(a) Strength
No data are yet available to indicate what the strength characteristics are under dynamic loading.

(b) Elastic Properties
Until dynamic modulus information is available, the instantaneous modulus of elasticity determined for concrete specimens at the time of initial loading should be the value used for analyses of dynamic effects.

(c) Average Properties
Necessary values of concrete properties may be estimated from published data for preliminary studies until laboratory test data are available. Until long-term tests are made to determine the effects of creep, the sustained modulus of elasticity should be taken as 60 to 70 percent of the laboratory value for the instantaneous modulus of elasticity. Criteria-Ifno tests or published data are available, the following average values for concrete properties may be used for preliminary designs until test data are available for better results (USBR):

Compressive strength-3,000 to 5,000 Ibs/in2 (20.7 to 34.5 MPa)
Tensile strength-5 to 6 percent of the compressive strength
Shear strength: Cohesion-about 10 percent of the compressive strength
Coefficient of internal friction- l.0
Poisson’s ratio- 0.2
Instantaneous modulus of elasticity- 5.0 x 106 lbs/in2 (34.5 GPa)
Sustained modulus of elasticity- 3.0 x 106 lbs/in2 (20.7 GPa)
Coefficient of thermal expansion- 5.0 x 10-6/“F (9.0 x l0-6PC)
Unit weight- 150 Ibs/ft3 (2402.8 kg/m3)

3.3 Foundation Properties

(a) Deformation Modulus
Foundation deformations caused by loads from the dam affect the stress distributions within the dam. Conversely, response of the dam to external loading and foundation deformability determines the stresses within the foundation. Proper evaluation of the dam and foundation interaction requires as accurate a determination of foundation deformation characteristics as possible. Although the dam is considered to be homogeneous, elastic, and isotropic, its foundation is generally heterogeneous, inelastic, and anisotropic. These characteristics of the foundation have significant effects on the deformation moduli of the foundation. The analysis of a gravity dam should include the effective deformation modulus and its variation over the entire contact area of the dam with the foundation. The deformation modulus is defined as the ratio of applied stress to elastic strain plus inelastic strain and should be determined for each foundation material. The effective deformation modulus is a composite of deformation moduli for all materials within a particular segment of the foundation. Good compositional description of the zone tested for deformation modulus and adequate geologic deformation modulus and adequate geologic logging of the drill cores permit extrapolation of results to untested zones of similar material.

(b) Shear Strength
Resistance to shear within the foundation and between the dam and its foundation depends upon the cohesion and internal friction inherent in the foundation materials and in the bond between concrete and rock at the contact with the dam. These properties are determined from laboratory and in situ tests. The results of laboratory triaxial and direct shear tests, as well as in situ shear tests, are generally reported in the form of the Coulomb equation:

$$R = C.A + N.\ tan\ \varphi \text{ or}$$

Shear resistance= unit cohesion times area+ effective normal force times coefficient of internal friction which defines a linear relationship between shear resistance and normal load. The value of shear resistance obtained as above should be limited to use for the range of normal loads used for the tests. Although this assumption of linearity is usually realistic for the shear resistance of intact rock over the range of normal loads tested, a curve of shear resistance versus normal load should be used for materials other than intact rock. The shear resistance versus normal load relationship is determined from a number of tests at different normal loads. The individual tests give the relationship of shear resistance to displacement for a particular normal load. The results of these individual tests are used to obtain a shear resistance versus normal load curve. The displacement used to determine the shear resistance is the maximum displacement that can be allowed on the possible sliding plane without causing unacceptable stress concentrations within the dam. Since specimens tested in the laboratory or in situ are small compared to the foundation, the scale effect should be carefully considered in determining the values of shear resistance to be used.

When a foundation is non-homogeneous, 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. If the intact rock shears, the shear resistance of the entire plane is equal to the combined sliding frictional resistance for all materials along the plane.

(c) Pore Pressure and Permeability

Analysis of a dam foundation requires knowledge of the hydrostatic pressure distribution in the foundation. Permeability is controlled by the characteristics of the rock type, the jointing systems, the shears and fissures, fault zones, and, at some dam sites, by solution cavities in the rock. The exit gradient for shear zone materials that surface near the downstream toe of the dam should also be determined to check against the possibility of piping. Laboratory values for permeability of sample specimens are applicable only to the portion or portions of the foundation that they represent. Permeability of the fore mentioned geologic features can best be determined by in situ testing. The permeability obtained is used in the determination of pore pressures for analyses of stresses, stability, and piping. Such a determination may be made by several methods including two- and three dimensional physical models, two- and three-dimensional finite element models, and electric analogy. If foundation grouting and drainage or other treatment is to be used, their effects on the pore pressures should be included.

3.4 Hydrological Consideration

Estimation of Maximum Discharge

The maximum discharge is calculated from annual peak discharge data of last 30 years peak flood data considered.

$$Q_m = \frac{1}{N}\sum_{i=1}^{N} Q_{mi}$$

Where Q_m= Maximum discharge of last 30 years peak flood data
N= no. of years
Q_{mi}=Annual peak discharge data

Estimation flood stage

The flood stage is estimated from the rating curve generated from the last 30 years peak flood data

Loading for Gravity Dam

The significant loadings on a concrete gravity dam include the self-weight or dead load of the dam, the water pressure from the reservoir, and the uplift pressure from the foundation. There are other loadings, which either occur intermittently, like earthquake forces, or are smaller in magnitude, like the pressure exerted by the waves generated in the reservoir that his upstream of the dam face. These loadings are explained in the following section.

Dead load

The dead load comprises of the weight of the concrete structure of the dam body in addition to pier gates and bridges, if any over the piers. The density of concrete may be considered as 2400 kg/m³. Since the cross section of a dam usually would not be simple, the analysis may be carried out by dividing the section into several triangles and rectangles and the dead load (self-weight) of each of these sections (considering unit width or the block width) computed separately and then added up. For finding out the moment of the dead load (required for calculating stresses), the moments due to the separate sub–parts may be calculated individually and then summed up.

Water pressure on dam

The pressure due to water in the reservoir and that of the tail water acting on vertical planes on the upstream and downstream side of the dam respectively may be calculated by the law of hydrostatics. Thus, the pressure at any depth **h** is given by γhkN/m² acting normal to the surface.

When the dam has a sloping upstream face, thewater pressure can be resolved into its horizontal and vertical components, the vertical component being given by the weight of the water prism on the upstream face and acts vertically downward through the center of gravity of the water area supported on the dam face.

In spillway section, when the gates are closed, the water pressure can be worked out in the same manner as for non–overflow sections except for vertical load of water on the dam itself. During overflow, the top portion of the pressure triangle gets truncated and a trapezium of pressure acts.

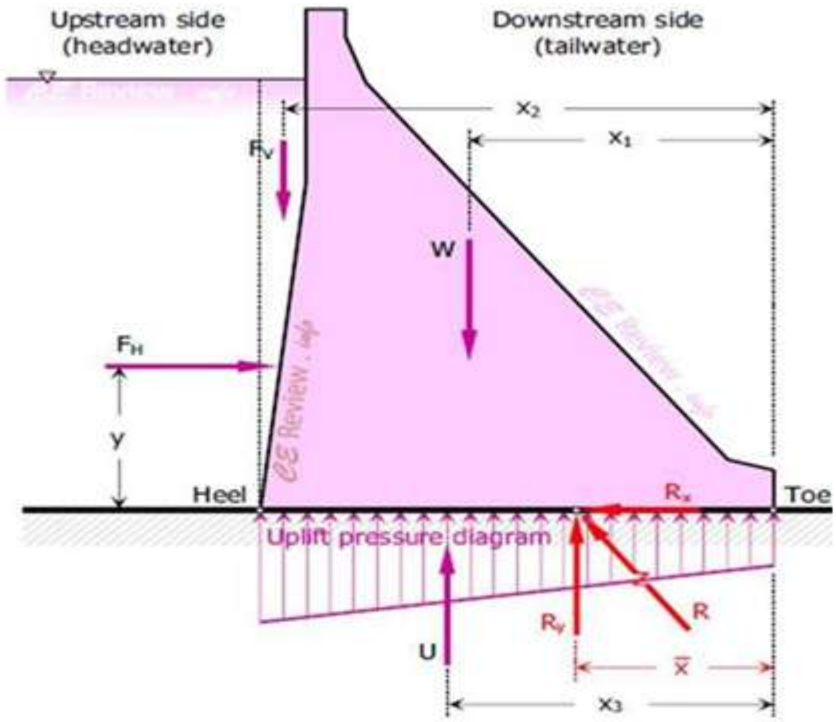

Figure 2: Cross section of a typical gravity dam

The pressure due to tail water is obtained in a similar manner as for the upstream reservoir water. In case of low overflow dams, the dynamic effect of the velocity of approach may be significant and deserve consideration.

