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

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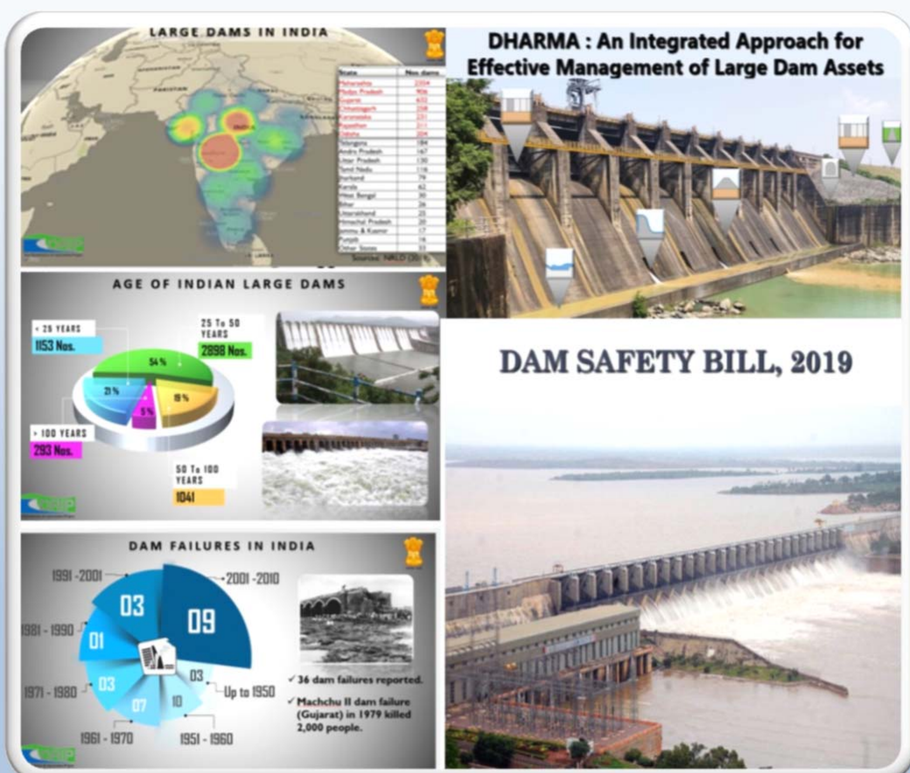
Module VI: Dam Safety and Instrumentation



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Dam Safety and Instrumentation

INDEX

Chapter 1

Introduction to the Safety of Dams

GENERAL

- The growth of civilization is inextricably woven around the availability of water the world over. Dams are human device for exploitation of water for irrigation; hydropower development; water supply; flood management; recreation etc, and thus occupy a pivotal role in the developmental activities of the human race.
- Dams however are not unmixed blessings. They do pose a major hazard in the unlikely event of a failure. Dam failures are rated as one of the major low probability, high loss events. The large number of dams that are 50 or more years old is a matter of great concern. Many of the older dams are characterized by increased hazard potential due to downstream development and increased risk due to structural deterioration or inadequate spillway capacity.
- The International Commission of Large Dams (ICOLD) have tabulated about 200 notable failures in the twentieth century in the world (Tables 1 & 2).
- An analysis of the causes of failure indicate that failures have not only occurred in dams built without application of engineering principles; but also in dams built to accepted state of art in dam engineering. The failure of the Malpasset dam in France in the year 1959 is one such example.
- In such cases, it is essential to remember that failures of this kind are; unfortunately; essential and inevitable links in the chain of progress in the realm of engineering, because there are no other means for detecting the limit to the validity of our concepts.

Table 1: Significant dam failures in twentieth century

Year	Number of significant failures
Period to 1900	38
1900 to 1909	15
1910 to 1919	25
1920 to 1929	33
1930 to 1939	15
1940 to 1949	11
1950 to 1959	30
1960 to 1965	10
Date unknown	25
Total	202

Table 2: List of important dam failures

Dam	Country	Year of Disaster	Lives lost
South Fork (21.9m, earthfill)	USA	1889	2209
Malpasset	France	1959	421
Vaiont(265m,thin arch)	Italy	1963	2600
Teton	USA	1976	11
Machhu(II) (26m earthfill)	India	1979	2000
Tailings dam	Italy	1985	200
Kantle	Sri Lanka	1986	100

- The failure need not be a consequence of an error in the engineering design. Investigations would reveal a factor or factors which in the past has not received the attention which it requires. The fact that it become manifest in the case of a dam failure is an occurrence lying at the border of our knowledge and is governed by the law of statistics , and these laws hit at random.

- The engineers involved in the design and construction of a dam that has failed and the innocent victims are the ones who pay one of the many fees which nature has stipulated for the advancement in the realm of dam construction.

- That disasters strike at random is evident from the list of important dam failures in Table 2.

- The International Congress on Large Dams (ICOLD) has been the pioneer in projecting various aspects of dam engineering since its inception to ensure proper design and construction of safe dams. However, it was during its conference in New Delhi in 1979 that a more direct and forceful action by the international body in the field of dam safety was suggested, as under:

- Several dam incidents with severe consequences during recent years has given rise to general concern about the safety of dams, and indicate the necessity for the introduction of a formal safety approach.

- The height of new dams and the volume of new reservoirs is increasing, while more older dams are approaching an age at which material deterioration and decreasing operational reliability may dictate some repair and upgrading. Certainly, both the growing dimensions of the new dams and the ageing of older dams suggest a somewhat more rigid approach to safety aspects.

- An ever increasing number of dams is being built in countries with little or no tradition and experience in dam engineering. The formalization of safety considerations and the issuance of summarized safety requirements would be part of the necessary transfer of technological know-how to these countries

- Following the ICOLD session at New Delhi in 1979,and the recommendations of the executive committee meeting during 1982 at Rio De Janeiro on Dam Safety, there was an international conference on dam safety at Coimbra in Portugal during 1984, to highlight the importance on dam safety.

- Realizing the importance of dam safety, many countries in the world initiated action during late seventies and early eighties to establish organizations within the public works department/irrigation and power departments, to deal exclusively with matters concerning the safety of existing dams. Such organizations initiated action to review the safety of dams in their countries and undertake appropriate remedial measures for dams in distress.

SAFETY REVIEW PROGRAM

- The most elaborate safety review programme for dams was undertaken by USA under their National Dam Safety Inspection Program during 1982, wherein an inventory of about 68,000 dams was developed and they were classified according to their potential for loss of life and property damage. About 8,800 high hazard dams were inspected and evaluated. Specific remedial actions were recommended, ranging from more detailed investigations to immediate repair for correction of emergency conditions.

- A similar format has been adopted by many other countries owning significant number of dams. The responsibility for the subsequent inspections, investigations and any remedial works rests with the owners of the dams. In most countries, the action or inaction of dam owners will be monitored by a government agency responsible for the safety of dams.

CAUSES OF DAM FAILURES

Dam Failure Surveys

- A number of studies have been made of dam failures and accidents. The results of one survey conducted by the International Commission on Large Dams (ICOLD) summarized the survey data in the form illustrated by figures 1 to 5. These data pertain to dams more than 15 meters in height and include only failures resulting in water releases downstream.

- Fig 1 shows the relative importance of the three main causes of failures: overtopping, foundation defects and piping. While the predominant reason for the failure of concrete dams is due to foundation problems, embankment dam failures are mainly accounted for by piping and seepage. Overall, these three causes have about the same rate of incidence.

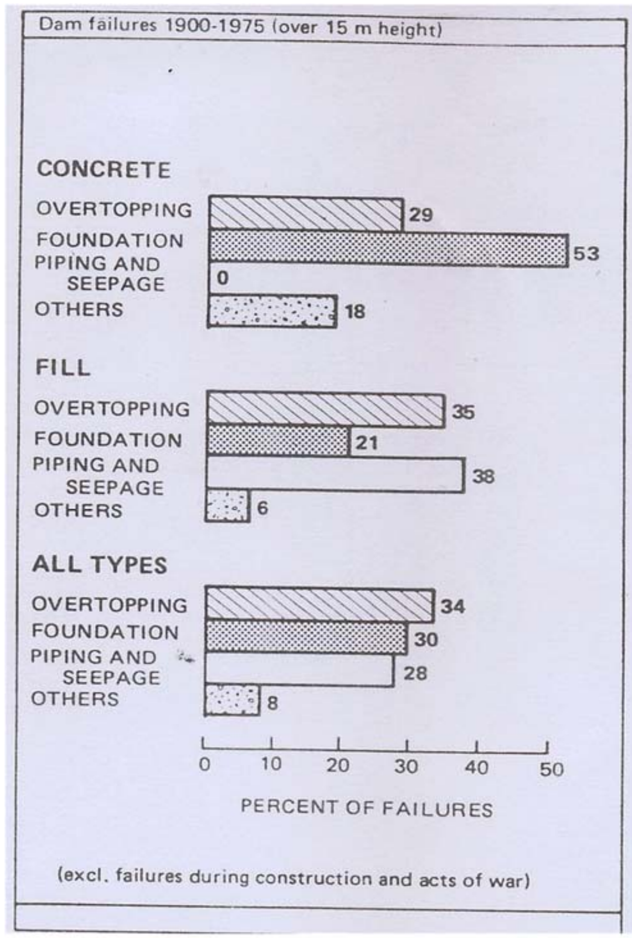


Fig 1: Causes of failure. SOURCE: ICOLD (1973)

- Figure 2 gives the incidence of the causes of failure as a function of the dam's age at the time of failure. It can be seen that foundation failures occur relatively early, while the other causes may take much longer to materialize.

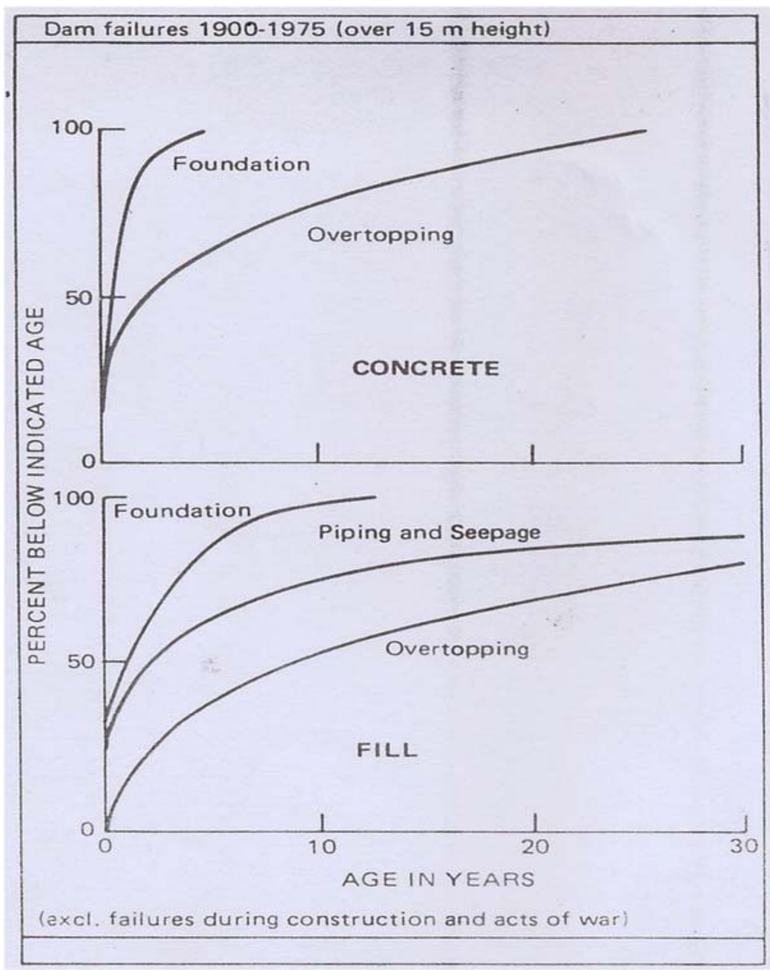


Fig. 2: Age at failure. SOURCE: ICOLD (1973)

- Figure 3 compares the heights of failed dams to those of all dams built and shows that 50% of the failed dams are between 15 and 20 meters high.

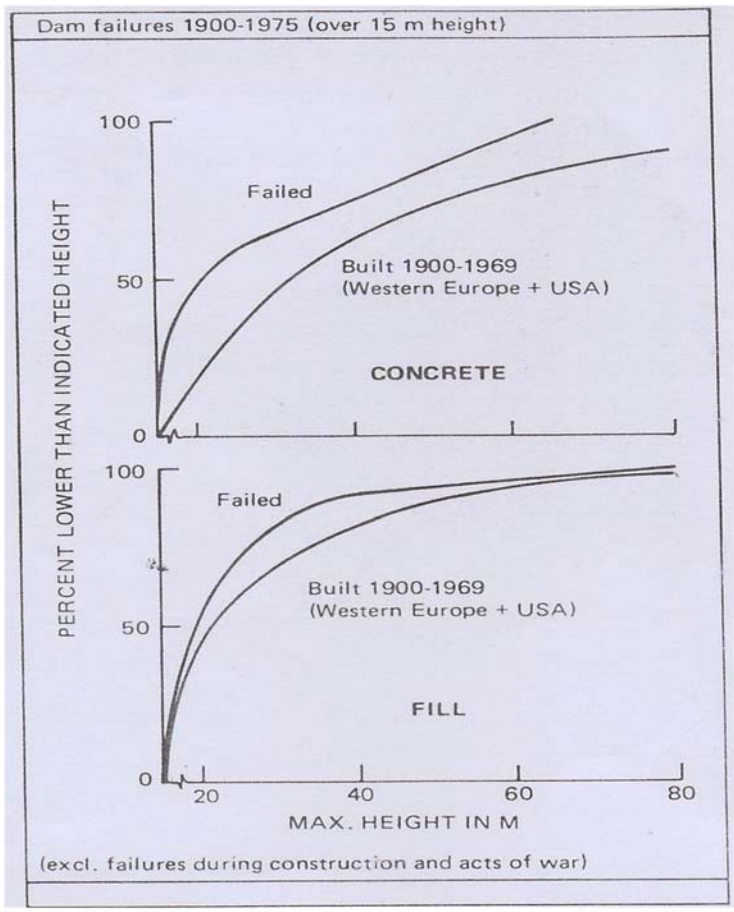


Fig. 3: Height of dams. SOURCE: ICOLD (1973)

- Figure 4 shows the relation between dams built and failed for the various types of dams from 1900 to 1969. According to the bottom graph, gravity dams are the safest, followed by arch and fill dams. Buttress dams have the poorest record, but are also the ones used least.

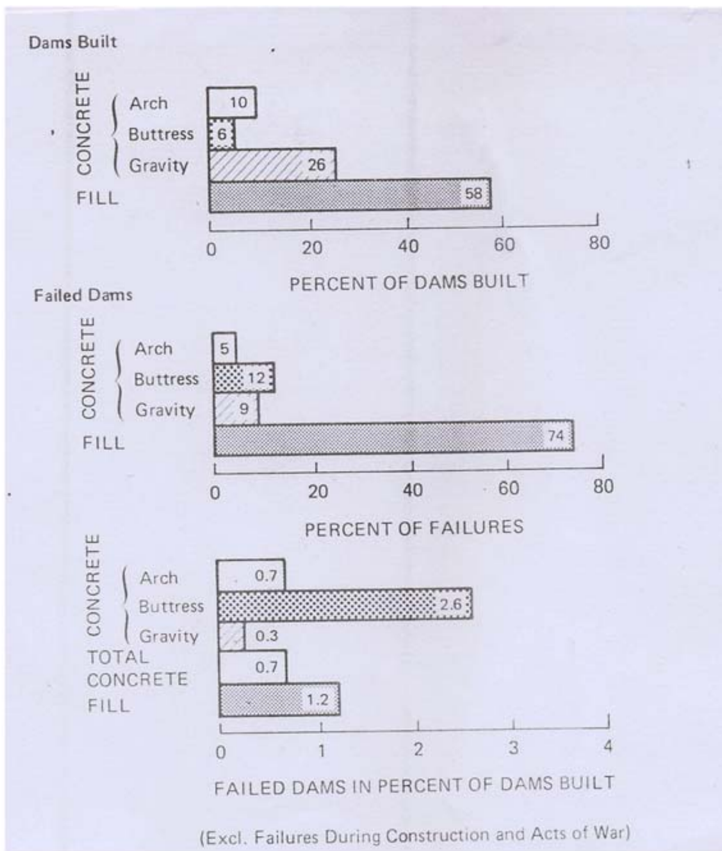


Fig. 4: Dam types (Western Europe & USA 1900-1969)

- Figure 5 shows the improvement of the rate of failure over the 1900-1975 period. The upper graph is in semilogarithmic scale and gives the percentage of failed dams in relation to all dams in operation or at risk at a given time. The lower graph gives the proportion of the built dams that later failed and shows that modern fill and concrete dams are about equally safe.

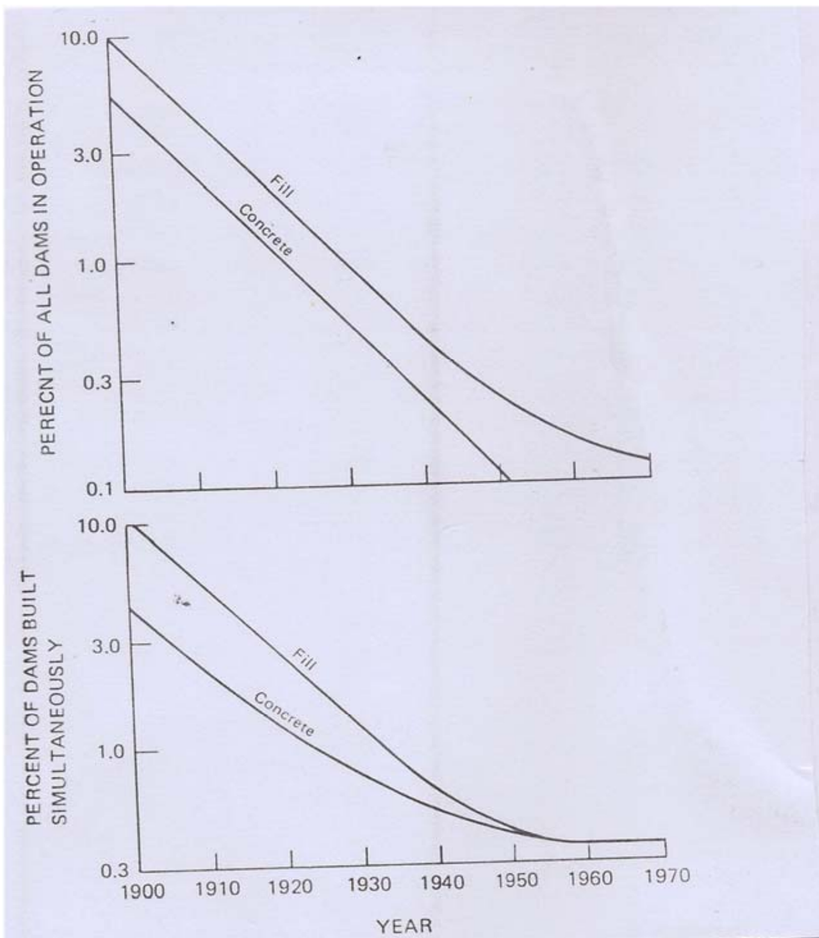


Fig.5: Probability of failure. Western Europe & USA)

- The United States Committee on Large Dams (USCOLD) and the American Society of Civil Engineers (ASCE) had jointly prepared a study which covered failures and accidents to dams till the year 1979. The information of this study is shown in Table 3. The "triggering" was considered as the principal cause of failure. For example, where a sliding failure was due to overtopping flows that eroded the foundation at the toe of a concrete dam, overtopping was listed as the cause of failure.

Failure Modes and Causes

- As it is often experienced, the modes and causes of failure are varied, multiple and often complex and interrelated, i.e. often the triggering cause may not truly have resulted in failure had the dam not had a secondary weakness. These causes illustrate the need for careful, critical review of all facets of a dam.

- This could be illustrated by the failure of a dam in Florida in USA which was lost due to a slope failure, triggered by seepage erosion of fine sandy soils at the embankment toe. The soils lacked sufficient cohesion to support holes or cavities normally associated with piping and were removed from the surface of the dam toe by excessive seepage velocities and quantities. The under-mining of the toe by seepage resulted in a structural failure, but the prime cause was the nature of the foundation soils.

Overtopping

- Overtopping caused about 26% of the reported failures and 13% of all incidents as given in Table 3. The principal reason for overtopping was inadequate spillway capacity. However, in 2 failure cases overtopping was attributed to blockage of spillway and 2 others to settlement and erosion of the embankment crest, thus reducing the freeboard. In one of the latter cases, the settlement was large enough to lower the elevation of the top of embankment below that of spillway crest.

- Six concrete dams have failed due to overtopping. Two of the overtopping failures resulted from instability due to erosion of the rock foundation at the toe of the dam, and four were due to washout of an abutment or adjacent embankment structure.

Flow Erosion

- This category includes all incidents caused by erosion except for overtopping, piping and failure of slope protection. Flow erosion caused 17% of the failures and 12% of all reported incidents. Of the 17 failures,14 were at embankment dams, where except in 2 cases, the spillway failed or were washed out. In one instance, the gate structure failed due to erosion of its foundation, and in another, the embankment adjacent to the spillway weir was washed out. In the latter case, overtopping and/or poor compaction of the spillway-embankment interface was suspected but not confirmed.

- With respect to the 3 concrete dam failures, the spillways were destroyed in 2 instances and in the other, a small buttress dam, the entire dam was destroyed.

Slope Protection Damage

- Damage to slope protection was not reported to be involved in any failures; however, in one accident the undermining of rip-rap by wave action led to embankment erosion very nearly breaching the dam.

- The 13 reported accidents represent about 4% of all incidents. Of the 13 accidents,6 involved concrete protection and the others rip-rap. In some of the latter cases, the wave action pulled fill material through the rip-rap, and in the others, rip-rap was either too small or not durable.

Embankment Leakage and Piping

- Embankment leakage and piping accounted for 22% of the failures and 13% of all reported incidents. In 5 of the 37 incidents, piping is known to have occurred along an outlet conduit or at the interface with abutment or concrete gravity structure

Foundation Leakage & Piping

- Foundation leakage and piping accounted for 17% of all failures and 24% of all reported incidents. It is the number one cause of all reported incidents. 5 concrete dams, one steel dam and 11 embankment dams were involved in the 17 failures.

- In at least 11 of the 49 accidents, which involved 6 concrete and 43 embankment dams, the leakage occurred in the abutments. Some reports cite inadequate grouting or relief wells and drains as causing the leakage and piping.

Sliding

- This category covers instability as represented by sliding in foundations or the embankment or abutment slopes. Sliding accounted for 6% of all failures and 12% of all incidents reported.

- Of the seven failures, 1 was a concrete gravity structure where, during first filling, the structures slide downstream of about 45 cm was preceded by an abutment slide downstream followed by large quantities of water leaking from the ground just downstream of the dam. The reservoir was emptied successfully, but before repairs were accomplished, the reservoir filled again causing large sections of the dam to overturn or open like a door.

- The five embankment failures occurred in the downstream slopes, one due to excessively steep slopes and the others probably due to excessive seepage forces.
- All the 28 reported sliding incidents involved embankment dams. In 2 cases; the slides occurred in abutment slopes, in 10 cases in the downstream slope, in 11 cases in the upstream slope, and in 2 case in both the upstream and downstream slopes.

Deformation

- This category involves instability cases other than sliding. Of the 6 failures, three involved timber crib dams, where either the logs slipped out of their sockets or ice or flood flows breached the dam.
- The other 3 failures were embankment dams where, in one case, deformation of the outlet pipe permitted the outward leakage of the full flowing pipe, causing piping of the embankment. In another case, ice pressures displaced the intake riser of the outlet works. In the third embankment dam, the concrete intake riser collapsed, with resulting leakage and piping along the conduit barrel.
- Of the 31 reported accidents, 29 occurred at embankment dams. However, in 19 of these cases, the outlet spillway was involved. In 5 instances, the accident occurred in tunnels where serious leakage developed. In the remaining 7 cases, excessive cracking, shearing, or collapse of outlet pipes occurred.

Deterioration

- Two of the failures and nine of the accidents were caused by deterioration. The two failures involved corrosion of outlet pipes, which allowed leakage and piping of embankment material into the outlet.
- The 9 accidents involved 3 embankment dams and 6 concrete dams. At the 3 embankment dams, leakage with piping of embankment material into the conduit was caused by pipe erosion in 2 cases and by concrete deterioration in the other.
- At 3 concrete dams, the accidents were due to concrete deterioration caused by freeze-thaw

damage. At another, alkali reactivity was the cause. Corrosion of the penstock and deterioration of the timber bulkhead were listed as causes of the accidents at 2 concrete dams.

Earthquake Instability

- Three incidents of earthquake instability were reported. Two of these were the Lower and Upper San Fernando dams that were damaged during the 1971 San Fernando earthquake. These were listed as accidents because reservoir water was not released downstream. However, complete reconstruction of the dams was required. The other was the Hebgen dam in Montana, which was damaged by the 1959 Madison Valley earthquake.

Faulty Construction

- Faulty construction was listed as the cause for 2 failures and 3 accidents. The failures occurred in concrete gravity dams and in one case was attributed to the omission of reinforcing steel. The 3 accidents occurred at embankment dams and in 2 cases were caused by poor bonding between old and new embankment material. At the third, poor concrete tunnel construction led to severe leakage through construction joints.

Gate Failures

- Spillway gate failure was listed as the cause of failure of the dam in 2 cases. Gate or valve failure was the cause of 5 of the reported accidents, resulting in damage to downstream structures and/or loss of reservoir pool.

Effects of Age and Aging

- Weathering and mechanical and chemical agents can gradually lead to accident or failure unless subtle changes are detected and counteracted. The engineering properties of both the foundation and materials composing a dam can be altered by chemical changes that occur with time. Dams constructed for the purpose of water quality control, sewage disposal and storing mining wastes are particularly susceptible to changes from chemical action.

- Foundation shearing strengths and bearing capacities can be reduced and permeabilities can be increased by dissolution. The permeabilities of critically precise filter zones and drain elements can be reduced or obstructed by precipitates. The effectiveness of cement grout curtains can be reduced by softening, solutioning and chemical attack.

- Concrete can gradually deteriorate and weaken from leaching and frost action. Alkali-aggregate reaction in concrete is irreversible and can gradually destroy the integrity of the structure.

- The metal components of appurtenant structures, such as trash racks, pipe, gates, valves and hoists, gradually corrode unless continuously maintained. Deterioration can be rapid in an acidic environment.

- Many dam failures could be cited to illustrate complex causes and the difficulty of identifying a simple, single root cause. For example, the 1976 Teton failure in USA may be attributed to seepage failure (piping). But several contributing physical and institutional causes were identified by an independent panel of experts that reviewed the cause of Teton dam failure.

- The complex interrelationship of failure modes and causes makes it extremely difficult to attribute a single major cause for a dam failure. It also explains why different evaluators could arrive at different conclusions regarding prime causes.

- All the more reason for a review of all facets of a dam to be conducted periodically by experts in the field of dam engineering.

Chapter - 2

Case Studies on Dam Rehabilitation

Case Studies on Dam Rehabilitation

- Construction of dams as is generally understood in the present context with its regular technical features and appurtenances commenced in India a century back. These were designed and constructed based on the then existing state of art.

- Compared to present day standards they would come under the category of "inadequate design" as certain features in the dam and foundation considered necessary at present would not be available in them, viz. chimney filters, drainage gallery, uplift release arrangement etc.

- Some have functioned well in spite of such inadequacies. On the other hand, some dams of recent origin, with all the current technical features had problems which were investigated and remedial measures applied.

- Issues of rehabilitation have been elaborated by examples of distress problems related to dams in this country. Specific distress related to individual dams are discussed in this article along with the remedial measures applied.

- The examples are not exhaustive in nature, but have been chosen carefully from a select list of cases covering concrete, masonry and earth dams. These are samples where adequate documentation is available for correct appreciation of the distress and remedial measures applied.

Hirakud Dam

- The Hirakud Dam in Orissa completed in 1957 is a 59m high composite structure of earth, masonry and concrete, constructed across river Mahanadi at Burla in Sambalpur district for the purpose of irrigation, hydropower and flood control. It impounds a huge reservoir of gross capacity 8.105 billion cubic meter. A cross section of the spillway is at Figure 1.

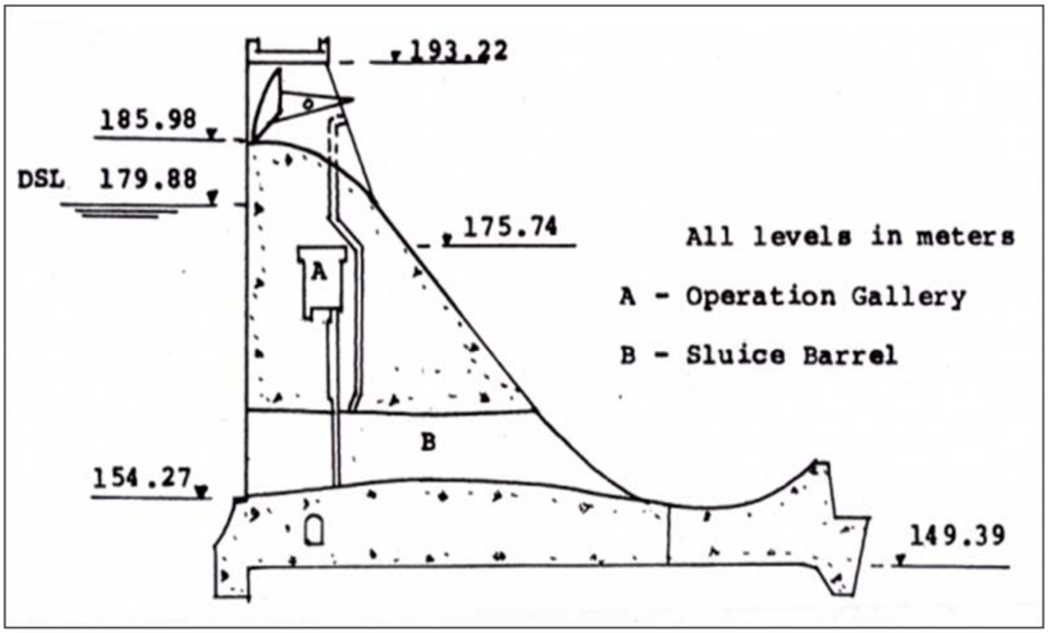


Fig.1: Hirakud dam spillway cross section

- At the time of completion of the dam, cracks were noticed in the operation gallery of the right concrete spillway. During initial filling, some time in 1956, cracks were noticed in the sluice barrel of the right spillway. There was bulging of concrete leading to difficulties in operation of sluice gates.
- **There** was excessive seepage in operation and foundation galleries at spillway blocks and also in powerhouse blocks. Cement grouting of cracks in 1963 and epoxy grouting of cracks in sluice barrels in 1975 did not solve the problem.

- At a later stage it was discovered that the upstream face of the dam had cracked, and these were mapped underwater. For the right spillway, the extent of cracking on the face of the spillway was 1,600 meters and 600 meters in the sluices. The surface of the left spillway was also cracked but to a much less extent compared to the right spillway. Details of cracks on the upstream face of the right spillway underwater is shown in Figure 2.

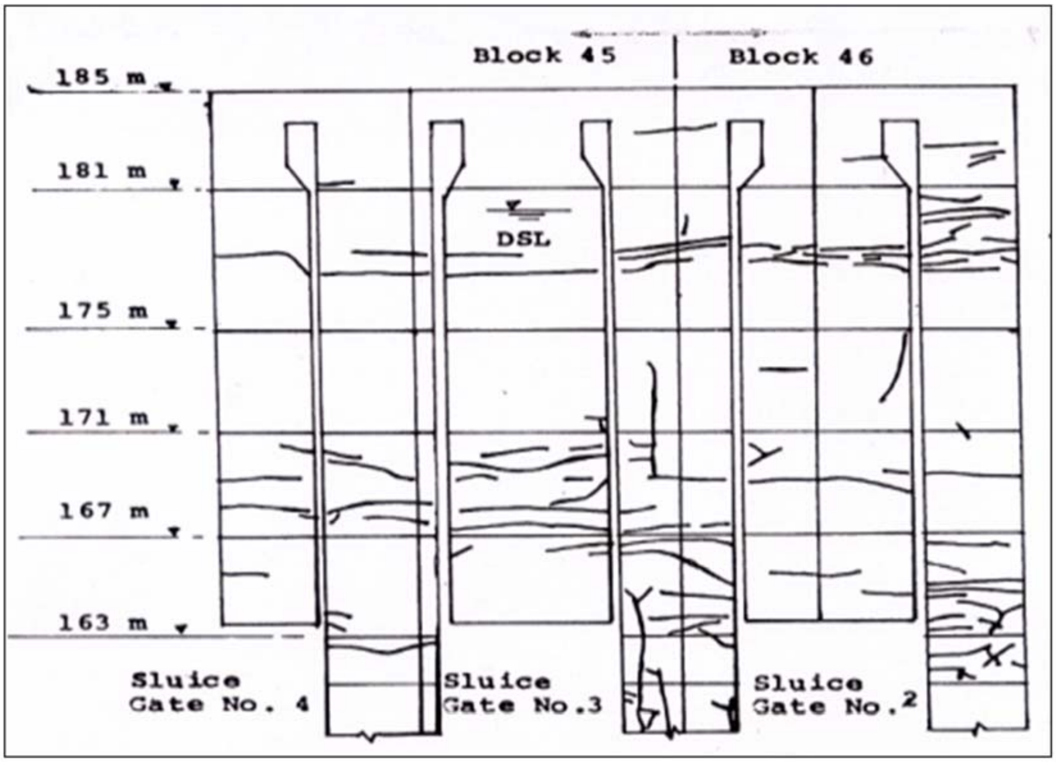


Fig. 2: Spillway section and cracks on upstream face underwater of Hirakud dam

- An expert committee constituted by the government of Orissa in 1981 opined in their report submitted in 1983, that cracks were due to opening up of lift joints owing to thermal stresses because of rapid rate of placement of concrete at the time of construction and later due to quicker access of moisture through the thermal joints. The alkali - aggregate reaction hastened continuance of further cracks.
- A detailed underwater photography of cracks in the year 1990 showed an increase in the length of cracks and the presence of honeycombing, pop outs, spalling and large holes on the upstream of the dam and also on the upstream side of sluice barrels submerged under water all in the right spillway.
- The expert Committee recommended the following remedial measures:
  a. Sealing of cracks in exposed faces, and underwater treatment of cracks on the upstream face.
  b. Grouting of the body of the dam with suitable chemicals.
  c. Chipping out bulgings and deflections in concrete walls of adit gallery, gate shafts, spillway radial gates. Removing weak concrete and replacing with sound concrete.
  d. Drilling of fresh drainage holes in foundation gallery and redrilling of porous drains in the body of the dam.
  e. Check efficiency of grout curtain and to resort to further grouting if necessary.
  f. Providing shear pins in block 46 as an experimental measure.