Water Pressure = $\frac{1}{2}\gamma_w H^2$ ………………. (1)

Where, γ_W = unit weight of water
H = Height of dam

Uplift pressures

Uplift forces occur as internal pressure in pores, cracks and seams within the body of the dam, at the contact between the dam and its foundation and within the foundation. The recent trends for evaluating uplift forces are based on the phenomenon of seepage through permeable material. Water under pressure enters the pores and fissures of the foundation material and joints in the dam. The uplift is supposed to act on the whole width plane that is being considered, either at the base or at any position within the dam. The uplift pressure on the upstream end of the considered horizontal plane is taken as γh_u where h_u is the depth of water above the plane. On the downstream the value is γh_d where h_d is again the depth of water above the plane.

Uplift pressures = $\gamma_w H_d + \frac{1}{3}(\gamma_w H_u - \gamma_w H_d)$……..(2)

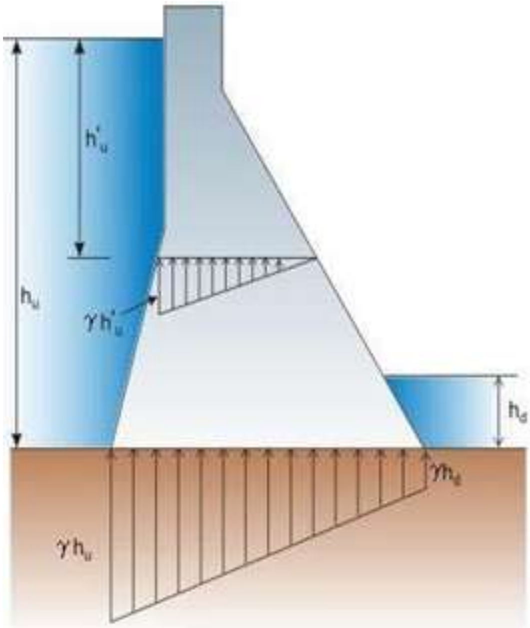

Figure 3: Uplift pressure at base and at any general plane in the dam body.

Fig3 illustrates the uplift pressure on a concrete gravity dam's non overflow section through two planes – one at the base and the other at the horizontal plane which is above the tail water level. In Fig2, the drainage holes either in the body of the dam, or within the foundation has not been considered. If the effects of the drainage holes are considered, then the uplift pressure diagram gets modified as shown in Fig 4. If there is crack at any plane of the dam, or at the base then the uplift pressure diagram gets further modified as shown in Fig 4.

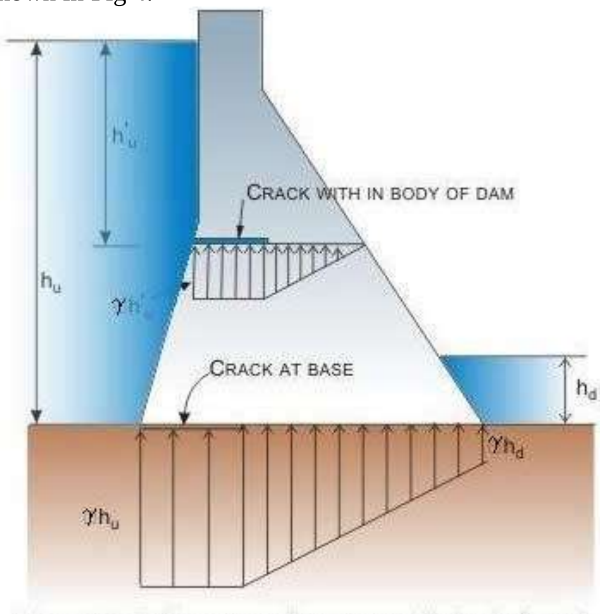

Figure 4: Uplift pressure diagrams considering horizontal cracks at any general plane/at the base.

As such, the uplift pressure is assumed to act throughout the base area. Further it is also assumed that they remain unaffected by earthquakes.

Silt pressure

The weight and the pressure of the submerged silt are to be considered in addition to weight and pressure of water. The weight of the silt acts vertically on the slope and pressure horizontally, in a similar fashion to the corresponding forces due to water. It is recommended that the submerged density of silt for calculating horizontal pressure may be taken as 1360 kg/m³. Equivalently, for calculating vertical force, the same may be taken as 1925 kg/m³.

Earthquake (seismic) forces

Earthquake or seismic activity is associated with complex oscillating patterns of acceleration and ground motions, which generate transient dynamic loads due to inertia of the dam and the retained body of water. Horizontal and vertical accelerations are not equal, the former being of greater intensity.

The earthquake acceleration is usually designated as a fraction of the acceleration due to gravity and is expressed as α· g, where α is the Seismic Coefficient. The seismic coefficient depends on various factors, like the intensity of the earthquake, the part or zone of the country in which the structure is located, the elasticity of the material of the dam and its foundation, etc. For the purpose of determining the value of the seismic coefficient which has to be adopted in the design of a dam, India has been divided into five seismic zones, depending upon the severity of the earthquakes which may occur in different places. A map showing these zones are given in the Bureau of Indian Standards code IS: 1893-2002 (Part-1) "Criteria for earthquake resistant design of Structures (fourth revision)".

Earthquake force = PW ………………(3)

Where, P = 0.1 for horizontal, 0.05 for vertical
W = Weight of dam

Zanger's formula $P_e = 0.726 p_e H$,……………(4)
Where $p_e = C_m.\ k_h.\ \gamma_w.H$

$C_m = 0.735 \text{ x } \frac{\theta}{90^0}$

Modes of Failure
A gravity dam may fail in the following ways:
1) By overturning about toe.
2) By crushing
3) By development of tension, causing ultimate failure by crushing.
4) By shear failure called sliding.
The failure may occur at the foundation plane or at any other plane at higher level.

Overturning
If the resultant of all the forces acting on a dam at any of its section passes outside the toe, dam shall rotate and overturn about the toe. Practically, such a condition shall not arise, as the dam will fail much earlier by compression.The ratio of the righting moments about toe to overturning moment about toe is called the FOS against overturning .Its value, generally varies between 2 to 3.

So, Factor of Safety Against Overturning= $\frac{+\Sigma M}{-\Sigma M}$

Crushing
A dam may fail by the failure of its materials, i.e. the compressive stress produce may exceed the allowable stresses, and the dam material may get crushed. The vertical direct stress distribution at the base is given by the equation:

$P_{max/\ min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$ …………..(4)

Tension
Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot with stand sustained tensile stresses. If subjected to such stresses, these materials may finally crack. However for achieving economy are designs of very high gravity dams, certain amount of tension may be permitted under severest loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time and would neither last long nor occur frequently. The maximum permissible tensile stress for high concrete gravity dams, under worst leadings, may be taken as500 kN/m^2.

Effect produced by tension cracks. In a dam, when such a tension crack develops, say at heel, crack width loses contact with the bottom foundations, and thus, become ineffective.

Hence, the effective width of the dam base will be reduced. This will increase P_{max} at the toe.

In order to ensure that no tension is developed anywhere, we must ensure that P_{min} is at the most equal to zero.
$P_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$…………..(5)

Putting $P_{min} = 0$, we have $\frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = 0$

Or $1 - \frac{6e}{B} = 0$ or $e = \frac{B}{6}$

Hence maximum value of eccentricity that can be permitted on either side of the centre is equal to B/6, which leads to famous statement: the resultant must lie within the middle third.

Sliding:
Sliding will occur when the net horizontal force above any place in the dam or at the base of the dam exceeds the frictional resistance developed at that level.

The friction developed between two surfaces is equal to is equal to $\mu \sum V$, where $\sum V$ is the algebraic sum of all the vertical forces whether upward or downward, and μ is the coefficient of friction between the two surfaces. In order that no sliding takes place, the external horizontal force $\sum H$ must be less than the shear resistance $\mu \sum V$.

Or, $\sum H < \mu . \sum V$

Or $\frac{\mu \sum V}{\sum H} > 1$

$\frac{\mu \sum V}{\sum H}$ represents nothing but the FOS against sliding, which must be greater than unity.

So, FOS against sliding = $\frac{\mu \sum V}{\sum H}$

In low dams the safety against sliding should be cracked only for friction, but in high dams, for economical precise designs, the shear strength of the joint, which is additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation of FOS against sliding which is measured by shear friction factor (S.F.F) becomes,

S.F.F = $\frac{\mu \sum V + B.q}{\sum H}$ ………………….(6)

Where B = width of the dam at the joint
q= Average shear strength

Stability Criteria
The loads listed in section will create different types of stresses in the dam body. Although every dam project is unique, problems with these stresses will often occur in the same areas. Figure shows these general critical areas. To create a clear overview of the figure none of the applied loads are displayed. To evaluate shear stress in thedifferent, carefully chosen, areas in and below the dam foundation, information about the friction coefficient both in concrete, rock and between the two is needed. The cohesion in concrete and rock is also needed. With the vertical stress, the shear stress, the friction coefficient and the cohesion a safety factor, K can be calculated. This safety factor is calculated according to Mohr-coulomb failure criterion. Unstable sliding surfaces can occur in numerous places in the dam and the foundation. Therefore it is important to single out the areas where the greatest risk of damage exists. For example such surfaces could be cracks in the ground, where a change of rock material occurs and in various places in the dam, the two most obvious of such being in the foundation plane just in the contact surface with the underlying rock and the horizontal plane where the slope of the downstream side of the dam body starts to flatten out shown in figure.
Stability criteria for concrete gravity dams accounts for four types of controls to be considered as follows:
(i) Sliding stability; to make sure the calculations are accurate, the element standards, described later in this chapter, will be considered when creating elements inside and around the critical areas, especially close to the dam foundation and in the batter.
(ii) Tension stress often occurs, in the region around the dam heel. When analysing the tension stress we use a simplification for FEM modelling that states that the number of elements with tension in the bottom layer of the dam cannot exceed seven per cent of the total amount of elements in that layer. The reason that we can use this simplification is that the elements in the foundation have roughly the same size, which leads to that the percentage of tension elements is considered the same as the percentage.
(iii) Compression stresses are handled by looking at the whole model and then determine where the greatest risk for compressive failure appears. Material characteristics have to be evaluated and compared to the computed compressive stresses.

Displacement control is based on the entire model. Since for example the displacement in the top of the dam depends on the displacement in the bottom part of the dam there is not really one area to focus on to receive good displacement results.

Eccentrity (e) = $\frac{B}{2} - \overline{X}$ ……………(7)

Where, $\overline{X}$ (Distance from toe) = $\frac{\Sigma M}{\Sigma V}$
ΣV= Summation of Vertical force

Average vertical stress = $\frac{\Sigma V}{B}$

Principal stress at toe, $\sigma_1 = P_V \sec^2\alpha - p' \tan^2\alpha$ ……..(8)

Principal stress at heel, $\sigma_2 = P_{V(heel)} \sec^2\theta - (P + p_e)\tan^2\theta$….(9)

Shear stress at toe $\tau_{e\,(toe)} = (P_{V(toe)} - P')\tan\alpha$……………(10)

Maximum/minimum pressure = $\frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$ ……. (11)
Shear stress at heel $\tau_{e(heel)} = -[P_{V(heel)} - (P + P_e)]\tan\theta$ …(12)

3.5.1 Factors of Safety

All loads to be used in design should be chosen to represent, as nearly as can be determined, the actual loads that will occur on the structure during operation, in accordance with the criteria under ―Load Combinations.‖ Methods of determining load-resisting capacity of the dam should be the most accurate available. All uncertainties regarding loads or load-carrying capacity should be resolved as far as practicable by field or laboratory tests and by thorough exploration and inspection of the foundation. Thus, the factor of safety should be as accurate an evaluation as possible of the capacity of the structure to resist applied loads. All safety factors listed are minimum values. Like other important structures, dams should be regularly and frequently inspected. Adequate observations and measurements should be made of the structural behaviour of the dam and its foundation to assure that the structure is functioning as designed.

Although somewhat lower safety factors may be permitted for limited local areas within the foundation, overall safety factors for the dam and its foundation after beneficiation should meet requirements for the loading combination being analyzed. For other loading combinations where safety factors are not specified, the designer is responsible for selection of safety factors consistent with those for loading combination categories previously discussed. Somewhat higher safety factors should be used for foundation studies because of the greater amount of uncertainty involved in assessing foundation load-resisting capacity. Safety factors for gravity dams are based on the use of the gravity method of analysis and those for foundation sliding stability are based on an assumption of uniform stress distribution on the plane being analyzed.

FOS against sliding= $\frac{\mu \Sigma V}{\Sigma H}$ ………. (13)

FOS against overturning= $\frac{+\Sigma M}{-\Sigma M}$ …………(14)

Shear friction factor = $\frac{\mu \Sigma V + Bq}{\Sigma H}$………..(15)

DESIGN OF CONCRETE GRAVITY DAM

The section of gravity dam should be chosen in such a way that it is the most economical section and satisfies all the conditions and requirements of stability. Hence, after the section of the dam has been arrived at, the stability analysis for the dam must be carried out.

Calculation of Dam height

First of all the height of the dam to be constructed, should be checked so as to ensure whether it is a low gravity dam or a high gravity dam. If the height of the dam is less than that given by,

$$H_1 = \frac{f}{\gamma_w (S_c + 1)} \quad \dots\dots\dots\dots (16)$$

Where, γ_w = Unit weight of water
S_c = Specific gravity in concrete
f = maximum compressive stress

Design of Dam

The economical section of low gravity dam of height H_1 after deciding the top width a and freeboard, can be drawn. The base width B_1 of the triangle can be choosen as given by equation as;

$B_1 = \frac{H_1}{\sqrt{S-C}}$ ………… (17)

The upstream face can be kept vertical up to a height H_1 to be determined by trial, and whose approximate first value may be chosen by equation as;

$H_1 = 2a\sqrt{S-C}$ ………(18)

Where, C=0 (in no uplift case
And C=1 (in uplift case)

Below this height H_1, the upstream face as well as the downstream face are slopped in such a manner that no tension is developed anywhere of the dam, and the resultant force remain as close to the outer third and inner third point as possible, for reservoir full and empty cases, respectively. This is accomplished by hit and trial method, and when it is so accomplished, all the stability requirements will be satisfied.

Spillway Design

A gravity dam should also have an appropriate spillway for releasing excess flood water of the river during monsoon months. This section looks slightly different from the other non- overflowing sections. A typical section of a spillway is shown in Figure 5.

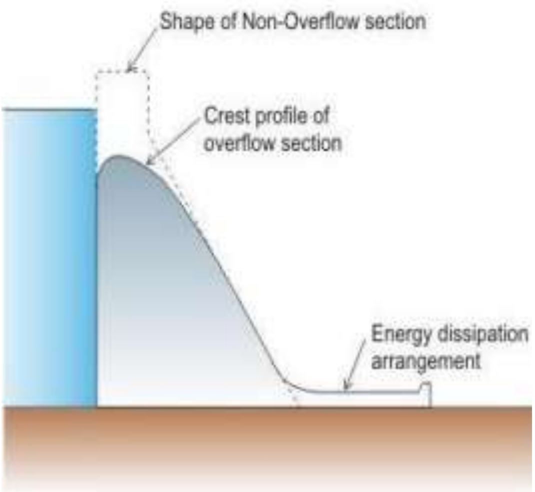

Figure 5: Typical overflow section of a gravity dam

The flood water glides over the crest and downstream face of the spillway and meets an energy dissipating structure that helps to kill the energy of the flowing water, which otherwise would have caused erosion of the river bed on the downstream. The type of energy dissipating structure shown in Figure 5 is called the stilling basin which dissipates energy of the fast flowing water by formation of hydraulic jump at basin location. This and other types of spillway and energy dissipaters are discussed in a subsequent section. Figure 5 shows the functioning of this type of spillways.