- The project authorities had undertaken remedial measures recommended by the expert Committee from (a) to (d) over a period of six years between 1990 to 1996 with reasonable success.

- Sealing of cracks on the upstream face of the dam underwater was given top priority. The job was carried out by the Indian associates of the European Structural Bonding Division of Netherlands using their own formulation for sealing and grouting of cracks underwater, in a situation where water is moving.

- The material used for surface sealing of cracks underwater is Concressive 1448 and is used along with silica powder as filler material for sealing wider cracks.

- For grouting these cracks underwater, Concressive 1380 is used. This material has a viscosity of 410 cps at 250C and developed a bond strength of 20 to 24 kg/cm2 after 7 days of curing, which is higher than the tensile strength of concrete. For finer cracks, Concressive 1468 having a lower viscosity of 200 cps at 250C and having a bond strength of 15.5 to 19.0 kg/cm2 after 7 days of curing was preferred.

- Underwater repairs were carried out by a team of divers operating from a pontoon placed adjacent to the upstream face in the reservoir. Application material, tools and tackle for sealing and grouting as well as arrangements for diving were all placed on the pontoon. Monitoring of the work and subsequent surveillance was done with the help of a remote operated underwater vehicle that was fitted with cameras and its movement underwater was controlled from a console placed on top of the dam.

Konar Dam

- Konar dam in erstwhile Bihar (now Jharkhand) is a 58 meter high earthen dam with a concrete spillway across Konar river, a tributary of Damodar completed in 1955 for the purpose of irrigation, flood control and industrial water supply. It is owned by the Damodar Valley Corporation. A cross section of the spillway is shown in Figure 3.

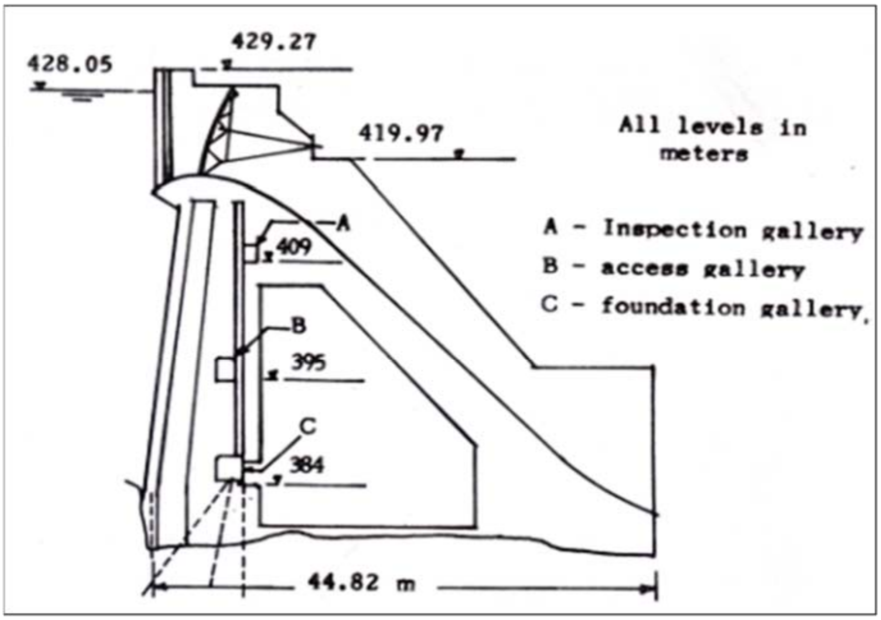


Fig. 3: Konar dam spillway cross section

• About 8 years after construction in 1963, cracks were observed on the walls of the inspection gallery. Comparatively less cracks were observed in the middle (access) gallery and much less in the lowest (foundation) gallery. The disposition of cracks is shown in Figure 4. No remedial measures were undertaken before 1970, except restricting the reservoir level.

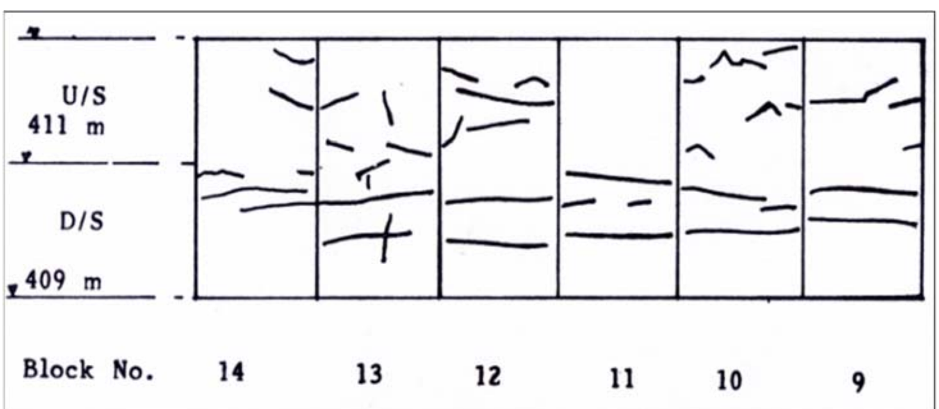


Fig. 4: Konar dam. Disposition of cracks in topmost gallery

- The penetration of cracks were first studied through sonic tests in 1969. Thereafter, grouting was carried out by injection of epoxy in the majority of cracks and cement in some places. This operation was carried during 1970-71. The remedial measure was not successful, as cracks reappeared in 1973. The finer cracks which were not grouted continued extending and widening. These cracks appeared either a little above the grouted cracks or along the original cracks.

- Realising the complexity of the problem, an expert Committee of prominent engineers were appointed by DVC in 1981 to analyse the causes of distress and to recommend remedial measures.

- The Committee suggested a variety of investigations like monitoring the cracks with dial gauges, observing deflection by plumb bob, observing solar radiation on the downstream face of the dam.

- Apparently it was considered that the continuous extreme exposure to sun of the downstream face of the dam, resulting in its continuous heating, while the upstream face remain cool because of shade and contact with water has a definite influence on the cyclic movement of cracks. During 1981, this hypothesis was required to be corroborated by observation and analysis.

- The project engineers had been monitoring the movement of cracks since early seventies with the help of dial gauges and an analysis of the dial gauge observations showed that the cracks are wider in the central blocks of the dam (block 13/14) and narrower at abutment ends. the width of such cracks is of the order of 2 mm or more. The depth of cracks vary from 1 meter to 4 meters.

- The majority of cracks open up when the reservoir level recedes and the cracks close when the lake level increases. Generally, the behavior of cracks on the downstream face of the gallery are identical at any time. However, same is not true for gauges located on the upstream face of the gallery.

- The opening or closing of cracks on the downstream face of the gallery is in general greater than those on the upstream face of the gallery. This is valid for both during rise and fall in lake level.

- The movement of cracks (in the inspection gallery) both vertical and horizontal at both ends of a spillway block are not equal. The movement being more at non-pier ends.

- The temperature effect on the cracks has been exhibited by excess opening of cracks in summer as compared to the cracking behavior in winter months. The effect of rising of lake level and the effect of lowering of air temperature are the same and similar are the effects of lowering the lake level and rising of air temperature.

- The behavior of cracks in the inspection gallery (which are very active) over a period of time (1974-81), as against variation of reservoir level have been plotted and is shown in Figure 5. The behavior of cracks continue to remain identical beyond 1981.

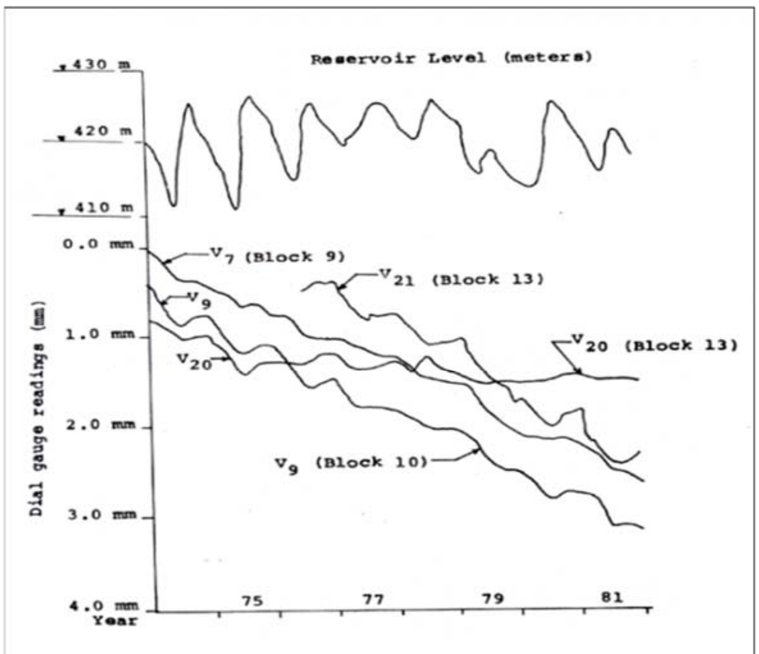


Fig. 5: Konar dam: Behaviour of cracks in inspection gallery

- Having known the behavior of the cracks through observations as already indicated, structural analysis of the dam was conducted by the Finite Element Method, taking into account stresses due to temperature variation between the downstream face and the gallery with a gradient of 26 to 29 degree Celsius per meter as verified in the field. The stresses obtained by adding thermal load to gravity and water loads at the base of the inspection gallery showed that surface temperature effects led to initiate cracking.

- The maximum tensile stresses occurred in summer at the vicinity of upper inspection gallery. The higher stress of the order of 18.0 kg/cm2 occur on the downstream side of the gallery, but much lower stresses occur on the upstream gallery face which is of the order of 8.0 kg/cm2

- During winter, tensile stresses develops on the downstream and upstream faces of the dam, the magnitude of which does not exceed 13.5 kg/cm2. Compressive stresses occur in the interior of the dam.

- The stress distribution is consistent with wider and more prominent cracks on the downstream face of the inspection gallery.

Talakalale Dam

- The dam is a masonry dam of 62.5 m height across Sharavati river in Karnataka completed in the year 1963. This dam impounds a balancing reservoir for the Sharavati hydro-electric project.

- During its first filling in 1963, heavy seepages were observed on the downstream face of the dam in many reaches. Grouting of the downstream face of the dam reduced the seepage. Subsequently, the entire body of the dam was grouted from the top in 1967, resulting in significant reduction of seepage from 3.91 cusec to 0.23 cusec.

- There was however an increase in seepage from 1970 onwards. A cross section of the dam including the earth backing is shown in Figure 6.

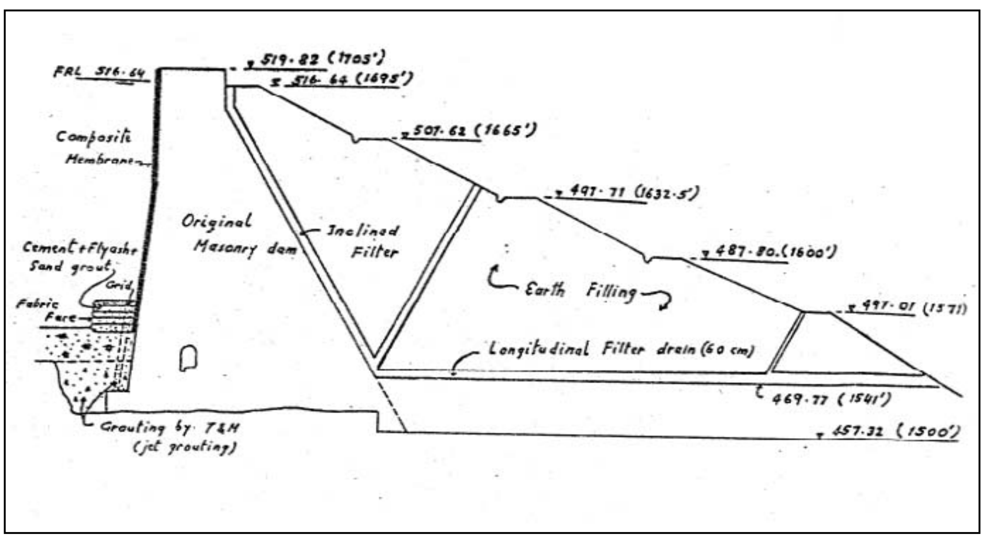


Fig. 6: Strengthening Talakalale Masonry Dam

- Underwater inspection of the upstream face of the dam in 1974 revealed the existence of open joints and suction points at depths varying from 40 ft to 80 ft below Full Reservoir level upto which the divers could go. Dye tests confirmed direct connection of cracks between upstream and downstream faces of the dam. Since most of the drainage holes were clogged, three experimental drainage holes were drilled from the drainage gallery, which resulted in huge quantities of water gushing into the gallery. These were immediately plugged.

- In order to reduce the tensile stress at the heel of the dam, resulting from the reduction in density of masonry, earth backing on the downstream face was carried out between 1976-79. Grouting of the dam from the top to render it water tight had to be stopped since grout was finding its way into the reservoir.

- Rescanning of the upstream face of the dam by the Indian Navy in 1987 indicated suction points on the upstream face which were plugged. Colgrouting the body of the dam was carried out for 100 feet length of the dam.

- The washing out of the binding material (mortar) from the upstream face of the dam continues through the foundation gallery and also through the downstream face of the masonry dam, passing into the filters provided for the earth backing. Chemical analysis on the sample of solids collected in the drainage gallery indicate presence of heavy leaching and increase in the percentage of lime content as calcium carbonate from 34.56% in 1979 to 98.21% in 1982. The leaching of the binding material has been a continuous process since completion of the dam in 1963.

- In the past, several dams have been strengthened by grouting, prestressing, buttressing and by earth backing. The problem with earth backing is that, it may sometimes be difficult for the filter and drainage arrangement to cope with the seepage. Thus it is necessary to ensure adequate control of seepage right at the upstream face of the dam through measures such as installation of membrane and by grouting, if the earth backing has to perform the necessary function.

- The Consultants; Messers Coyne-et-Bellier of France who were commissioned to study the problem by the State government suggested mounting a membrane system of stable synthetic polymer material along with resin capsules to bond the membrane system to the existing masonry surface.

- The seepage hazard will therefore arise only in the portion of the dam below the lower boundary of the upstream membrane. This zone can be treated by grouting the upstream debris which would be mainly silt deposits by suitable techniques, using a mixture of sand, flyash and cement.

Mulla Periyar Dam

- The Mulla Periyar dam built across the river Periyar, at the Kerala-Tamil Nadu border completed in the year 1895 is a 54m high masonry dam operated and maintained by the Tamil Nadu state. The front and rear faces of the dam were built with uncoursed rubble masonry in lime sukhi mortar of 2:1:3 with a core of concrete in between. The hearting concrete was in lime surkhi mortar using 6 cm size broken stones, the proportion of broken stone to mortar being 3.125:1. Cross section of the dam is at Figure 7.

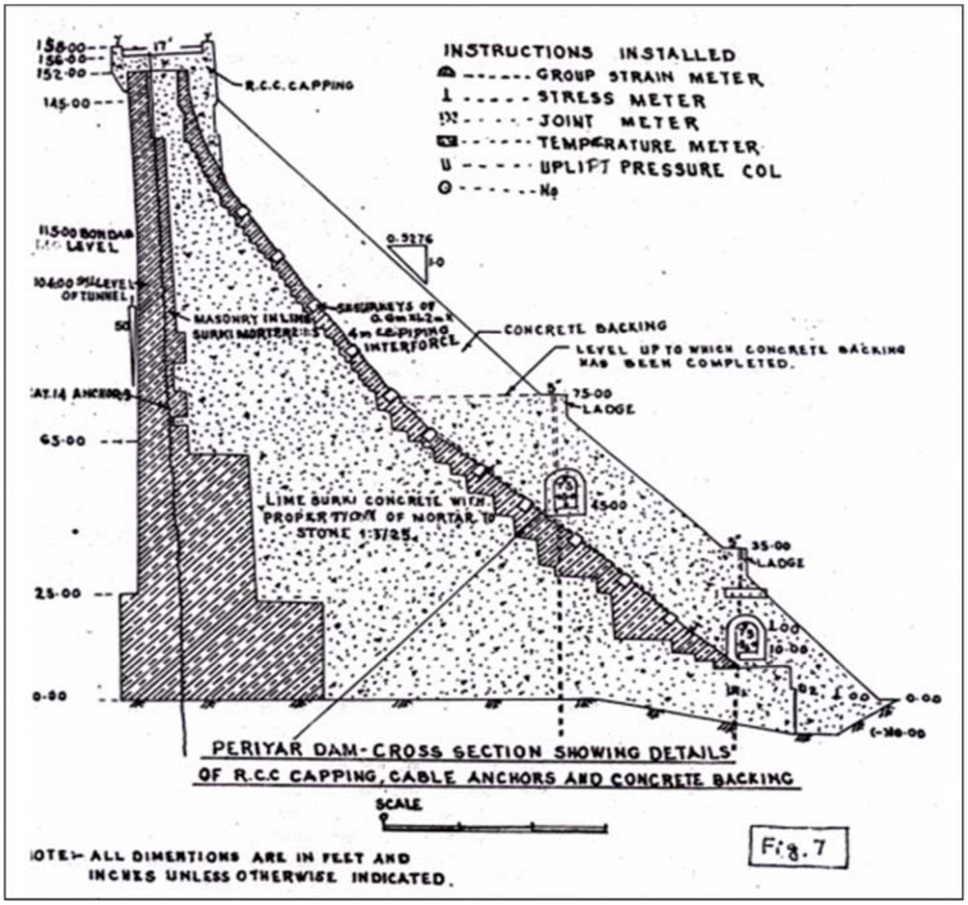


Fig. 7: Mulla Periyar dam cross section

- Drilling and grouting either for foundation consolidation or for impervious curtain was not done since these operations were not developed at that time. Similarly, provision for drainage gallery and release of uplift was not made and contraction joints were also not provided.
- Around the year 1930, the leakage from the dam was about 9.5 litres per second and the quantity of lime leached was found to be 3.1 gm per gallon of water. The dam was grouted with cement between May 1930 to June 1932 which reduced the seepage to 4.75 litres per second and the leaching of lime to 0.78 gm per gallon of water. The grouting

operation was followed by guniting the front face from 40.23m (132 ft) to 46.33 m (152 ft). The guniting material consisted of Portland cement, hydraulic lime and sand.

- Lime leaching on the downstream face of the dam in 1960 led to the repetition of cement grouting of the dam body as was done in 1930. The depth of grouting was however quite high as some of the holes were drilled through the foundation base rock for a substantial depth below the masonry. In this case, grouting reduced the leakage from 7.6 litres per second to 1.70 litres per second. It was also seen that the seepage water did not contain any lime. The seepage in terms of quantity continues to be significantly small till date and chemical analysis of seepage water shows only occasional traces of lime.

- A review of the structural safety of the dam in early 1970 showed that the stresses at the heel at an elevation of 19.81 m (65 ft), having an abrupt reduction in cross section is 2.29 kg/cm2 in tension taking into account uplift forces and 3.12 kg/cm2 in tension if earthquake forces are also taken into account.

- As a result of the structural review, the project authorities in 1979 decided to undertake the following remedial measures:

- Maintain the water level in reservoir to 41.45 m (136 ft)

- Provide RCC capping over the dam.

- Anchoring the dam section with the foundation rock through high tension cables.

- Strengthening the existing dam with concrete backing on the rear face.

- The regulator gates were raised in December 1979 to bring down the water level in the reservoir, and in 18 days, it stabilized to 41.45 m.

- The parapets of the dam were knocked out, and the top one meter of the dam was dismantled.

- The capping slab was designed for one meter thickness, from EL 46.25 (152 ft) to 47.25 m (155 ft) with adequate reinforcements, and it was made wide enough to provide a 5.2 m clear roadway on top at El 47.25 m, against the original width of 2.4 m. RCC parapets of 0.6 m (2 ft) thickness were raised to 48.16 (158 ft) the same height as the old parapet. The RCC capping provided at the top of the section added an additional weight of 33.63 tonnes per meter run of the dam and to that extent improved the stability of the gravity structure.

- Stress analysis showed that a tensile stress of 0.90 kg/cm2 could develop at 19.81 m (65ft), even with the lake level at 44.21 m (145 ft) when uplift and seismic forces are considered. The RCC capping provided extra gravitational weight and reduced the maximum tension at EL 19.81 m by0.22 kg/cm2 leaving 0.68 kg/cm2 as balance to be nullified.

- For taking care of the balance tension as already stated, cable anchoring the body of the dam to the foundation was resorted to. since the dam body was in lime surkhi concrete, cable anchoring was done so that the cables run through this material.

- The spacing between the cables were kept at 2.74 m (9 ft). High tension steel wires of 7mm dia, 34 numbers in each group was provided to carry a load of 120 tonnes. The cables had a maximum length of 6.1 m (20 ft), grouted anchorage in rock. The anchoring operation was carried out at 95 points over a length of 365.85m (1,200 ft) of the main dam. The cable anchoring job which was started in February 1981 took ten months to be completed.

- As a permanent measure for strengthening, provision of concrete backing was preferred to other alternatives such as earth backing or buttressing.

- For placement of backing concrete, foundation treatment up to a depth of 12 m was undertaken by drilling and grouting. For the interface treatment, shear keys of 60 cm depth at 4.0 m centres were provided. Appropriate measures were taken to ensure proper bonding of the masonry surface with new concrete.

  For effective drainage at interface, a network of horizontal and vertical drains were provided. These drains were connected to the drainage gallery. The old dam does not have any drainage gallery. In the backing concrete , drainage holes have been provided, close to the interface at elevations of 3.05 m (10 ft) and 13.72 m (45 ft). Holes have been drilled from the foundation gallery into the old masonry and taken to a minimum depth of 2m into foundation rock. These measures will reduce uplift in the dam considerably.

Conclusion

- The need for rehabilitation of dams in India is on account of two major factors; viz. dams that are in distress or dams though not in distress which do not measure up to present standards. The distress conditions strike at random and are not limited to old dams built with ancient technology, but also involves modern structures that were built in accordance with the current state of art.

- The experience gained in the rehabilitation of these dams, a few of which are described in this paper, could be used as a source of knowledge for undertaking rehabilitation of dams in future.

- Development of underwater techniques in the rehabilitation of dams and monitoring the efficiency of remedial measures will in future allow remedial measures to be undertaken without depletion of reservoirs.

Chapter 3

Comprehensive Approach to Dam Safety

1.0 Introduction

Unlike a purely technical discipline, dam safety is a techno-management discipline. It can be categorised as a part of asset management for the owner and public safety assurance on part of the government.

The dams are manmade structures and need as much attention as any other engineering structure. However, the dams- like diamonds- are required to last almost for ever and the concerns arising out of an aged dam are quite different than that arising for a skeletal structure like buildings.

One can consider demolition of a building and rebuilding the same. In almost all cases except very small check dams, the option will not be available.

The dam safety assurance programme has to deal with the fundamental concerns mentioned above.

1.1 Historical Background

Dam safety has enjoyed varying degree of attention in inverse proportion to the time and distance from the most recent catastrophic event. Engineers have been understanding and continue to understand the hazards associated with operation and maintenance of a dam.

In the United States, the dam safety aspects received a focussed attention in the aftermath of Teton dam failure and systematic programmes like SEED (Safety Evaluation of Existing Dams) and other measures were taken up. Legislative efforts were also carried out.

In 1979, on recommendation of the state irrigation ministers' conference a Dam Safety Organisation was established at central level to look into dam safety aspects for the country.

Due to a few dam disasters like Machhu II dam in Gujarat, the attention was focussed on the systematic management of the dams for safe operations.

Consequently, a large number of states established their own dam safety organisations and have taken up measures for ensuring dam safety in their respective jurisdictions.

In India, by and large, dams are owned, constructed and maintained by the governments at state or central level. However, the scenario is changing in the light of current thrust on privatisation. Very soon, we will have a significant number of dams which are owned and operated by non-state agencies.

With the increasing population of dams in the country, there have been safety concerns on account of distress phenomena arising out of natural factors as well as dam materials or geotechnical setup related problems have been increasing. There are related issues like prioritization and management of an increasingly ageing population of the dams in the country.

1.2 Components of a Dam Safety Program

Every owner of one or more dams is required to set up a dam safety programme in respect of the dams owned and operated by them.

1.2.1 Organisation

Establishment of a nodal setup for the dam safety related concerns is the prime requirement for a dam safety programme.

In respect of state governments, depending upon the number of dams, the setup comprises of a Chief Engineer (for states having relatively large number of dams) or a Superintending Engineer level officer in- charge of the dam safety management operations. The setup is assisted by the Superintending/ Executive Engineer level officers assigned to specific functions.

The dam safety operations have to be assisted by Design office as well as hydrological analysis personnel for resolving any issues that may arise.

Regular inspection and reporting programme is at the heart of the responsibilities of the dam safety organisation. We will discuss more on the aspect later in the notes.

Sensitization of the field operating personnel in various aspects of observations on a routine basis and keeping them prepared for handling any emergencies that may arise is also the responsibility of the organisation.

The organisation has to ensure the development and implementation of dam specific operation and maintenance manual and revise the same from time to time.

The organisation is also responsible for maintaining the detailed inventory of the engineering features of the dam for use at a later date.

The organisation will also have to act as an interface with the state regulatory agencies for satisfying the public safety needs.

1.2.2 Inspection programme

There are two levels of inspection programmes.

1.2.2.1 Routine inspections

These inspections have to be carried out by trained and experienced engineers from dam safety organisation atleast twice in a year – pre monsoon and post monsoon periods.

During the inspection, the general health of the dam and appurtenant works are examined and preparedness of the dam and hydromechanical structures for handling any reasonably expected flood upto the design flood is reviewed.

Preparedness for the dissemination of flood operation warnings are reviewed and in the post monsoon period, the adequacy of the arrangements are examined.

Records of the observations made by the field personnel are reviewed and the data transmission of the instrumentation data is reviewed.

Any unusual phenomena is noted and flagged for further examination by the experts.

A checklist based approach is followed. A checklist developed by DSO, CWC is given as reference.

Central Dam safety organisation monitors the completion of the inspection programme and provides a report to the owner for further action needed.

1.2.2.2 Comprehensive problem specific inspections

These inspections are ordered when the problems noticed as a part of the routine inspection programme can not be resolved with the technical and managerial expertise at the disposal of the O&M personnel.

At the outset, a specialised team is setup for looking into the specific problem and advise on the remedial measures. Associated personnel from the general discipline of hydrology, seismology, numerical modelling and material scientists are also included in the team for a holistic view of the situation.

The team undertakes the visual inspections from time to time till a satisfactory solution is evolved and orders additional field and laboratory investigations as well as numerical simulations. Review of the results of visual inspection and studies is carried out and is submitted to the dam owner in form of detailed reports including recommendations.

The team also supervises the implementation of the recommendations and consecutive performance of the structure for a suitable period post-remediation.

1.3 Comprehensive Safety Review of Dams

While adequate care is taken at the time of planning and design stage for the dams, the database of natural phenomena is always rather limited at these stages.

The natural loads and phenomena is essentially random in nature and many manifestations are revealed later during the operational phase of the dams.

Comprehensive safety reviews of the dams address these issues and bring the health status of the dam back to that of a new dam which is being designed at the time of review.

In view of ever changing economic importance of the dam as well the changing hazard perceptions for the downstream, the comprehensive safety reviews of a dam should be initiated at regular intervals of typically 10-15 years.

Comprehensive safety reviews should not be associated with occurrence of a distress in the dam. They are also a part of the regular dam safety assurance programme.

1.3.1 Components of a comprehensive safety review of a dam

Hydrological review: to take into account the latest hydro-meteorological data and reassess the design floods as well as flood management capacity of the dam.

Seismological review: Similar to above in respect of the seismic status of the dam. Reassessment of the seismic hazard to the dam and appurtenant works based on the post construction data and current design practices.

Material Properties review: to assess the current mechanical properties of the dam body and foundations as well as abutments. This includes the examination of the reservoir rim areas, which can have an impact on the dam operations and safety. The activity will necessarily involve field and laboratory investigations.

Structural Adequacy Studies: assess the adequacy of the structure in terms of present design standards and present material properties. This may include numerical modelling for the structural, hydraulic and flood wave propagation studies for the dam, spillway and energy dissipation measures.

Downstream hazard assessment and preparedness review: to revise the emergency preparedness of the d/s areas under various types of operational scenarios developed as a result of the above mentioned studies.

Operational policies for the reservoir: to take into account changed input parameters like flood intensities, d/s channel constraints. The recommendations involve change in flood impingement levels, change in the rule curves, provisions of structural and non-structural measures for inflow flood forecasting etc.

Managerial measures of composition of a team and funding for the works can be handled by the dam safety organisation.

1.4 Ensuring Safety of Dams

1.4.1 Structural Modifications/ improvements

Grouting: to improve the structural integrity of the concrete by sealing the cracks, to reduce the water ingress in the body of the dam. The grouting operations involve selection of the materials- typically specialised epoxy based compounds for concrete dams and special formulations for embankment and geotechnical applications. Design of appropriate patterns and pressures as well as criteria for length of operations like grout intake and refusal pressures.

Prestressing: to improve the stability of a concrete dam. The measure was earlier used as a temporary one. However, the same has the potential as a permanent one with the advances in the technology. Careful analysis and evaluation of the material properties of the dam and anchorage area is needed.

Backing: concrete backing on the d/s face of the dam like buttresses has been provided to a number of old concrete/ masonry dams in the country. The behaviour of the strengthened section is quite complex. The treatment of the interface for integrating the structural action between the old and new material requires careful planning and design.

Earlier, there was a practice of providing earth backing to concrete/ masonry dams for improving their stability. Some of the dams especially in South India were designed as earth backed dams. However, it has been found that the measure does not provide any effective stability during earthquake events and the measure is not adopted currently for improvement of stability.

Sliding stability problems: arise out of shear seams in the foundations. Many times, such seams are exposed due to deep scour on d/s of energy dissipaters. Provision of the shear keys has been carried out at a number of places in the country. The planning for such measures for existing dams is quite costly and difficult.

Seepage prevention: for such dams having deteriorated concrete requires provision of impervious membranes on the u/s face. Use of specialised geotextiles and non-woven material is quite popular. For successful implementation, requirement of depletion of the reservoir upto the river bed is often helpful. The measure can be quite successful for old concrete and masonry dams having porous material creating problems of increased uplift.

Rock bolting and stitching the discontinuities in the abutment slopes is an effective measure for improving its sliding stability and prevent safety implications for the dam. Such measures can involve passive bolting or prestressed anchors.

Loading berms are provided for embankment dams for prevention of sinkholes on the d/s due to excessive exit gradient and to impart stability to the slopes. They serve an important function of preventing piping failures and at times have to be provided at a short notice.

Parapet walls are provided to compensate for the inadequate free board on the top of the dams. As a rule, such parapets should be strong enough to withstand the wave impacts and should not have on openings on reservoir side.

1.5 Administrative Measures for Safety Program

Various steps in ensuring the safety of the dams in the portfolio are:

- Setting up a dam safety management unit in the organisation.
- Working out an inspection and reporting workflow process.
- Establishing mechanisms for addressing safety concerns
- Management of serious safety concerns by setting up a standing body of consultants generally known as “Dam Safety Review Panel” DSRP.
- Planning and running a systematic disaster preparedness programme in collaboration with the other stakeholders.

Providing adequate and sustained finances for the O&M activities as well as dam safety programme.

1.6 Oversight Mechanisms in Place

Each state government is entrusted with safety assurance of dams in their jurisdiction.

The central DSO in CWC helps the state level organisations by providing necessary technical expertise and enabling mechanisms for running their programmes.

National committee on dam safety (NCDS) has been setup by government to centrally define and resolve issues related to dam safety at the national and international level.

Central dam safety legislation is in the final stages of approval for adoption across the country. The legislation defines the duties and responsibilities at various levels and lays down the mechanisms for enforcing them.

Government of India has carried out a UNDP project (1989), World Bank aided Dam Safety Assurance and Rehabilitation Project (1999) for generating the awareness and provide necessary financial wherewithal to selected states.

A World Bank loan assistance for Dam Rehabilitation and Improvement Project (DRIP) addressing these areas is being implemented. This project focuses on rehabilitation and improvement of selected dam structures in four participating states. It will improve the safety and operational performance of dams. It addresses shortcomings in maintenance and provide for long term operation of the dams, which in turn will help to keep dams safe, and keep d/s population safe from floods. It will improve central and state-level institutional capacity to sustainably manage dam safety administration and operation & maintenance.

1.7 Various Types of Distress Phenomena

Though the dam safety programme is oriented towards keeping the dam in a safe and sound condition, distresses in the some of the dams out of those in the portfolio can not be avoided. Essentially, the dam design involves extrapolation of the loading and material behaviour estimates derived from relatively short term observations. Many unknown trends can occur in spite of best efforts at the time of design. Such situations lead to distress and operational constraints, which have to be dealt with by the dam safety programme.

1.7.1 Concrete/ Masonry Dams

1.7.1.1 Cracking is a symptom that may arise out of a number of different underlying causes and the remedy at times involves tackling the underlying problem and then treating the cracks proper. Cracking in concrete dams can be attributed The cracking in concrete is attributed to the exhaustion of the extensibility of the concrete. Thermal stresses arise due to excessive heat of hydration generated due to fast rate of construction or inadequate cooling. There can be effects of ambient temperatures on the dam body which can lead to excessive localised thermal stresses. Occurrence of thermal cracking is quite common in Rolled Concrete dams.

Thermal stresses
Alkali Silica Reaction
Foundation and/ or Abutment Settlements
Seismic Loadings
Inadequate stability/ overturning resistance

Cracks due to Alkali silica reaction occur due to the reaction of free alkali in the cement paste with specific minerals in the aggregate in the presence of moisture. Such reaction products are having larger volume and induce pattern cracking in the concrete. The reaction, once set in, is very difficult to contain and the dam body experiences progressive cracking. There is no remedy for preventing the ASR but the moisture ingress can be reduced to the extent possible by providing the treatment to the u/s face like epoxy grouting, membranes etc. in extreme cases, the dams affected with ASR have been replaced with new dams on d/s. While selecting construction materials for the dam, special care of this aspect has to be taken.

Cracks occurring due to thermal or ASR causes, are generally random in nature and do not indicate a favoured trend in their disposition on the surfaces of the concrete. They are characterised as pattern cracking. Many times, these cracks follow the lift joints in the dam body. The cracks due to ASR can be effectively treated only after the reactive potential of the aggregate material is exhausted. In many cases, new cracks may appear by the side of the treated cracks in case of ongoing reactive expansion in the concrete.

Cracks occurring due to excessive loading as is the case for the rest of the causes mentioned above are localised in nature and exhibit a trend commensurate with the direction and magnitude of the loading. Such cracks pose a serious concern to the stability of the dam body and any such occurrence has to be dealt with urgently. The grouting (epoxy or special cements), stitching with active or passive anchors and prestressing of the dam for its overall stability are recommended practices.