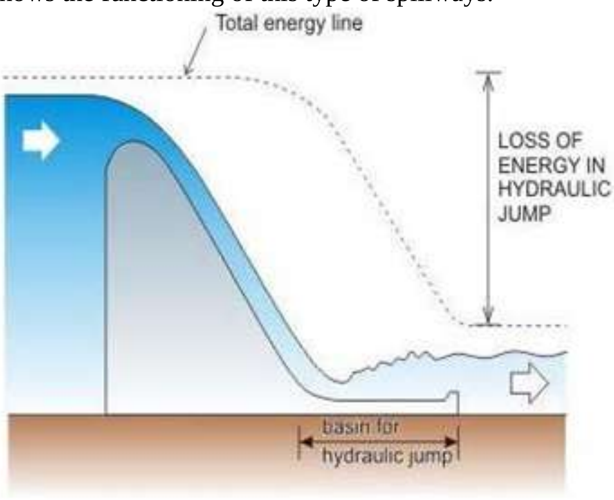

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Figure 6: Water flowing over a spillway

Usually, a spillway is provided with a gate, and a typical spillway section may have a radial gate as shown in Figure. The axis or turning of the gate is held to anchorages that are fixed to piers.

Also shown in the figure is a guide wall or training wall that is necessary to prevent the flow crossing over from one bay (controlled by a gate) to the adjacent one. Since the width of a gate is physically limited to about 20m (limited by the availability of hoisting motors), there has to be a number of bays with corresponding equal number of gates separated by guide walls in a practical dam spillway.

The upstream face of the overflowing and non-overflowing sections of a gravity dam are generally kept in one plane, which is termed as the dam axis or sometimes referred to as the dam base line. Where is normal acceleration of the dam body on the upstream face and n is normal vector on the interface of the dam-reservoir outwards the dam body and is the mass density of the reservoir water.

Spillway discharge (Q) =

$C.\ L_e.\ H_e^{3/2}$ ………………(19)

Where, L_e(Effective length of spillway) = $L - 2[NK_p + N_a]H_e$

Velocity head = $\frac{V_a^2}{2g}$

Downstream profile, $(x^{1.85}) = 2H_d^{0.35}y$ ………..(20)

Upstream profile, $y = \frac{0.7249\,(X + .27H_d)^{1.85}}{H_d^{\ 0.85}} + 0.126\ H_d - .4315\ H_d^{\ 0.375}\ (x + .27H_d)^{0.625}$ …..(21)

Data Collection

Information about Study Area

The Baitarani-Brahmani Section: The areas lying between the Salandi in the north and the .Birupa in the south are inundated by the Baitarani, the Brahmani and their tributaries. This covers the lower sections of the Brahmani-Baitarani interfluve and its adjoining areas. This is the most active portion of the compound delta. This is the largest section. Prone to frequent severe floods both from the river and the sea which includes the areas of Ali and Kanika. Besides this there are isolated patches in the upper deltaic section of the Brahmani-Baitarani interfluves and the Brahmani- Mahanadi interfluve7 around Jajpur and Dharmasala.

The Mahanadi-Brahmani Section: The lower section of the Mahanadi-Brahmani interfluve to the east of Pattamundai and the ill drained areas between the Nuna and the Mahanadi River are inundated by the frequent floods.

Data collected from CWC website

Table 1: Flood stage for the Measure Rivers of the Orissa coastal plain
(Source: Annual flood reports of the Irrigation Department Bhubaneswar)

Sl. No.	River	Location of the gausing station	Flood stage of the river in mts.
1.	Subarnarekha	Rajghat	4.60
2.	Budhabalanga	Fuladi	7.32
3.	Baitarani	Akhuapada	19.23
4.	Brahmani	Jenapur	20.12
5.	Mahanadi	Naraj	26.52

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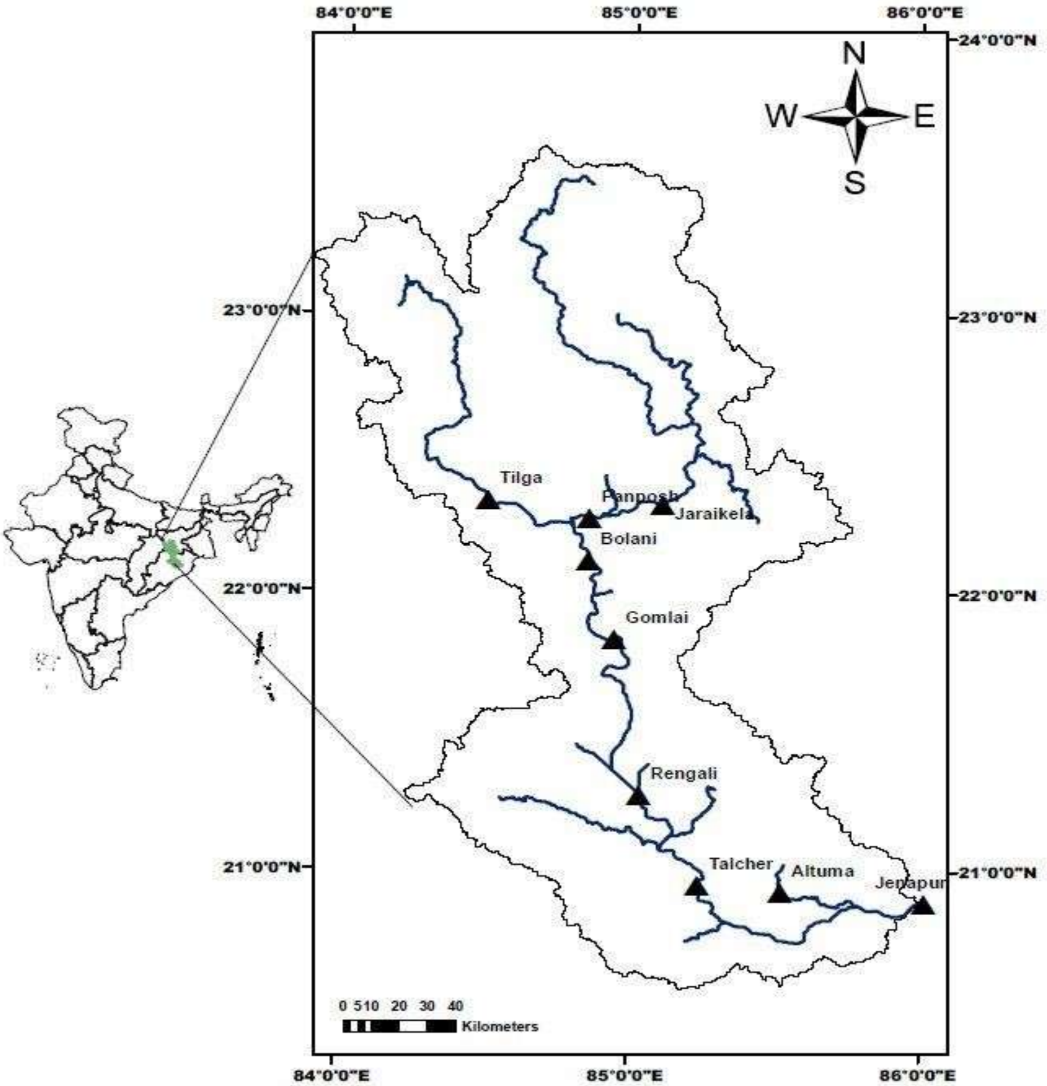

Figure 7: Index Map of the Brahmani River basin with CWC stage discharge gauging station

Crossection of Jenapur Gauging Station

Table 2: X Section at CGL at Jenapur

Station Code	Date	Serial No	Reduced Distance	Elevation CGL
EB000G6				
	30-01-2012			
		0	-	25.31
		0	10.00	24.95
		0	20.00	23.50
		0	40.00	23.28
		0	60.00	23.92
		0	80.00	22.61
		0	81.10	22.17
		0	100.00	18.00
		0	140.00	17.90
		0	150.00	18.10
		0	160.00	18.10
		0	180.00	18.11
		0	200.00	18.73
		0	220.00	18.75
		0	240.00	18.64

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Station Code	Date	Serial No	Reduced Distance	Elevation CGL
EB000G6	30-01- 2012	0	260.00	18.76
		0	280.00	18.78
		0	300.00	18.86
		0	320.00	18.98
		0	340.00	19.14
		0	360.00	18.97
		0	380.00	18.80
		0	400.00	18.35
		0	420.00	18.14
		0	440.00	18.75
		0	450.00	18.68
		0	460.00	18.39
		0	480.00	18.36
		0	500.00	18.46
		0	520.00	18.29