1.7.1.2 **Seepage** is a symptom indicating presence of porous material and non-homogeneous materials. The seepage is also a symptom and the underlying cause has to be assessed for taking remedial action for the dam. The excessive seepage –if allowed unchecked- affects the longevity of the dam and generally leaches out the free lime thereby making the concrete porous and unsound. Seepage should be monitored for the dam blocks as a whole as well as for the isolated locations where such concentrated seepage is observed. The seepage appearing in the foundation gallery should be examined for its chemical and physical properties to find the health status of the dam. The treatment of the seepage should be focussed on the u/s face which is the source for the reservoir water to enter into the body and appear as seepage.

1.7.2 Embankment Dams

1.7.2.1 **Seepage and Piping** is a very serious concern for the safety of the embankment dams of all types (rockfill as well as earthfill). The turbidity of the seepage occurring within various reaches of the embankment dams should be closely monitored as the migration of fines first manifest as turbid seepage water. The seepage monitoring mechanisms in the embankment dams at the toe drains usually attract significant vegetative growth and the maintenance of the toe drains and rock toe faces is an important surveillance measure. In most of the embankment dams, there is no systematic isolation of the reaches of the dam for seepage measurement. Such measures should be enforced at all the dams.

Potential piping has to be immediately arrested by provision of appropriate loading berms and inverted filters with proper gradation. The surveillance should be maintained round the clock after the measure has been implemented till such time that the seepage quantum as well as quality comes back to the desired values.

1.7.2.2 Desiccation related Cracking occurs in dams having significant drying periods during the year. The deep desiccation of the core leads to internal shrinkage cracks and can be a cause of failure if the reservoir is filled after a long time. This phemomena is quite prevalent in peninsular India as the ambient temperatures are very high and the reservoir remains empty for almost 3-4 months in a year. Investigations can be carried out in the field by test pits and also the rate of filling of the reservoir has to be closely controlled.

1.7.2.3 Gully erosion on the slopes of the dam occurs with poor maintenance and leads to reduction of section of the dam. Deep gullying can expose the filter and core materials to the surface and can lead to serious failure hazard with high reservoir levels. The problem can be quite acute for the medium sized embankments of the order of 10-15m. The gully formation will also reduce the effective width of the top roadway and may pose problems during monsoon by preventing use of heavy machinery. The phenomena can be tackled by providing adequate surface drainage arrangements on the d/s face and continuous maintenance programme. Gullying can be initiated by local paths setup by animals and people for climbing up the dam face. Access control is the natural solution.

1.7.2.4 Vegetative growth on the d/s face as well in the areas of reservoir subject to long dry periods is a serious concern. The growth at times result in massive trees having deep roots penetrating into the dam body. Eventual death of the tree or uprooting of the same can then lead to potential piping paths. Many embankment dams suffer with this problem which has a simple solution in form of regular maintenance of the slopes.

1.7.2.5 Wave erosion on the u/s pitching should also be observed and managed to reduce the local slope failures at the predominant reservoir levels. Such predominant reservoir levels can be other than the FRL. Careful observation and inspection programme for the u/s face with replacement of the damaged pitching at regular intervals is the solution. In case of severe wave attack, the provision of miniature wave breakers like concrete blocks and tetrapod shapes can also be considered.

1.7.3 Hydro Mechanical Problems

These are encountered for the gates and control structures and become dam safety concerns. One of the most common problems is non-maintenance or delayed maintenance of the dam gates. This leads to sudden failure of the component and sudden release of impounded waters. If the failure is localised, the d/s hazard may not be much but the implications during the flood season are significant almost leading to the safety of the dam as a whole. Provision and implementation of the proper maintenance programme for the mechanical components is often a casualty of the budget cuts in post construction era.

Sometimes, the problems are exhibited due to other associated phenomena like ASR or differential settlements. Such phemomena have to be addressed in totality by providing adequate measures for the underlying causes.

Provision of adequate electrical power under all conditions through fail-safe mechanisms is a prime concern for a dam safety incharge. It has been observed that many catastrophic failures (Machhu II) have taken place due to non-availability of power for operation of gates at a crucial juncture.

1.8 Dam Safety Implications arising out of D/S Concerns

1.8.1 D/S channel capacity restrictions pose serious implications on safety of the dam during the flood operation. In an over populous country, the dam brings up migration of population inside the river gorge and the reduced channel carrying capacity puts a restriction on the maximum allowed spillway discharges. In quite a few cases, the developments around the reservoir rim put restriction on maximum permissible levels in the reservoir. Both these constraints put an unexpected demand on the dam operator and hazard to the dam. The issue being complex in nature, the solution lies in a multi disciplinary approach to the land use planning.

1.8.2 Growth of population and activities in immediate vicinity: The availability of water attracts many industries in the d/s area of the dam and also along the periphery of the reservoir. Nuclear power plants and thermal plants are generally located near such reservoirs and can increase the hazard rating of the dam. In many medium height dams, the nearby villages have a tendency to shift just next to the d/s toe of the dam thereby making the maintenance and surveillance quite difficult. Such implications have to be anticipated and flagged in due time for remedial action.

1.9 Operational Phase Concerns

As against the design and construction phase of the dam, the operational phase of the dam witnesses reduction of focus of the project managers to the other areas of development. Quite naturally, the personnel strength associated with the dam construction reduces drastically for the operation and maintenance phases. In this context, the dam safety manager has to take care of some of the aspects mentioned below

1.9.1 Archive Management: It has been observed as a rule rather than exception that a large wealth of information generated during the construction of the dam gets lost due to sudden shifting of manpower while winding up the construction phase. Systematic efforts are needed by the expert personnel to preserve such records, which may provide insight to the dam materials and other special conditions that were experienced during the construction. Quite often, such data is needed long after the completion of the construction of the dam (almost 50 years or longer) and hampers the planning of remedial action and also increases the costs of investigations at that late stage. Modern day electronic methodologies of document management should be followed.

1.9.2 Training : Extreme events for which the dam has been designed are quite rare and may occur once in a lifetime of a person. However, at the time of occurrence of such an event, no failures can be permitted. Continued training of the dam operational staff at all levels is necessary for maintaining the preparedness of the personnel available at the time of emergency. This is especially true for the flood operations for the dam. Mock exercises for handling the floods of various magnitudes under different types of operational constraints should be devised and mandatorily conducted. In case of a cascade of dams, some of the exercises should also involve the entire cascade. Trainings should involve the central design or operations monitoring units also.

1.9.3 Emergency Preparedness: This is an essential ingredient of dam safety programme. Standard Operating Procedures for meeting various kinds of emergencies should be got defined and documented with the help of experts in the relevant fields. Command and control mechanisms in the organisation should be evolved and codified with appropriate administrative orders. It is desirable to equip and maintain emergency control rooms for use during the emergencies. The control rooms not only have to be setup at the dam site but counterpart facilities should also be setup at the administrative headquarters of the project. With the advent in the communication and data processing technologies, planning effective communications between the dam operators and the control rooms at various locations do not pose a problem. However, while planning the communication facilities, their reliability under extreme weather conditions should be kept in mind.

Chapter 4

Safety Assurances of Hydro-Mechanical Equipments

1. Introduction

1.1 Controlled releases & utilization of water from the Reservoir is an important necessity in order to derive the maximum benefit from any project in the water resources sector in an economical manner. For controlling the flows through the dams, canals, penstocks or outlets for purposes of irrigation, domestic use, flood control, navigation and power, the hydro-mechanical equipments which consists mainly the hydraulic gates and their operating system; form the most vital component. The hydro-mechanical equipment forms a very small component of the total cost of the project, but they are one of the most crucial components in determining the success of the project.

1.2 There have been numerous incidents in the world, in which the gates have malfunctioned sometimes with disastrous results. The major causes of many such incidents and failures are attributed to faulty hydraulic design, maintenance or improper operation, especially in case of high head gates. Phenomenon of cavitations and vibration in and around the high head gate threatens the very safety of the hydraulic gates. Therefore, measure should be evolved at planning and design stage itself to control, if not eliminate, the hazardous consequences by making such hydro-dynamic alignment and layout along the flow path and in the gate body, which assure a positive pressure or at least reduce the negative pressure as much as possible.

1.3 The hydraulic gates are moving facilities provided in dams, barrages, hydropower projects, and reservoir and river control. These are neither like concrete dam nor like other reinforced concrete hydraulic structure, which always remain stagnant. Therefore, the gates are more complicated and critical components than the dam proper and other hydraulic structures. Evidently, gates are of vital importance for water resources projects and should always be in smooth functioning. Their failure to fulfill the intended purpose can have devastating results, sometime may even endanger the safety of the entire project. While most recorded instances of unsatisfactory operation or failures of gates are due to hydrodynamic causes, however mechanical or electrical inadequacies can be just as serious because the structure does not perform as designed. In practice, shortcomings due to hydraulic or hydrodynamic factors are difficult to rectify, whereas mechanical or electrical faults can usually be repaired or improved.

1.4 The higher the head, the more serious the hydraulics problems; the larger the gate area, the more serious the structural problems and manufacturing difficulties; the larger the total dynamic pressure, the more serious the hoist problems. These three parameters, i.e. the water head, the gate opening area and the total dynamic pressure are the major indices, which require to be seen in totality for smooth functioning. The expertise in design, fabrication and erection are important factors, which contributes significantly for assurances of the smooth and satisfactory operation of the gated structure.

The safety to the Hydro Mechanical Equipments can be assured by making appropriate provision/taking appropriate action right from the planning to the maintenance stage. Such provisions/actions are discussed below:

2.0 Planning and Design Stage

The maintenance affects the reliability and safety of any structures significantly. Therefore, the gated structures should always be designed and constructed with facilities for placing emergency, bulkheads/ stop logs gates at upstream of the main gate for attending to its maintenance requirements and as well as for closing /isolating the waterway in case of any emergency/ malfunctioning of service gate (Fig-1 & 2).

Floating bulkheads (Fig-3) may provide a solution in the circumstances, where the provisions are not made initially in the original planning for emergency closure of the bay by means of an emergency/ bulkheads gate but their requirement becomes important and the equipment or space is not available on the dam for handling them. However, the design of such floating bulkheads is complicated in regards to the control for their sinking.

Orifice or tunnel spillways have submerged inlets and such structures should be generally provided with one gate just downstream of the entrance while at the entrance there should be provision of a stop logs/bulkhead gate. The latter is to serve as a guard gate in case of emergency or when the former requires servicing/maintenance (Fig-4).

The installations having water head less than 30m, a Bulk head gate to be installed in the u/s of the regulating gate (slide or fixed wheel type) can be provided for maintenance purposes. Whereas for water head more than 30m, an emergency gate of fixed wheel type for maintenance purposes and as well as a service gate in its downstream is preferable. However, the installation exposed to extreme higher head, two gates in tandem with similar construction is generally adopted (Fig-5).

Special design of gated structures is required for combined operations of gates e.g. flood release combined with sediment flushing. Control gates for such applications are preferably top sealing radial gates and in some case vertical-lift wheel type gates. In either case, the gates are designed for upstream sealing and the maintenance gate is usually vertical-lift type.

The crest gates for dams, barrages and open channels are designed generally to have upstream skin plate & adequate (at least 0.3m) freeboard between the top of gates and the maximum water level. These provisions could reduce significantly, the maintenance requirement and the damages to the downstream structural members of gate.

The parts of high head gates should be stress relieved to resist the dynamic and fatigue loading resulting from vibrations. Stainless steel cladding of bottom lip of gate will be advantageous to resist cavitations & abrasion under high velocity flow.

The vertical lift gate (Fig-6) should have a sloping bottom lip (generally 45^0 for high head) for ensuring a reduced down pull forces and eliminating unstable flow reattachment in order to have vibration free operation of the gated installations especially during small openings when the gate is subject to be worst affected.

The high head gated installations (water head exceeding 30 m) invariably suffer from cavitations and vibration problems resulting from either due to inappropriate design of gate slots or due to high velocity jet flow caused by serious water leakage. Therefore the gate slot in such case should be appropriately designed. This will reduce cavitations/ vibration problems significantly. Apart from that the bottom lip of the gate should be appropriately designed and an air vent should be provided just downstream of the gate slot for facilitating proper ventilation (i.e. air supply) to the waterway. However, it should be ensured that the inlets of the air vents should be located in such a manner that the high water levels, waves, debris, etc., do not interrupt the aeration.

Other factors, affecting the cavitations phenomena are roughness of surfaces, distance between two adjacent gate slots and entrance shape of piers/ conduit. Therefore, these should be provided suitably based on the result obtained from the proper model test particularly in case of the high head gates.

In case of high head outlets, the bonnet type gated installation is necessary for satisfactory performance. In a bonnet type arrangement, the gate is withdrawn into embedded steel bonnet, when fully opened. It is desirable to design the top 300mm of the embedded bonnets to withstand internal hydrostatic loads and the imposed hoist loads without the aid of surrounding concrete. The bonnet covers should be invariably bolted to bonnets or gate frames, so that the water load on the bonnet covers is distributed to the concrete surrounding through the bonnets and gate frames. Direct transfer of load from bonnet covers to the top concrete lift should be avoided for safety.

The high head intake gates for power tunnels and bottom outlets should invariably provided with shaft type arrangement (Fig-7). However, in such cases it must be ensured that gate shafts should never be allowed to be submerged under water otherwise they may be damaged due to the vertical flow.

The rubber seals are provided invariably for ensuring the water-tightness of gates (Fig-8 & 9). Thus the seal should be designed in such a way that it should not cause any flow irregularities. This is must for having safe and satisfactory operation of the gates. At the same time, the seal assembly should be simple and should allow for easy installation and removal. The seal material and its mounting must be capable of transferring the initial stress (due to interference) to the seal. The seal fixture must allow the seal to deform/bend under the water pressure and allow easy adjustment to take care of reasonable inaccuracies in sealing surfaces during fabrication and erection.

For the vertical lift gates under extremely high head condition, the metal sealing surfaces have been found to be more suitable. However, such sealing require high degree of accuracy during manufacturing and erection.

For radial gates having top seal, it is important to design the top seals in a proper manner to avoid undesirable water jets during partial gate operation. It is the usual practice to provide an anti-jet seal fixed on the embedded frame at the top of the embedded frame, in addition to the standard rubber seal fixed to the gate. The standard seal ensures water tightness when the gate is fully closed, whereas the anti-jet seal fixed to the embedded frame make reasonable watertight contact with the gate at partial gate openings. It is often preferable to provide a stainless steel skin plate suitably machined on the upstream side to provide a smooth surface for anti-jet seal contact.

3.0 FABRICATION AND ERECTION STAGE

Consideration of reliability and minimum maintenance is an important aspect for a gate. Use modern and reliable components for gates like maintenance free self lubricating bearings, fluorocarbon clad seals, stainless steels of different grades etc. These items not only provide long term maintenance free service, but also permit very high load carrying capacities.

The selection of suitable bearing surfaces, especially for underwater applications and trunnion bearings of radial gate plays a significant role. Self-lubricating type bearings for gated installations offers satisfactory performance over long period of time. One of the most important requirements in design of a radial gate is to ensure the satisfactory performance of its trunnion bearings even with a reasonable misalignment. Misalignment is likely to occur during erection, or during operation as a result of malfunctioning of gate, due to seismic/ flood loading, due to deflection of piers when an adjacent bay is dewatered.

Similarly, the wheel type vertical lift gates provided with self-lubricating bronze bearing & stainless steel pin are always preferable. Designs of various components of the fixed wheel gates for high head also require careful attention, especially during the analysis of stresses in the wheels, tracks, and track beams, which carry heavy concentrated loads. Reliable bearing installations having a significantly lower coefficient of friction than the self-lubricating bronze bearings for the underwater uses can be engineered, but this type of arrangement relies completely on prevention of ingress of water /silt. Therefore, crucial factor for such applications is the sealing arrangement, because breakdown of the seals are likely to take place and cannot be expected to last longer.

The elastomeric side and top seals, plain or PTFE-faced are satisfactorily used for achieving water tightness of gated installation exposed to water head ranging from low to moderately higher head. Rubber seals with PTFE cladding and metal sealing for gates operating higher heads are preferable. The top and side seals should preferably be effective throughout the travel of the gate to avoid undesirable water jets and consequent problems especially in case of high head regulating gates.

The seal material must have high tensile strength, high resistance against tear/abrasion, low water absorption, adequate resistance to aging apart from low coefficient of friction. For low head application, plain rubber seal can be adopted, whereas the composite rubber seals with PTFE cladding having low friction coefficient is preferred for moderate to high head applications. The PTFE coating is less resistant to abrasion than the plain rubber, it is therefore obvious that any contamination of the water by suspended abrasive material will have an influence on the erosion of the seals and thereby its life.

By using two rows of top seal fixed on the gate leaf (in case of top seal radial gate one fixed to gate leaf and other fixed on breast wall) help to control the water leakage as well as safe guard against the damages resulting from the emerging water jets from the sealing plane. Therefore, such measures could be very effective in smooth functioning of gated installations.

The regions where cavitations can occur should generally be provided with steel lining, in some case even lined with stainless steel or high strength/hardened alloy steel.

Steel lining of conduit is required for the portion in between maintenance and regulating gate in addition to some distance u/s of maintenance gate and d/s of regulating gates for installations subjected to water head in the range of 30-40m or more. However, complete lining of conduit is desirable for the portion of conduit passing through the dam or gate operating under a head of 100m or more. The conduit area at the gate location is generally kept somewhat lesser than the u/s conduit area to ensure positive water pressures for the fully opened position of the gate.