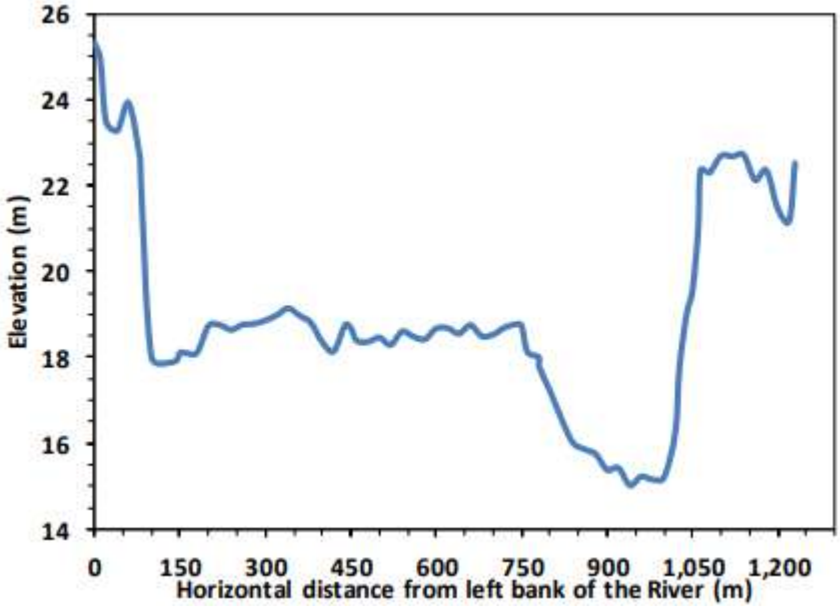

Figure 8: Cross-section of Jenapur Gauging Station

Stage Discharge

Maximum daily discharge and Stage corresponding to daily discharge are calculated using the developed Rating Curve.

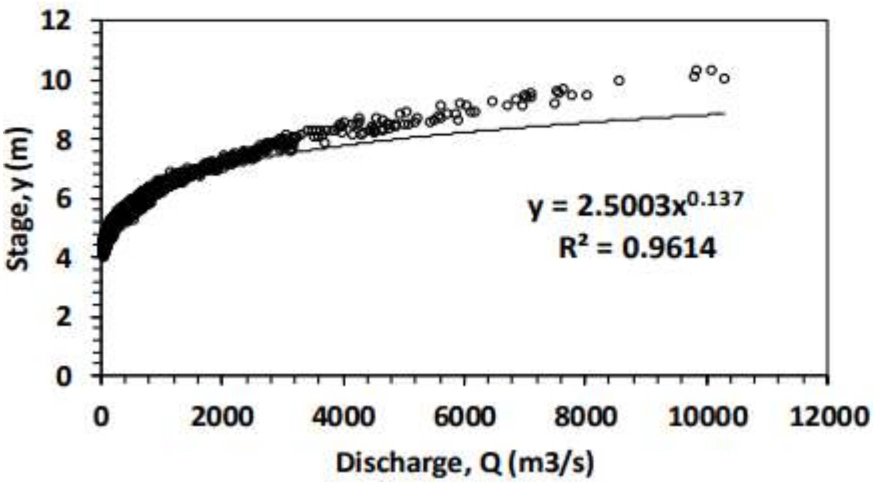

Figure 9: The Rating Curve

4. Result and Discussions

Hydrological Calculation

Estimation of maximum discharge

$$Q_m = \frac{1}{N}\sum_{i=1}^{N} Q_{mi} = 10372\text{m}^2/\text{s}$$

Where Q_m= Maximum discharge of last 30 years peak flood data
N=no of years
Q_{mi}=Annual peak discharge data

Estimation flood stage

Flood stage is estimated from the developed Rating curve, generated from the last 30 years flood data.

$$y = 2.5003x^{0.137}$$

$$R^2 = 0.9614$$

Design and Stability Check of Dam

Depth water at upstream side z_1 = 10.73m
Assume, tail Maximum discharge Q =10372.06 m^3/s
Water, $z_2 = 0$
Z= height of dam
=z_1 + free board + silt deposited + extra height
=10.73 + 3 + 3 + 3 = 19.73 ≈20m

Assume,
Grade of concrete M_{30}

Maximum compressible stress 3000 kN/m^2

Specific gravity in concrete, s_C = 24

Limiting height of dam, $H_L = \frac{f}{\gamma_w\,(S_C + 1)}$

$$= \frac{3000}{9.81\,(2.4+1)}$$

= 90m

As Z is lesser than H_L, It is a low dam.

5.2.1 Calculation of geometry of Dam

Top width, a = $\sqrt{\frac{H_1}{3.28}}$

= $\sqrt{\frac{20}{3.28}}$ = 2.46 ≈ 2.5m

Base width, B = $\frac{H_1}{\sqrt{S_C - C}}$

C=1 (When uplift is there)

B = $\frac{20}{\sqrt{2.4-1}}$

$=16.90 \approx 17m$ (a_1)

C = 0 (no uplift)

$B = \frac{20}{\sqrt{2.4}}$

$=12.9 \approx 13m$ (a_2)

Limiting condition, $B = \frac{H_1}{\mu\,(S_C - C)}$

B_{limit} (uplift) = $\frac{20}{0.7(2.4-1)}$

$=20.40m \approx 21m$ (b_1)

B_{limit} (nouplift) = $\frac{20}{0.7(2.4-0)}$

$=11.90 \approx 12m$ (b_2)

For uplift case,

B should be maximum of equation a_1 and b_1

B=21m

For no uplift case,

=13m

The upstream projection from the vertical face required = 1.5m

Total base width (B_1) provided

B_1 = 21+1.5 = 22.5m (uplift)

B_2 = 13+1.5 =14.5m (no uplift)

The upstream batter start at a depth of

$2a\sqrt{S_c} = 2 \times 2.5 \times \sqrt{2.4} = 7.75m$

From below the maximum water level and it ends at a depth of

$$3.1 \times a \times \sqrt{S_c} = 3.1 \times 2.5 \times \sqrt{2.4} = 12m$$

Stability Check When Reservoir is Full

Calculation of P_e from Zanger's formula $P_e = 0.726 p_e H$,

Where $p_e = C_m . k_h . \gamma_w . H$

$C_m = 0.735 \text{ x } \frac{\theta}{90^0}\left(\tan\theta = \frac{17}{2.5} = 6.8, \theta = 81.63^0\right)$