Hydraulic Hoist are more versatile, smooth and dependable in so far as performance & reliability is concerned, therefore, these can be used in place of Rope drum and screw type of hoists for the operation of gates as far as possible. Overall design of the gates should be sturdy and should provide for adequate rigidity. It is not advisable to be very economical in respect of provisions for gates and hoisting equipment. It may happen sometime that due to some of the unforeseen & unpredictable forces or circumstances, economical provisions may result in unsatisfactory performance or even failure of gate/its operating mechanisms.

4.0 OPERATION STAGE

Hydraulic Hoists (Oil hydraulics) is frequently used now days to operate gates. The hydraulic fluid has to be environmentally compatible.

There should be a suitable provision for arresting the gate and stopping delivery of hydraulic fluid in the event of the burst of a flexible hose or a pipe have to be incorporated.

For smaller span of waterway single cylinder hydraulic hoist while for larger span waterway hydraulic hoist having twin cylinder should generally be adopted.

However, when lifting the gate from two points for twin cylinder arrangement, it is essential to synchronize the movement of piston of the cylinders to avoid distortion/ malfunctioning of gated structure. The development of accurate electronic piston rod position sensors built into the cylinder head has simplified synchronization and therefore has increased reliability of hydraulic hoists.

From reliability considerations, an oil-hydraulic installation also offers the possibility of automatically interchanging the power packs so that one can act as a standby to another by incorporating a few additional directional control valves. Moreover, in order to avoid/ reduce the vibration problems, it is required that the high head gates be rigidly supported during their operation. Rigidity of supports is also required in case the gate is not capable of self-closing i.e. gravity closure by its own weight. Requirement of rigidity of suspension for high head gates can be fulfilled either by using a screw hoist (limited to small size gate) or by hydraulic hoists more often.

The hydraulic hoist can also meet satisfactorily the important operational requirement of controlled rate of closure to avoid or limit water hammer and also to slowdown further for the final 5 to 10% of travel to avoid damage to the bottom of gate/sill in case of high head installation. Moreover, the rate of opening can also be critical, because it can initiate a reflected wave.

Similarly, if hydraulic hoist is used to operate the radial gates, the hydraulic cylinder must be properly hinged to ensure smooth gate operation without undue stresses in the stems and lifting mechanism.

The crucial & important load components, concerning design of hoisting arrangement for high head vertical lift gates, are the hydrodynamic (down pull/uplift) forces and the sliding/rolling friction. These two forces can vary appreciably for different installations and the problem is further compounded because of their variations throughout the length of the travel of the gate. In addition, the coefficient of friction may even vary from time to time, because of its dependency on state of sliding/rolling surfaces and lubrication provided. Model studies can be helpful for satisfactory assessment of hydrodynamic forces, but the accurate assessment of frictional forces is quite difficult in real life situations. Therefore, guaranteeing a satisfactory performance of gated installations in such circumstances is really a challenge. However, a conservative estimate of frictional force (sliding/ rolling, seals, guide etc.) is, therefore, imperative in design of hoist for satisfactory performance. In case of self-closing gate, it must be ensured that the submerged weight of the gates always exceeds the opposing forces during the closure operation by at least 20 percent.

As the coefficient of friction depends on the deposition of a lubricant film on the mating surface, the selection of the appropriate values of the same is difficult. Also, the bearing surfaces in the gate slots are subject to high velocity flow and eddy circulation. Such forces can remove the lubricants and thus may increase the frictional coefficient; therefore, provisions for forcing grease between the gate and the gate seats by suitable means to prevent such eventuality are must to ensure smooth operation.

In addition to such provisions, conservative estimate of required hoist capacity is made by assuming a higher value of sliding friction coefficient between the sliding bearing plate and embedded load bearing track plates, which are generally of stainless steel and bronze respectively.

The Rule curve for proper operation of a gate must be developed and should be followed accordingly. Further, it should be reviewed from time to time whenever required and revised accordingly.

5.0 MAINTENANCE STAGE

It is not sufficient only to provide properly designed gates and their operating equipment while the dam is constructed but it is also essential to effectively maintain the installations. Instances are reported in India as well as all over the world, in which failures of gates have occurred due to poor Maintenance of the equipment as stated above. Proper Maintenance of the installed equipment can be assured by providing a qualified and dedicated team of engineers and operators for attending to the maintenance needs from time to time, adequate financial allocation for all the maintenance related issues. The norms for proper maintenance and operation as stated under different BIS Codes and operating manuals are to be followed. Post installation inspection and maintenance is crucial for the proper functioning of the hydro-mechanical equipment. It also affects the performance of the civil structure and eventually success or failure of the whole project. It is very much important that the inspection procedure & operating manual are based on sound practices and as per the design criteria adopted. The inspection responsibilities are required to be delegated to senior and experienced group or groups of project personnel who are well versed with the activities involved in this operation. In the past, a number of cases of distress of hydro-mechanical equipment have been reported e.g. Failure of Singur Dam Radial Gates (A.P.) (Fig-10), Failure of Radial gate of Mohini Pickup Weir (MP), Malfunctioning of Intake Gates of Mahi Project (Rajasthan) (Fig-11), Failure of Hoist of Srisailam Radial Gates (A.P.), etc.

For ensuring satisfactory performance of gated installations, it is desirable to have operation and maintenance (O&M) logbooks for all hydro-mechanical equipments and have adequate spare parts such as Hoist Motor, wire ropes, bearings, oil seals, limit switches, rubber seals, etc. All gates to the extent possible should be operated every year especially before the onset of monsoon. All equipment should be cleaned and lubricated at least prior to onset of monsoon. Debris and ice build-up around gates and wire ropes should be removed. Repair damaged painted surfaces at least once in five years. Hoisting wire ropes and roller-chains should be regularly lubricated to minimize corrosion and ensure smooth hoist operation. The gear teeth should be checked for proper alignment, wear, excessive backlash, cleanliness, corrosion. Housing and mountings of Speed Reducers and their lubricant level should be checked. Mounting fasteners of machinery parts should be checked for tightness and corrosion. Shafts & couplings should be checked for cracks, twisting, strain and misalignment. Bearing housings, pedestals, etc should be checked for cracks and misalignment. Check trunnion bearings for excessive wear, lubrication, lateral slip and loose or missing fasteners. All braking devices should be checked for proper setting of braking torque and complete release. Check for proper ventilation of hoist Motors, unusual noise or odor if any from internal wiring's insulation. Check all components of Hydraulic Systems: i.e. valves, piping/tubing, switches, pumps, cylinders, hydraulic power units, etc. for signs of leakage and for proper operation. Inspect supporting steel members/concrete. Check that all drain holes are clear that there is no standing water. Inspect wire rope, drums, sheaves, etc.

For safety assurance of gated installations, it is required that Gate Leaf is inspected for deterioration of coatings and loss of metal if any. Trunnion and arms/bracing of a radial gate are crucial which must be inspected (Fig-12) for any deformed elements and/or members (i.e. beams, flanges, webs etc.). Welded joints should be checked for any discontinuity, loss of weld metal, cracks, corrosion etc. Ensure that drain holes exist and are open on various structural members (Fig-13). Contact surfaces of Wheels, Rollers and Tracks should be checked for corrosion as pitting can cause increased rolling friction. Also check that all wheels and rollers rotate freely. Check condition of gate seals for any damage. Check that embedded sealing surfaces are free of corrosion, pitting, scratches, etc. Check that seal mounting components and clamp bars are in place and that fasteners are tight.

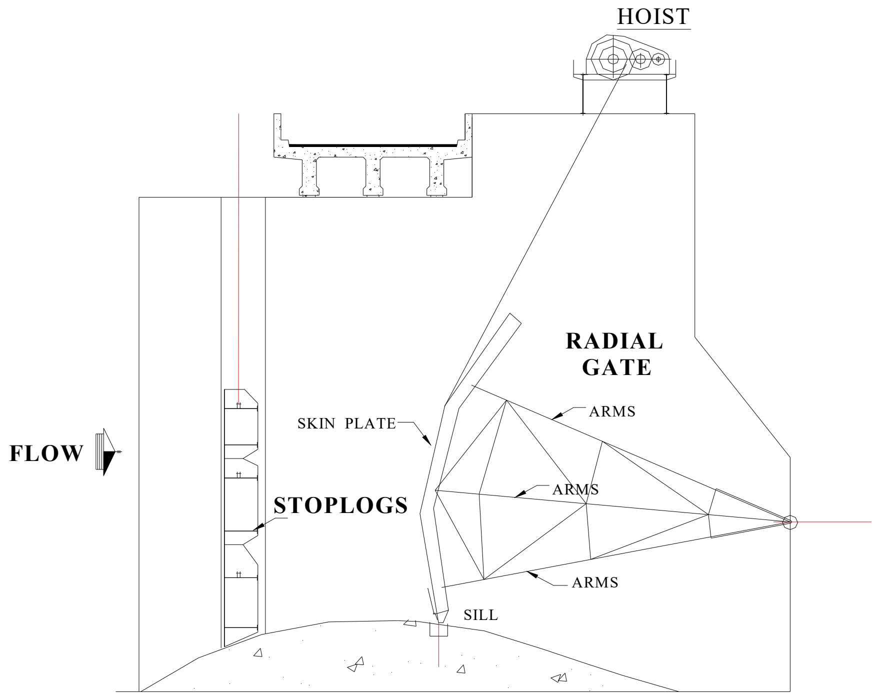
6.0 OTHER FACTORS

In the design of spillway gate for a dam/barrage, to a lesser extent river outlets & other low level sluice gates, reliable mechanical and electrical installations should take into account the possibility of common cause failures, such as seismic disturbance, extreme floods apart from fire, explosion, collision, and of course possible organizational, management, communication and forecasting weaknesses. The trunnions of the radial gates should be protected against corrosion and debris.

The mechanical and electrical considerations of earthquakes comprise precautions such as the rigid support for light fittings and long hydraulic hoist cylinders, where pipe couplings, gate position indicators, etc., can be affected. During a severe earthquake, electrical contactors/ main switches can be inadvertently operated and the operating cranes are liable to be derailed. The above factors may result in malfunctioning of gated installations or render them inoperative.

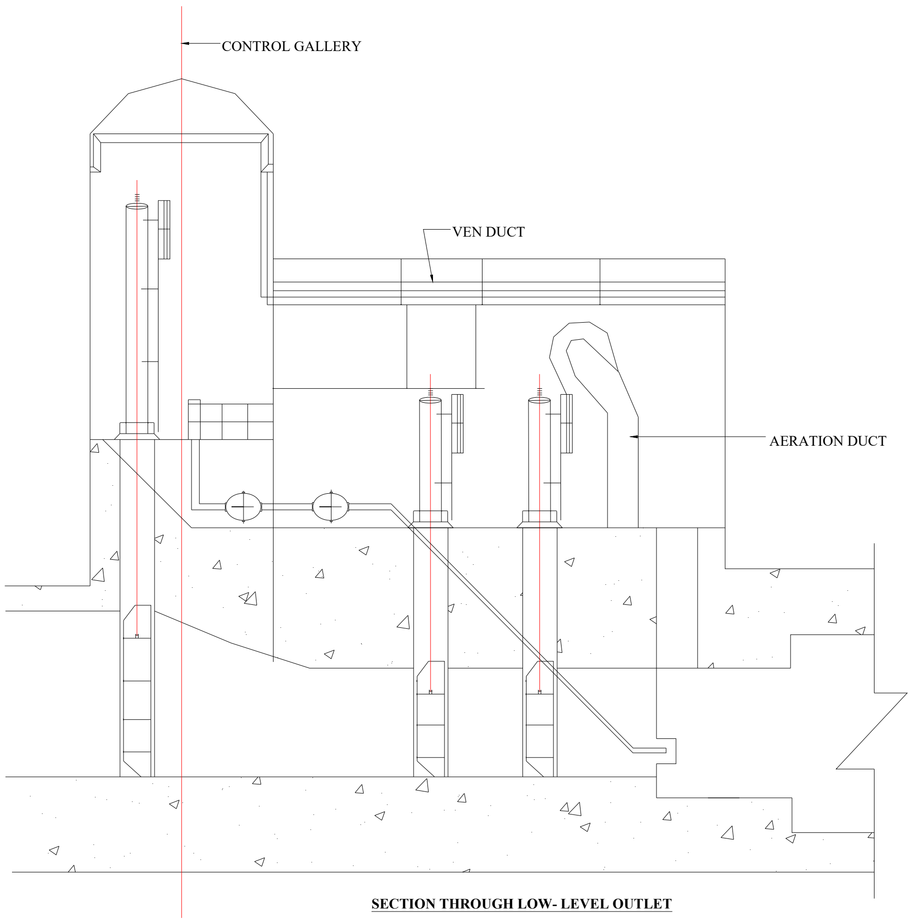
An essential requirement to avoid common cause failures is to provide at least two opening for outlets and spillways and to separate their operating mechanism physically. For the safety of structures, dependence in regards to operation of gates by highly elaborate devices or computerized electronic equipment can be provided such that the system has proper feedback controls. The use of partly computerized system for flood routing may lead to malfunctioning of the spillway gates equipment by opening them suddenly, or it would equally well happen that when required gate may not operate, and therefore the dam is likely to be over-topped.

There is a considerable preponderance of failures caused by overtopping, especially in the case of embankment dams. Overtopping has generally occurred because the gate operator failed to react to the arrival of a flood in time, or because of power failure at the critical moment and thereby the gates could not be opened for passing the flood. Power failures are common in the area in stormy weather, and it is in just such circumstances that power failure to the gate operating motors would be most likely to happen at the time when it would be most important for the gates to open. Therefore, safety can be ascertained by such designs, which would facilitate the opening of the gate without fail at the right time, and by the right amount, and totally without dependence on power or operator.



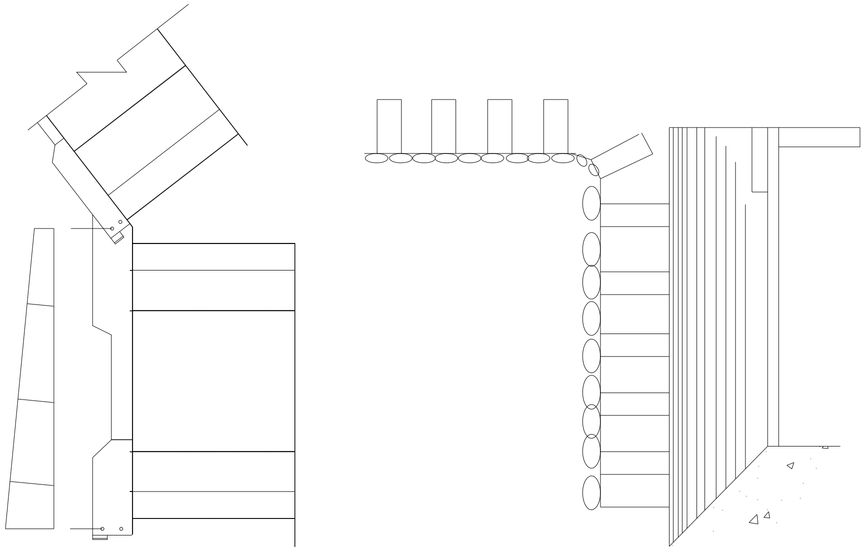
SPILLWAY RADIAL GATE & STOPLOGS

Fig-1



SECTION THROUGH LOW- LEVEL OUTLET

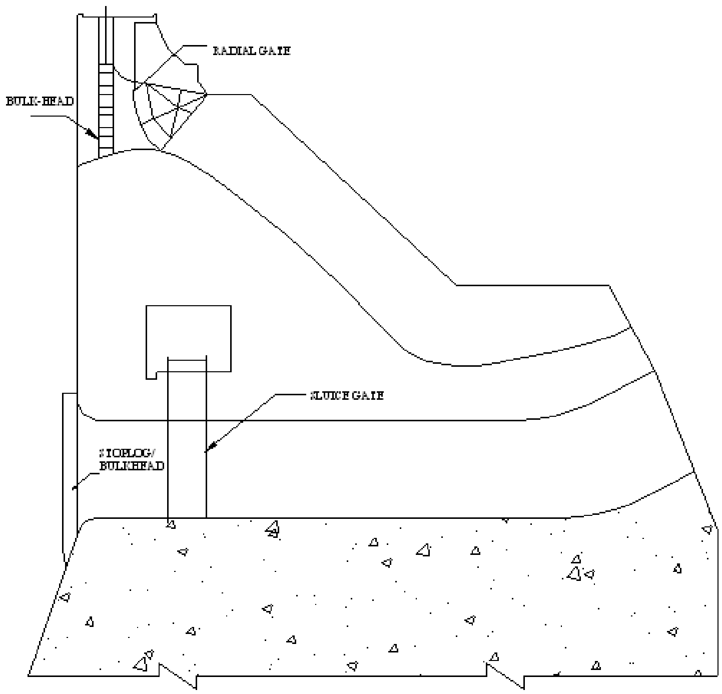
Fig-2



SECTION THROUGH BULKHEAD

BULKHHEAD INSTALLATION

Fig-3



SECTION THROUGH SLUICE AND SPILLWAY

Fig-4

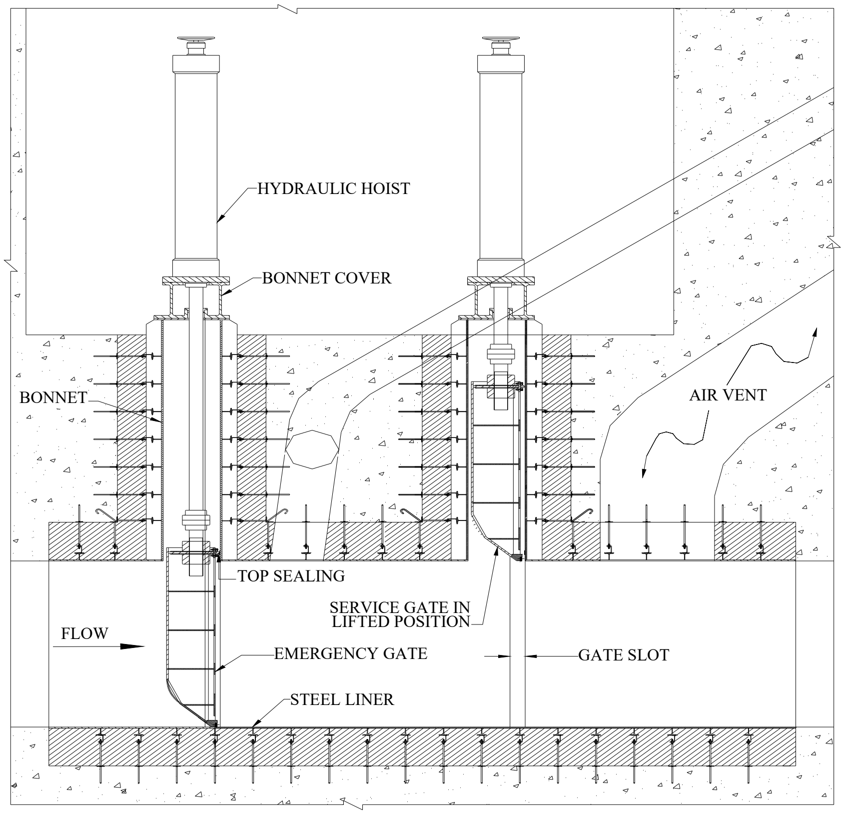
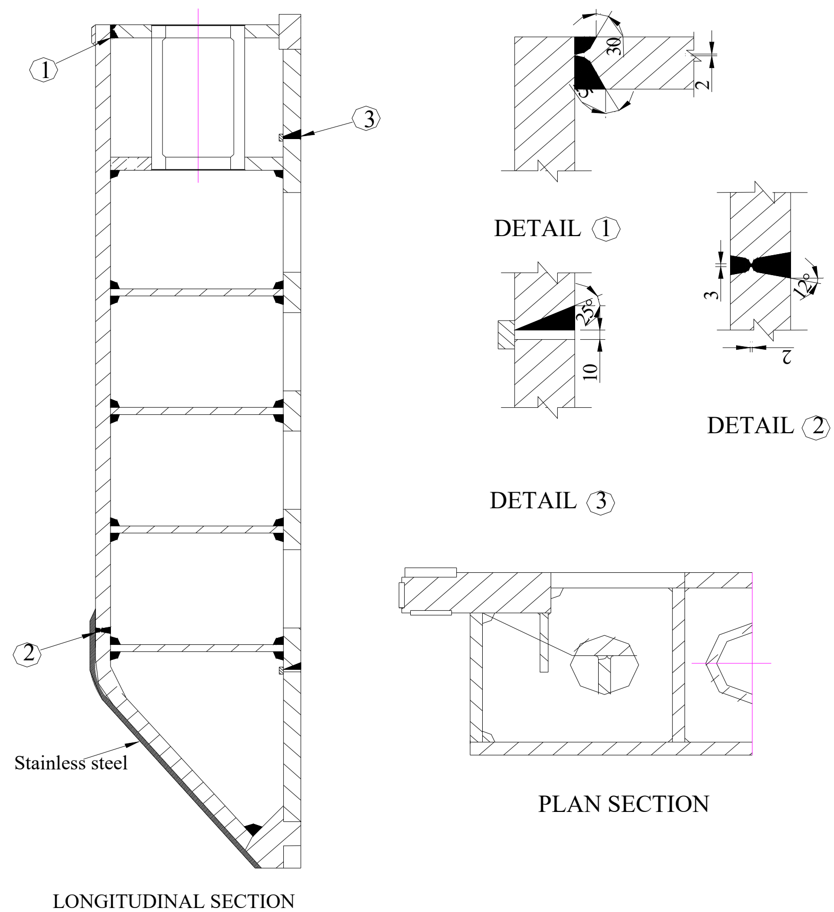


Fig-5: VERTICAL LIFT SLIDE GATES IN TANDEM



HIGH HEAD GATE CONSTRUCTION DETAILS

Fig-6

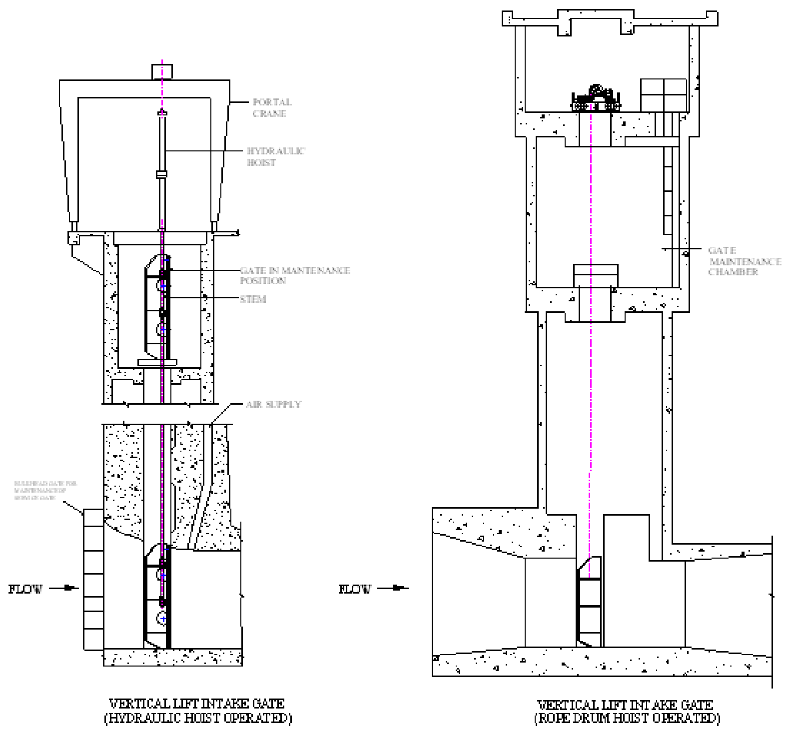
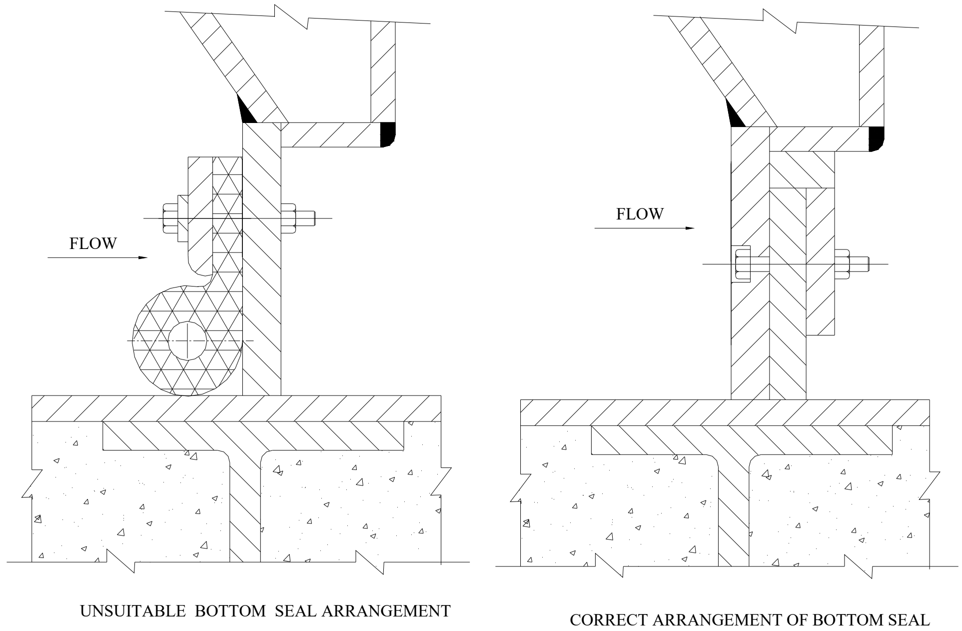

PORTAL CRANE
HYDRAULIC HOIST
GATE IN MAINTENANCE POSITION
STEM
AIR SUPPLY
FLOW
FLOW
GATE MAINTENANCE CHAMBER
VERTICAL LIFT INTAKE GATE
(HYDRAULIC HOIST OPERATED)
VERTICAL LIFT INTAKE GATE
(ROPE DRUM HOIST OPERATED)

Fig-7



FLOW
UNSUITABLE BOTTOM SEAL ARRANGEMENT
FLOW
CORRECT ARRANGEMENT OF BOTTOM SEAL

Fig-8

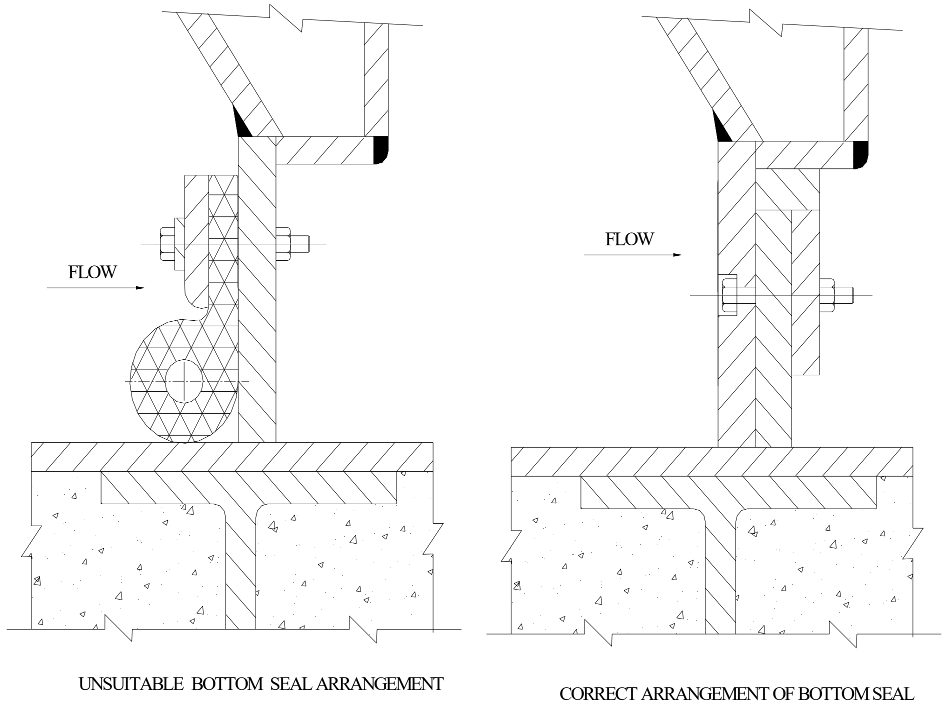

FLOW
UNSUITABLE BOTTOM SEAL ARRANGEMENT
FLOW
CORRECT ARRANGEMENT OF BOTTOM SEAL

Fig-8

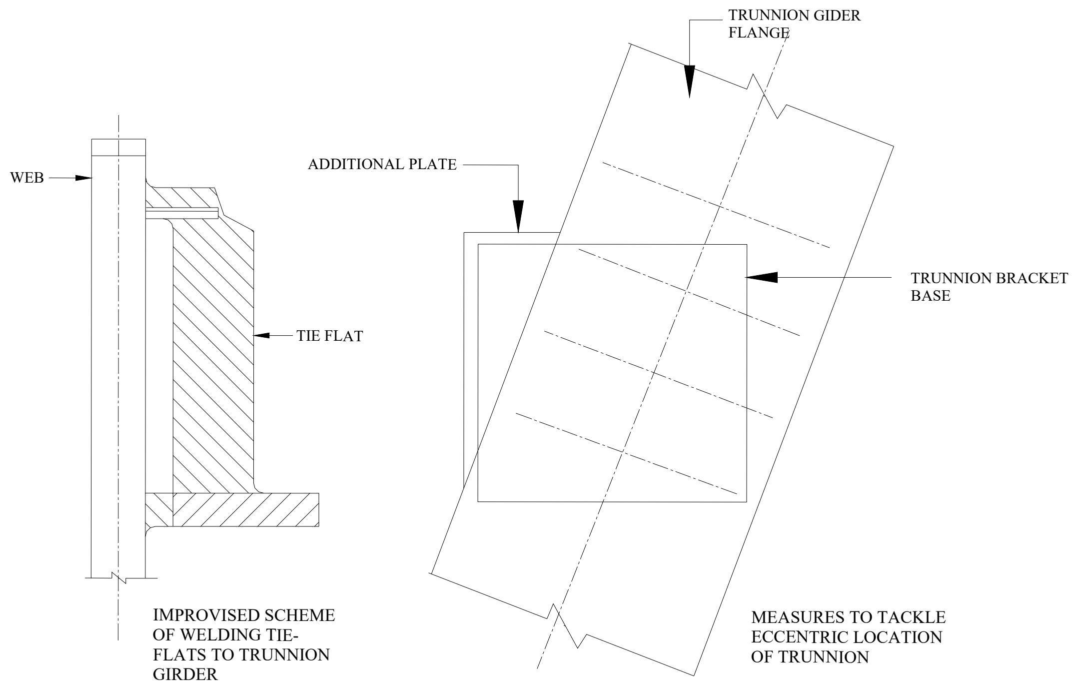
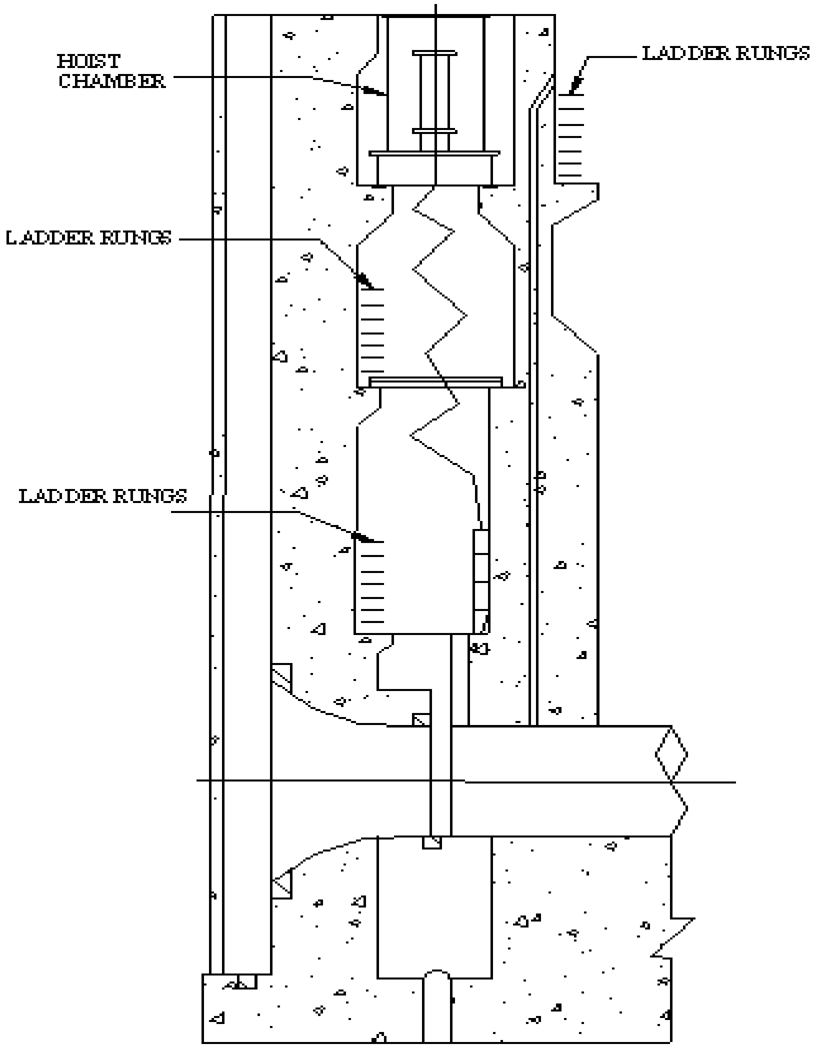
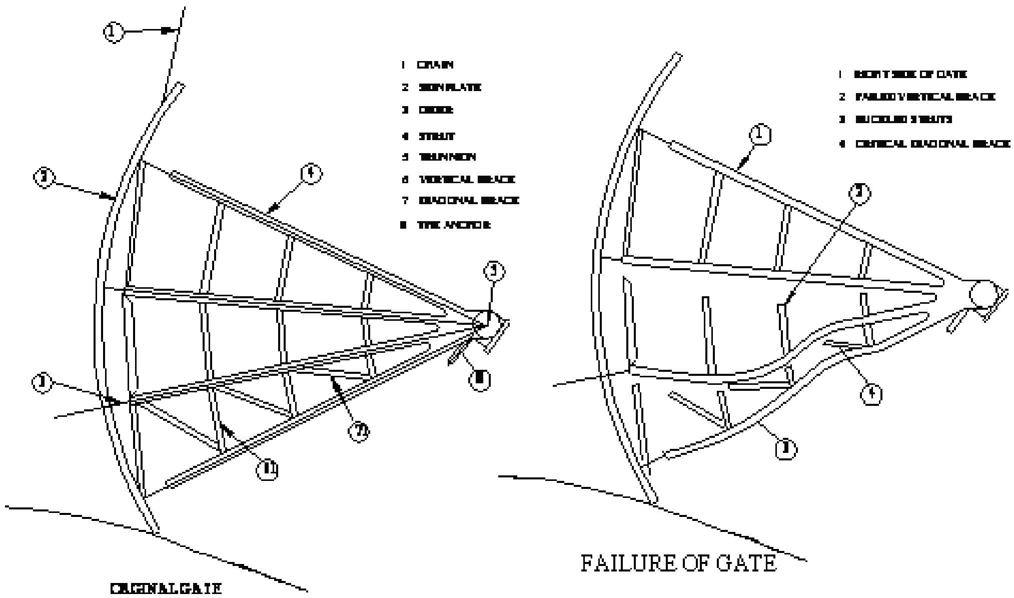