$C_m = 0.735 \times \frac{81.63}{90^0} = 0.667$

$$P_e = 0.667 \text{ x } 0.1 \text{ x } 10 \text{ x } 17 = 11.339$$

$$P_e = 0.726 \text{ x } 11.339 \text{ x } 17 = 139.94 k\text{N}$$

$$M_e = 0.412\, Pe\, H = 0.412 \text{ x } 139.94 \text{ x } 17 = 980.18\ KN$$

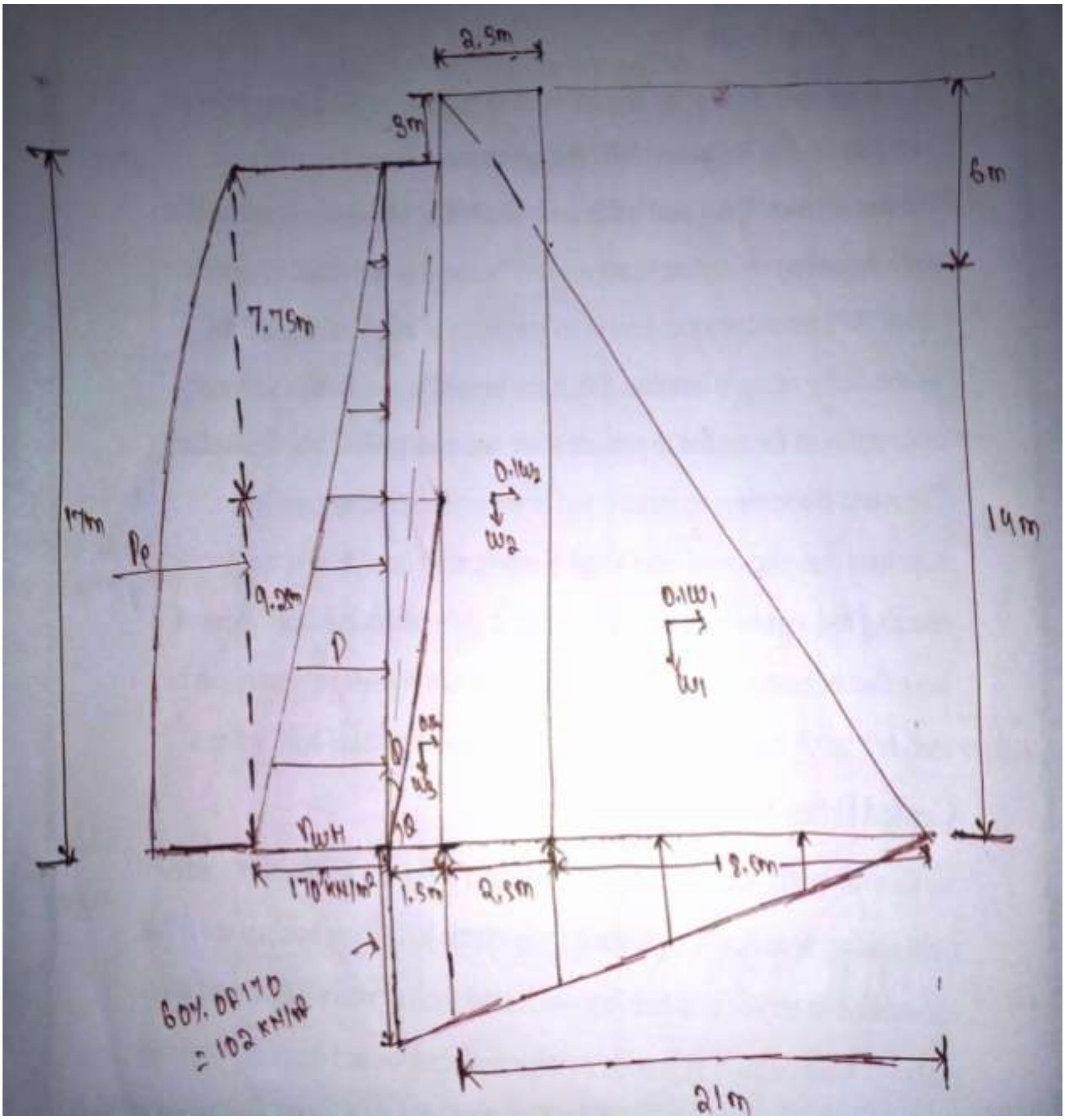

Figure 10: Dam crossection of reservoir full case

Table 3: Reservoir full case

Name of forces	Designation	Magnitude of forces(in kN)		Lever arm (in m)	Moment about toe(M) anticlock wise(+ve) Clock wise (-ve)
		Vertical (V) d/w=+ve u/w=-ve	Horizontal(H) towards u/s=+ve d/s=-ve		
Weight of dam	W_1	(+) $\frac{1}{2} \times 18.5 \times 14 \times 24 = 3108$		12.33	(+)38321.64

	W_2	$(+)2.5 \times 20 \times 24 = 1200$		19.75	(+)23700
	W_3	$(+)\frac{1}{2} \times 1.5 \times 9.25 \times 24 = 166.5$		21.5	(+)3579.75
		$\Sigma V_4 = 4474.5$			$\Sigma M_4 = 65601.39$
Weight of water supported on u/s slope	-	$(+)7.75 \times 1.5 \times 10 \times 1 = 116.25$		21.75	(+)2528.43
	-	$(+)\frac{1}{2} \times 9.25 \times 1.5 \times 10 = 69.375$		22	(+)1526.25
		$\Sigma V_2 = 185.625$			$\Sigma M_2 = 4054.68$
Uplift force	U_1	$(-)\frac{1}{2} \times 22.5 \times 102 = (-)\ 1147.5$		15	(−)17212.5
		$\Sigma V_3 = 185.625 = (-)1147.5$			$\Sigma M_3 = (-)17212.5$
Upward vertical earthquake force(0.05w)		$\Sigma V_4 = (-)0.05V_1 = (-)0.05 \times 4474.5$			$(-)0.05\ M_1 = (-)0.05 \times 65601.39$
		$\Sigma V_4 = (-)\ 223.725$			$\Sigma M_4 = (-)3280.06$
Horizontal hydrostatic pressure	P		$(-)\frac{1}{2} \times 170 \times 17 \text{ x } 1 = (-)\ 1445$	5.64	(−) 8193.15
			$\Sigma H_a = (-)1445$		$\Sigma M_5 = (-)8193.15$
Horizontal hydrostatic dynamic pressure	P_e		$\Sigma H_2 = (-)139.94$		$\Sigma M_e = (-)980.18$
Horizontal inertia forces due to earthquake forces	PW_1		$(-)\ 0.1\ W_1 = 310.8$	4.67	(−)1451.43
	PW_2		$(-)\ 0.1\ W_2 = 120$	10	(−)1200
	PW_3		$(-)\ 0.1\ W_3 = 16.65$	3.08	(−)51.28
			$\Sigma H_3 = (-)\ 447.45$		$\Sigma M_7 = (-)2702.71$

Reservoir full with all forces including uplift

$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3 + \Sigma M_4 + \Sigma M_5 + \Sigma M_e + \Sigma M_7$$

=65601.39+4054.68-17212.5-3280.06-8193.15-980.18-2702.71=37287.47kN/m

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_3 + \Sigma V_4$$

=4474.5+185.625-1147.5-223.725=3288.9kN

$$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3$$

$$= -1445 - 139.94 - 447.45 = -2032.39$$

$$\bar{X} = \frac{\Sigma M}{\Sigma V} = \frac{37287.47}{3288.9} = 11.33$$

$$e = \frac{B}{2} - \bar{X} = \frac{22.5}{2} - 11.33 = 0.08 < \frac{B}{6} = 3.75$$

Average vertical stress= $\frac{\Sigma V}{B} = \frac{3288.9}{22.5} = 146.17$ kN/ m^3

$P_{max/min} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right] = 146.17 \text{ x } \left[1 \pm \frac{6 \ x\ 0.08}{22.5}\right]$

P_v (at toe)=149.28<3000 kN/m^2 (safe)
P_v (at heel)=143.05<420kN/m^2 (safe)

Principal stresses at toe $(\sigma) = P_V.\ \sec^2\alpha - p'p' tan\alpha$

Where, $P_v = 149.28$, $p' = 0$, $\tan\alpha = \frac{21}{20} = 1.05$

$\sigma = 149.28\times(1+\tan^2\alpha) = 149.28(1+1.05^2) = 313.86 < 3000 kNm^2$(safe)

Principal stresses at heel $(\sigma_1) = P_{V(heel)}\sec^2\theta - (P + P_e)\tan^2$

$=143.05(1+\tan^2\theta)-(170+11.39)\tan^2\theta$

$=143.05(1+0.1^2)-(170+11.39)\,0.1^2=142.66<420kN/m^2$ (safe)

Shear stress at toe, τ_0 (toe) $= (P_{v(toe)} - p')\tan\alpha = 149.28\times1.05 = 156.74kN/m^2$
Shear stress at heel, $\tau_0(heel) = -[P_{v(heel)} - (P + P_e)]\tan\theta$
$= [143.05 - (170 + 11.33)] \times 0.1 = -(-3.82)\ kN/m^2$

Factor of safety against overturning $= \frac{(+)\Sigma M}{(-)\Sigma M} = \frac{(+)69656}{(-)32368.6} = 2.15$ (*Safe*)

Factor of safety against sliding $= \frac{\mu\Sigma V}{\Sigma H}\ \frac{0.7\ x\ 3288.9}{2032.39} = 1.13 > 1$ (*Safe*)

Shear friction factor = $= \frac{\mu\Sigma V+B.q}{\Sigma H} = \frac{0.7\ x\ 3288.9+22.5\ x\ 1400}{2032.39} = 16.63$ (Safe)

where,
q =Average shear strength of the joint which varies from about 1400 kN/m^2
μ = Generally varies from 0.65 to 0.75

Reservoir full without uplift

$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_4 + \Sigma M_5 + \Sigma M_e + \Sigma M_7$$

=65601.39+4054.68-3280.06-8193.15-980.18-2702.71=+54499.97

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_4$$

=4474.5+185.625-223.725=+4436.4

$$\overline{X} = \frac{\Sigma M}{\Sigma V} = 12.28$$

$$e= \frac{B}{2} - \overline{X} = \frac{22.5}{2} - 12.28 = -1.03 < \frac{B}{6} = 3.75$$

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] = \frac{4436.4}{22.5}\left[1 \pm \frac{6\ x\ 1.03}{22.5}\right]$$

P_v (at toe)=-251.33kN/m^2<3000kN/m^2 (safe)
P_v (at heel)=143.01kN/m^2<3000 kN/m^2(safe)

Principal stresses at toe $(\sigma) = P_V.\sec^2\alpha - p'\tan^2\alpha = 251.33 \times (1 + 1.05^2) = -528.42$

Principal stresses at heel $(\sigma_1) = P_{V(heel)}\sec^2\theta - (P + P_e)\tan^2\theta$

$=143.01\times(1+1.01^2)-(170+11.39)0.1^2$

$=142.6kN/m^2<420$(safe)

Shear stress at toe, $\tau_0 = (P_V - p')\tan\alpha = 251.33\times1.05 = 263.896kN/m^2$

Reservoir is empty

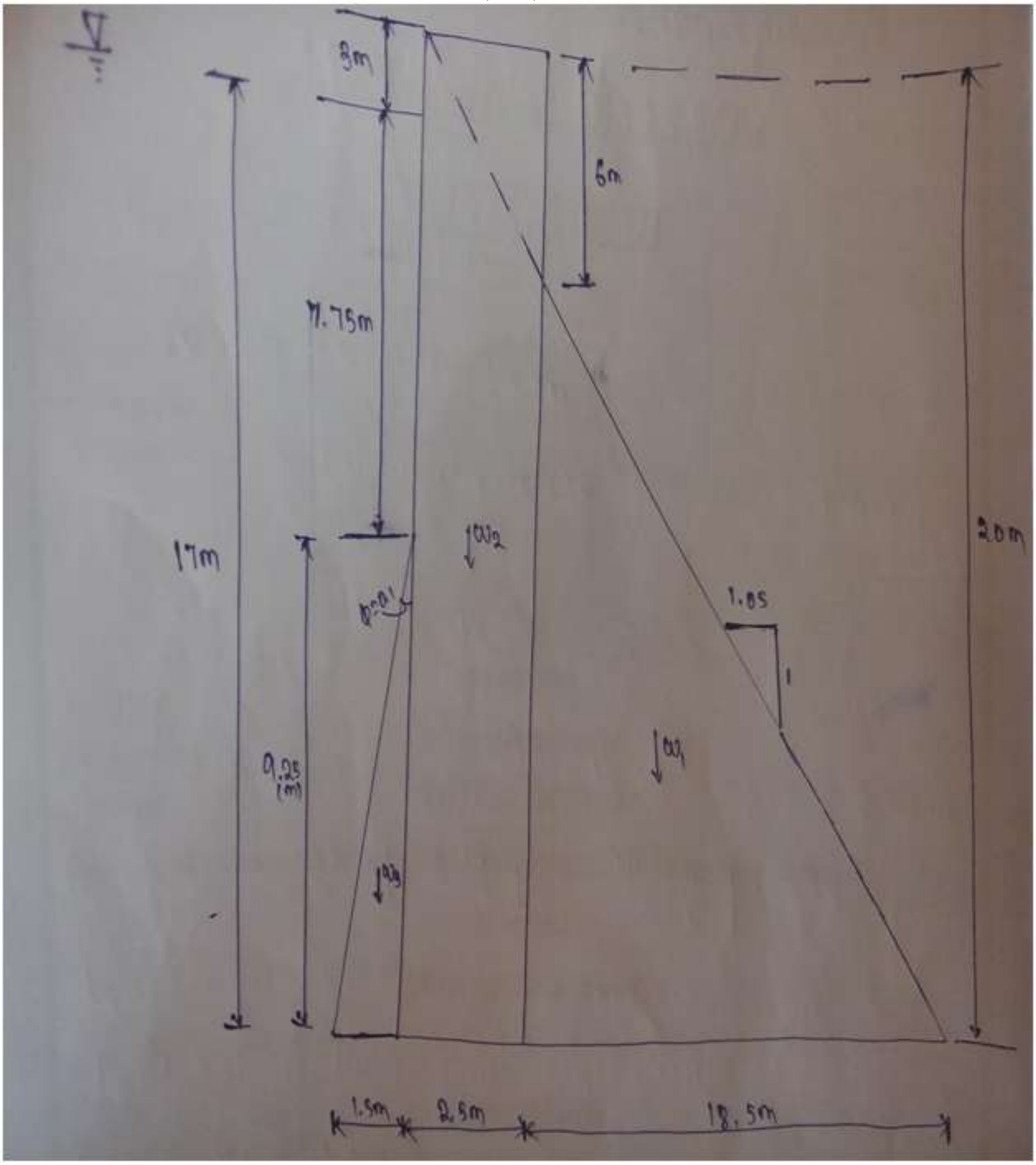

Figure 11: Dam crossection of reservoir empty case

Table 4: Reservior empty case

Name of forces	Designation	Magnitude of forces (in kN)		Lever arm (in m)	Moment about to toe (anticlock wise +ve)
		Vertical	Horizontal		
Downward Weight of Dam	W_1	(+) $\frac{1}{2}$ × 18.5 × 14 × 24 =3108		12.33	(+)38321.64
	W_2	(+)2.5×20 × 24 = 1200		19.75	(+)23700
	W_3	(+) $\frac{1}{2}$ × 1.5 × 9.25 × 24 =166.51		21.5	(+)3579.75
		ΣV_1= 4474.5			ΣM_1=65601.39
Horizontal earthquake forces	PW_1		$0.1W_1$=310.8	4.67	(+)1431.43
	PW_2		$0.1W_2$=120	10	(+)1200
	PW_3		$0.1W_3$=16.65	3.08	(+)51.28
			ΣH = 447.45		ΣM_2=2702.71
Vertical	ΣV_2	0.05 × ΣV_1= 0.05 × 4474.5			0.05 × ΣW_1= 0.05 × 65601.39

earthquake forces		$\Sigma V_2 = 223.725$			$\Sigma M_3 = 3280.06$

Reservior empty and vertical earthquake forces are acting downward

$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3 = 65601.39 + 2702.71 + 3280.06 = 71584.16$$

$$\Sigma V = \Sigma V_1 + \Sigma V_2 = 4474.5 + 225.725 = 4698.225$$

$$\overline{X} = \frac{\Sigma M}{\Sigma V} = \frac{5024.04}{250.775} = 15.29$$

$$e = -\frac{B}{2} - \overline{X} = \frac{22.5}{2} - 15.29 = -3.98 > \frac{B}{6} = 3.75$$

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] = \frac{4698.225}{22.5}\left[1 \pm \frac{6 \, x \, 3.98}{22.5}\right]$$

$P_V(\text{at toe}) = -12.807 kN/m^2 < 420 kN/m^2 (\text{safe})$

$P_V(\text{at heel}) = -430.42 kN/m^2 < 3000 kN/m^2 (\text{safe})$

Average vertical stresses, $\frac{\Sigma V}{B} = 208.81 kNm^2 < 3000 (\text{safe})$

Principal stresses at toe(σ) = $P_V \sec^2\alpha = -12.807 \times (1 + 1.05^2)$

$= 26.926 kN/m^2 < 420 kN/m^2$ (safe)

Where, $\sec^2\theta = 1 + \tan^2\alpha = 1 + 1.05^2$

Principal stresses at heel ($\sigma 1$) = $P_{V(heel)} \sec^2\alpha = 430.42 \times (1 + 0.1^2)$

$= 434.72 kN/m^2 < 3000 kN/m^2$ (safe)
Where, $\sec^2\theta = 1 + \tan^2\alpha = 1 + 0.1$

$$\text{Shear stresses at toe, } \tau_{0(toe)} = (P_{V(toe)})\tan\alpha = 12.807 \times 1.05 = -13.44 kN/m^2 (\text{safe})$$

$$\text{Shear stress at heel, } \tau_{0(heel)} = P_{V(heel)}.\ \tan\theta$$

$$= 430.42 \times 0.1 = 43.042 kN/m^2 (\text{safe})$$

Reservoir empty with vertical earthquake force are acting upward

$\Sigma M = \Sigma M_1 + \Sigma M_2 - \Sigma M_3 = 65601.39 + 2702.71 - 3280.06 = 65024.04$

$\Sigma V = \Sigma V_1 - \Sigma V_2 = 4474.5 - 223.725 = 4256.775$

$$\overline{X} = \frac{\Sigma M}{\Sigma V} = \frac{5024.04}{250.775} = 15.29$$

$$e = \frac{B}{2} - \overline{X} = \frac{22.5}{2} - 15.29 = -4.04 > \frac{B}{6} = 3.75$$

Average vertical stresses = $\frac{\Sigma V}{B} = \frac{4250.775}{22.5} = 188.92 < 3000$ (safe)

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] = 188.92\left[1 \pm \frac{6 \, x \, 4.04}{22.5}\right]$$

P_v (at heel) $= -\frac{392.44kN}{m^2} < 3000kN/m^2$(safe)

P_v (at toe) $= -14.60kN/m^2 < 420kN/m^2$ (safe)

Principal stresses at toe(σ) = P_v. $\sec^2\alpha = -14.60 \times (1 + 1.05^2)$

$=30.69kN/m^2$

Stresses at toe, $\tau_{0(toe)} = (P_{v(toe)}).\tan\alpha = -14.60 \times 1.05 = -15.33kN/m^2$

5.3 Design of spillway

$Q = C.\ L_e.\ H_e^{3/2}$

Where $L_e = L - 2[NK_p + K_a].\ H_e$

Let us assume,

L_e = L= clear waterway=25×13=325m

[As number of span= 13, each span of clear width=25m]

Therefore, 10372=2.2×325× $H_e^{3/2}$

$H_e^{3/2}\frac{10372}{2.2\ x\ 325} = 4.5$

$H_e = 5.94$

The height of spillway above the river bed

H=10.73m

Since, h/Hd = 10.73/5.94 = 1.8 > 1.33,

It is a high spillway.

The effect of velocity head can be neglected.

Since (He + h)/He = (5.94 + 10.73)/5.94 = 2.8 > 1.7

The discharge coefficient is not effected by water conditions, and the spillway remains a high spillway.

Upstream Slope:

The upstream face of dam and spillway is proposed to be kept vertical.
A batter of 1:10 will be provided from stability considerations in the lower part. The batter is small and will not have any effect on the coefficient of discharge.

Effective length of spillway:

$$Le = L - 2[N.Kp + Ka].He$$
$$Kp = 0.01$$
$$Ka = 0.1$$

Number of piers=12

Assuming that the actual value H_e is slightly more than the approximate value worked out (i.e 5.94), say, let it be 6.0m

Therefore, $L_e = 325 - 2[12 \times 0.01 + 0.1] \times 6 = 322.36$

$Q = 2.2 \times 322.36 \times H_e^{3/2}$

$H_e^{3/2} = \frac{10372}{2.2 \, x \, 322.36}$

$H_e = 5.98 = 6$m(assumed)

Hence, the assumed for calculating L_e is all right the creet profile will be designed for H_d=5.98 (neglecting velocity head)

Downstream profile:
$X^{1.85} = 2.\ H_d^{0.35}.\ y1.85$

$y = \frac{X^{185}}{2 \, x \, H_d x \, 0.85} = \frac{X^{185}}{2 \, x \, 5.98 x \, 0.85} = \frac{X^{185}}{9.14}$

The downstream slope of dam is given to be 1.05H :11

Hence $\frac{dy}{dx} = \frac{1}{1.05}$

$\frac{dy}{dx} = (1.85 \text{ X } x^{0.85})/\ 9.14 = 1/\ 1.05$

$X^{0.85} = \frac{9.14}{1.05 \, X \, 1.85} = 4.4$

X = 6.18

Y= (6.18)1.85/9.14 = 3.17

Table 5: Downstream profile of spillway

Horizontal dimension(X)	Vertical dimension(Y)=($X^{1.85}$)/(9.14)
1	0.109
2	0.394
3	0.835
4	1.421
5	2.148
6	3.01
6.18	3.17

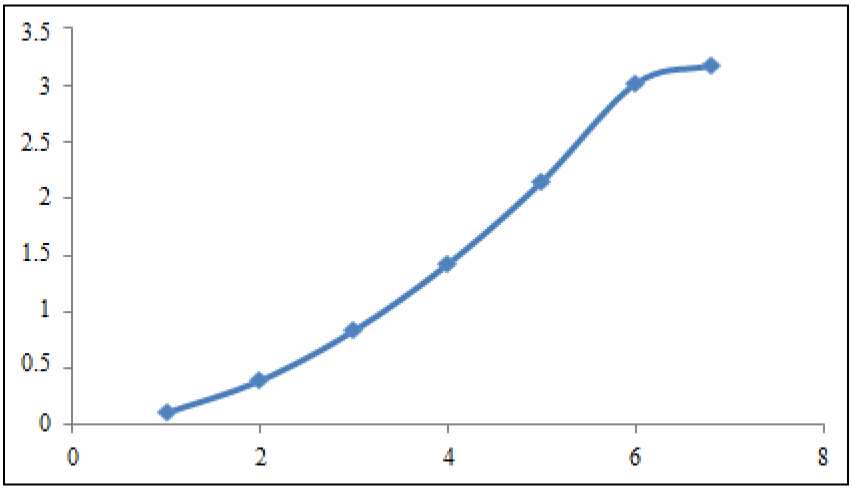

Figure 12: Spillway downstream profile

Upstream profile:

$$y = \frac{0.724\,(x+0.27\,X\,H_d)1.85}{H_d 0.85} + 0.126 \text{ x } H_d - 0.4315\,H_d 0.375\,(x + 0.27H_d)\,0.62$$

Using $H_d = 5.98$, we get

$$y = \frac{0.724\,(x+0.27\,X\,Hd)1.85}{(5.98)0.85} + 0.125 \text{ x } 5.98 - 0.431\text{x } 5.98^{0.35}\,(x + 0.27\text{x } 5.98)^{\,0.625}$$

$$= \frac{0.724\,(x+1.614)1.85}{4.57} + 0.753 - 0.843 \text{ x } (x + 1.614)\,0.625$$

$= 0.158 \text{ x } [x + 1.614]\,1.85 + 0.753 - 0.843 \text{ x } [x + 1.614]^{0.625}$

These curve should go up to $x=-0.27\times H_d$

$X= - 0.27\times.98=-1.614$m

Table 6: Upstream profile of spillway

x in metres	y in metres
-0.25	0.01
-0.5	0.044
-0.75	0.104
-1.0	0.195
-1.25	0.329
-1.50	0.538
-1.614	0.753

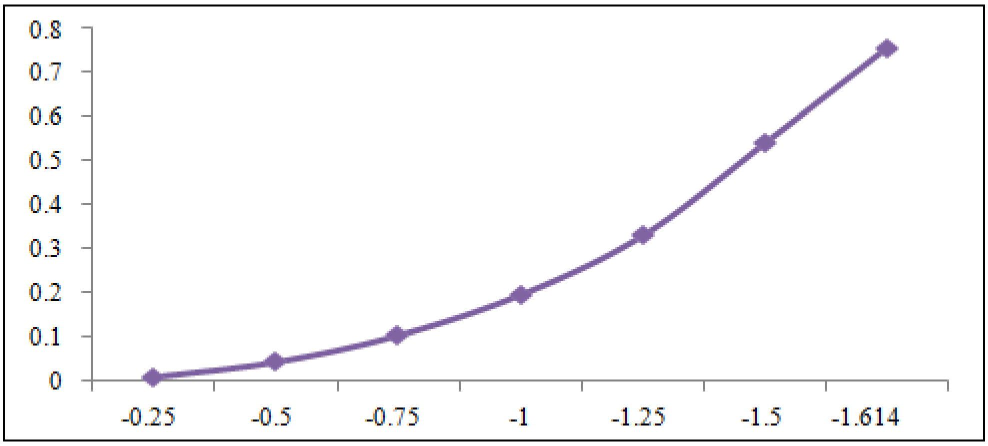

Figure 13: Spillway upstream profile

5. Conclusion

The section of gravity dam should be chosen in such a way that it is the most economical section and satisfy all the conditions and requirement of stability. The maximum discharge estimated is 10372m^3/s.

The height of a dam we are designing is 20m. So it is a low dam. The final width of the dam is 22.5m, top width is 2.5m,the freeboard we are considering is 3m and the upstream batter starts at a depth of 7.75m from below the maximum water level and it ends at a depth of 12m. The factor of safety against overturning is 2.15, factor of safety against sliding is 1.13, and shear friction factor is 16.63. When the reservoir is full with all forces including uplift, the average vertical stress is 146.17 KN/m^2 ,P_{max}=149.28 KN/m^2 ,P_{min}= 143.05 KN/m^2 . The principal stress and shear stress at toe are 313.86 KN/m^{2}and 156.74 KN/m^2. The principal stress and shear stress at heel are 142.66 KN/m^2 and 3.82 KN/m^2 respectively. Hencethe dam is safe against overturning, crushing, sliding and all other necessary factors.

Here we are providing the ogee spillway. The effective length of spillway is 325m having 13 numbers of spans. Each span of clear width 25m. The crest profile is designed for design head 5.98m. The downstream profile for vertical upstream face and the upstream profile for a vertical upstream face are designed.

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