TRUNNION GIDER FLANGE
ADDITIONAL PLATE
WEB
TIE FLAT
TRUNNION BRACKET BASE
IMPROVISED SCHEME OF WELDING TIE-FLATS TO TRUNNION GIRDER
MEASURES TO TACKLE ECCENTRIC LOCATION OF TRUNNION

Fig-10



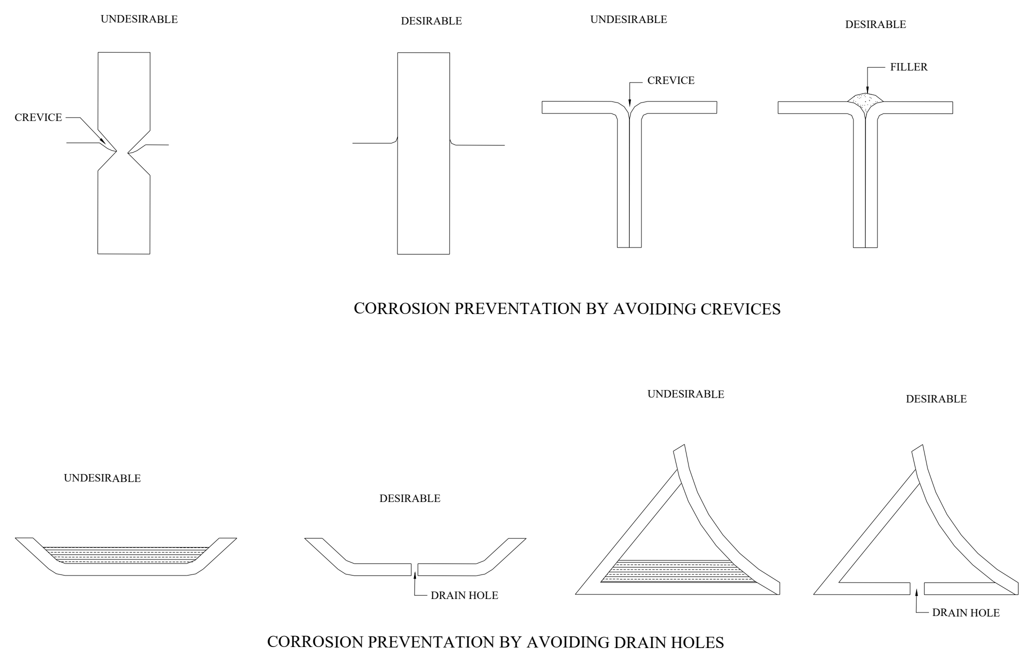
CATAPULTING OF SERVICE GATE

Fig-11



2 SKIN PLATE
4 STRUT
5 TRUNNION
6 VERTICAL BRACE
7 DIAGONAL BRACE
8 TIE ANCHOR
ORIGINAL GATE
1 RIGHT SIDE OF GATE
2 FAILED VERTICAL BRACE
3 BUCKLED STRUTS
4 CRITICAL DIAGONAL BRACE
FAILURE OF GATE

Fig-12



UNDESIRABLE
DESIRABLE
UNDESIRABLE
DESIRABLE
CREVICE
CREVICE
FILLER
CORROSION PREVENTATION BY AVOIDING CREVICES
UNDESIRABLE
DESIRABLE
UNDESIRABLE
DESIRABLE
DRAIN HOLE
DRAIN HOLE
CORROSION PREVENTATION BY AVOIDING DRAIN HOLES

Fig- 13

Chapter 5

Seismic Parameter Assessment and Analysis For Safety Evaluation Of Dams

1. INTRODUCTION

For earthquake resistant design of important engineered structures like dams, nuclear power plants, and long span bridges etc., it is necessary to estimate the site specific design ground motion. There are two basic approaches for developing the site specific design ground motions that are commonly used in practice i.e Deterministic Seismic Hazard Analysis (DSHA) and Probabilistic Seismic Hazard Analysis (PSHA). The basic input requirements for both the approaches are the same which includes the data on past seismicity, geological and tectonic features, knowledge on site soil condition, and attenuation characteristics of the strong motion parameters etc. for quantifying the seismic hazard. The DSHA proposes the design of a structure for the maximum earthquake that is expected to produce most severe ground motion at the site of interest. Seismic hazard may be analyzed probabilistically in PSHA, in which uncertainties in earthquake size, location, and time of occurrence are explicitly considered. Ground motion relations in terms of peak ground acceleration or velocity or response spectral ordinates of specific periods of vibration is common practice in engineering applications. To estimate the site-specific design response spectrum one has to perform a seismic hazard analysis for a project site of interest.

The characteristics (intensity, duration etc.) of seismic ground vibrations expected at any location depends upon the magnitude of earthquake, its depth of focus, distance from the epicenter, characteristics of the path through which the seismic waves travel, and the soil strata on which the structure stands. The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal. The response of a structure to ground vibrations is a function of the nature of foundation soil; materials, form, size and mode of construction of structures; and the duration and characteristics of ground motion. The common practice in earthquake engineering applications is to first specify the design ground motion in terms of a smoothed target response spectrum and synthesize the design acceleration time-histories to be compatible to this spectrum. The Bureau of Indian Standards (BIS) code of practice for earthquake resistant design of structures (IS: 1893 (Part-1) 2002) provides the response spectra for different types of site soil conditions, which are of fixed shape and differ for different zones only in amplitudes. However, depending upon the location of various active tectonic features and the spatial distribution of the past earthquakes, the site-specific response spectrum will differ significantly from site to site within a broad seismic zone. It is therefore necessary to have site-specific studies to evaluate the seismic design parameters for all-important projects. Nevertheless, the codal provisions are very useful to provide necessary insurance against sudden collapse of common type of structures.

In general, two different levels of ground motion i.e. maximum credible earthquake (MCE) and design basis earthquake (DBE) are considered for the design of important structures. Former advocates the largest possible earthquake that can reasonably be expected to be generated by specific seismic sources (SSZ) in a given seismo-tectonic framework is referred to as Maximum Credible Earthquake (MCE) for that SSZ. This is the largest event that could be expected to occur in the region under the presently known seismo-tectonic environment. Latter, on the other hand, infers the level of ground motion at the site of interest at which only minor and easily repairable damage is acceptable. DBE, would have a reasonable chance of occurrence during the life time of the structure, is also evaluated keeping in mind the degree of safety required. The ground motion associated with MCE is used only to check that a structure of interest does not collapse in a sudden and uncontrolled manner. If the damage of a project may lead to high-level of off-site hazard, all the systems and components necessary for ensuring the safety of the project are required to be functional during and after the occurrence of this earthquake. The design approach adopted in IS code is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur, and aims that structures withstand a major earthquake (MCE) without collapse.

2 DETERMINISTIC SEISMIC HAZARD ANALYSIS (DSHA)

In the deterministic approach, the design ground motion is estimated for a fixed earthquake magnitude and source-to-site distance combination. Although there is no generally accepted DSHA approach for all parts of the world and all application areas (e.g.; design of dams, nuclear power plants or ordinary structures), in its most commonly used forms, it intends to obtain the most severe ground motion by maximizing the magnitude and minimizing the distance (Reiter, 1990; Krinitzsky, 1995; Anderson, 1997; Gupta, 2002). DSHA can be described as four step process (Reiter, 1990) as given below.

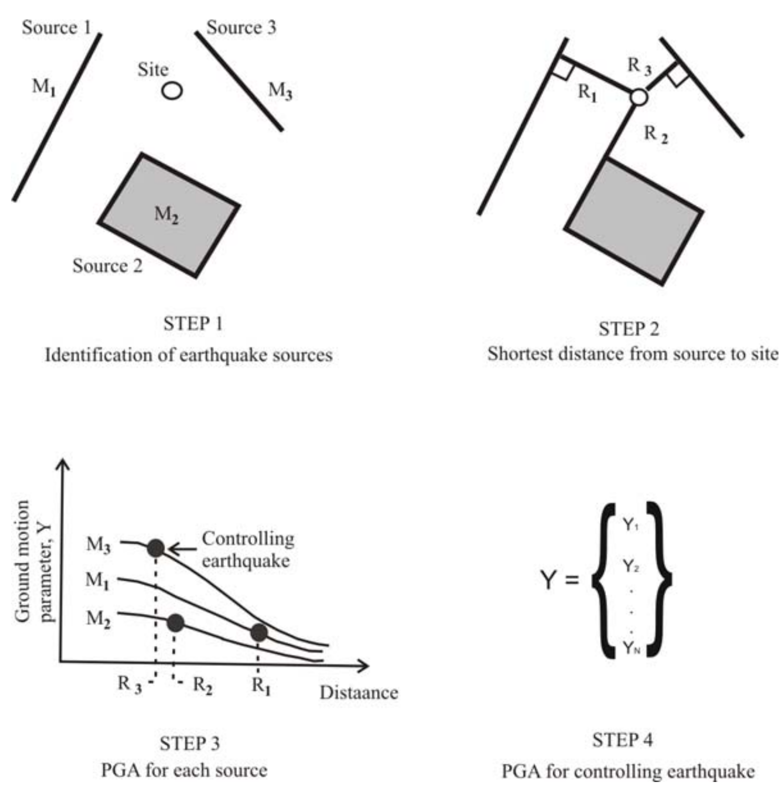


Fig.1. Four steps of deterministic seismic hazard analysis

1. Identification and characterization of all seismic sources.
2. Selection of source to site distance parameter.
3. Selection of controlling earthquake.
4. Defining the seismic hazard using controlling earthquake

To obtain the deterministic MCE level of ground motion, it is first necessary to identify and characterize the all earthquake sources capable of producing significant ground motion within an area of about 300 km radius around a project site. Then shortest distance from site to all earthquake sources would be considered and the controlling earthquake magnitude (M_{max}) for each of the seismic sources needs to be assessed. The M_{max} for each source is assumed to occur at a location, which makes the distance from the project site as the shortest possible. The ground motion is then predicted by using empirical attenuation relation or some other appropriate technique. Finally, seismic hazard is defined at the project site in terms of ground motion by controlling earthquake.

2.1 Identification of Seismic Sources

A seismic source represents the zone of the earth's crust with distinctly different characteristics of earthquake activity from those of the adjacent crust. The source zones in a region are identified on the basis of some sort of geological, geophysical, geodetic and seismotectonic uniformity. The seismic potential of a source zone has to be distinctly different from the other adjacent sources. As the earthquakes are caused by faulting, in an ideal situation all the seismic sources should be specific faults. However, due to lack of knowledge about all the faults and wide dispersion of the epicenters of past earthquakes in relation to the known faults, broad area sources encompassing several faults are used commonly in real practice. Such seismic sources may be associated with the geological structures like uplifts, rifts, folds and volcanoes, which release the tectonic stresses and localize the seismic activity. Another type of seismic source used in practical applications is the "tectonic province", which generally covers a large geographic area of diffused seismicity with no identifiable active faults or geological structures. The following three types of source zones can be considered sufficient for most practical applications.

Line Source: A nearly vertical fault with seismicity related closely to its surface trace can be idealized as a line source (not necessarily straight). This can be considered the simplest type of seismic source; the geometry of which is specified completely by the coordinates (latitudes and longitudes) of a series of points defining the fault trace, and the depth to the upper edge and width of the fault plane.

Dipping Plane Source: This can be considered the most realistic type of seismic source, in which the expected seismicity is associated closely with a dipping fault plane of length L and width W. The geometry of a dipping plane source can be specified by a series of coordinates (latitudes and longitudes) defining the surface projection of the upper edge of the dipping fault plane at depth D, and the dip angle φ.

Area Source: This is the most widely used type of source zone in practical applications. Large size area sources of diffused seismicity has to be used when exact knowledge of the causative faults is lacking and the observed seismicity is associated with a localizing geologic structure or a tectonic province. The area source can be of any arbitrary shape and located anywhere with respect to the site of interest. The geometry of an area source of arbitrary shape is defined by the coordinates (latitude and longitude) of its boundary.

2.2 Estimation of site to source distance

To obtain the deterministic target response spectra, it is necessary to assign the shortest possible source-to-site distance for each of the seismic sources. It is straightforward to define this distance for the line and dipping plane sources. For an area source, if the site lies outside the source, the distance is defined to the point on the boundary of the seismic source closest to the site. On the other hand, if the site happens to be within the source, it may be more rational to consider a non-zero minimum distance based on past seismicity or tectonic considerations. Also, for the source zone of the project site it may be necessary to consider the known tectonic features in the vicinity of the project site and their seismic potential, if the M_{max} for the entire source zone is considered to occur on some distant tectonic feature.

2.3 Estimation of controlling earthquake magnitude M_{max}

Estimation of the maximum potential for each seismogenic sources in terms of M_{max} is the most important aspect of DSHA method. However, due to limited earthquake history and incomplete understanding of the earthquake generating processes in most cases, this task is rendered very difficult and suffers from considerable subjective decisions. The various possible methods for estimating M_{max} are described briefly in the following:

(a) Addition of an Increment to the Largest Historical Earthquake

The recurrence interval for large earthquakes may generally be much longer than the available historical record. Thus, there is the possibility that the ``maximum" earthquake has not been recorded in a given seismic source. According to the Poisson statistics, there is only 63 % chance that an earthquake catalog contains an earthquake with recurrence interval equal to or greater than the length of the catalog. Hence, on average, in more than one third of the cases the largest shocks may be missing in the available catalogs. This will further be much less for earthquakes having recurrence periods much longer than the period of the earthquake history. The available paleoseismic studies indicate that the recurrence interval of M8+ earthquakes in Himalayas may be of the order of 500 - 1000 years (Sukhija et al., 2002). In the intraplate environment of Peninsular India, the recurrence periods of the largest earthquakes may be of the order a thousand years (Sukhija et al., 2006; Rajendran and Rajendran, 1999).

To account for the fact that an available catalog of recent earthquakes for a seismic source may not represent the largest possible event, magnitude units of 0.5 to 1.0 are commonly added to the largest historical earthquake to get an estimate of the maximum shock for a source zone. Higher increment is recommended for areas of lower seismicity. However, no increment be made, if it is considered that the reported maximum magnitude in the available short duration of catalogue itself is the maximum potential of the source. This approach should be viewed only as a means for reaching an appropriate engineering decision due to lack of scientific understanding, and not as a method as such.

(b) Use of Fault Rupture Parameters

For a fault specific seismic source, the maximum earthquake potential can also be evaluated from the estimates of fault rupture parameters like surface and sub-surface rupture lengths, l and $\hat{l}$, rupture width, w, and rupture area, A. Out of a large number of empirical relations between magnitude and various fault rupture parameters published by different investigators, the following relations based on worldwide data due to Wells and Coppersmith (1994) are the most widely used one:

$$M = \begin{cases} 5.08 + 1.16 \log l \pm 0.28 \\ 4.38 + 1.49 \log \hat{l} \pm 0.26 \\ 4.06 + 2.25 \log w \pm 0.41 \\ 4.07 + 0.98 \log A \pm 0.24 \end{cases}$$

(1)

It may be noted that the geologic conditions and the hypocentral depths have significant effect on the surface rupture length, *l,* and hence it cannot be considered a good representation of the source strength. On the other hand, the subsurface rupture length, $\hat{l}$, which can be estimated from the spatial distribution of aftershocks, is less subjected to such uncertainties. Thus, the subsurface rupture length regressions can be considered more appropriate for estimating magnitude for expected rupture along a fault. For very large earthquakes, the fault width reaches the maximum width of the seismogenic zone and the rupture continues to grow mainly through the fault length. Thus a relationship between magnitude and the rupture area (*A*) of the fault is physically and theoretically more meaningful. The maximum magnitude rupture area relationships are found to be more consistent and characterized by smaller variations compared to the rupture length regressions.

2.4 Estimation of Target Spectrum

Using all the M_{max} and their minimum distances, the target response spectra of horizontal and vertical components with damping ratio of 5% are obtained using suitable frequency-dependent attenuation relationship or some other simplified approximation. For the purpose of design, the largest of all the response spectra is selected in some suitable way as the deterministic target spectra, because none of the response spectra may be the largest over the entire frequency range.

The median estimate of the response spectrum has about 50% chance of being exceeded due to future earthquakes. Therefore, to take into account the effect of random scattering of the observed data around the median attenuation relationship, the MCE level of target response spectrum is commonly taken as the median plus one standard deviation estimate (84^{th} percentile). All the systems and components necessary for ensuring the safety of a dam (e.g., bottom outlet and/or spillway gates) are required to be functional during and after the occurrence of MCE level of ground motion. The notion of using the 84^{th} percentile values is based on the fact that if the consequences of failure are greater, the frequency of exceeding the design ground motion should be very low. In fact, it the recurrence of the selected M_{max} is rare (say, every 2000 years), the median response spectrum will have a return period of 4000 years, which may be considered adequate. On the other hand, if M_{max} recurs more frequently (say, every 500 years), the median plus one standard deviation ground motion having return period of 3570 years will be more appropriate. As the recurrence period of the maximum possible magnitude is not known in most cases, the deterministic method is generally unable to quantify the return period of the ground motion.

The DBE level of deterministic target response spectra of horizontal and vertical ground motion are commonly taken subjectively as a fraction (say, 1/2) of the corresponding MCE level of spectra. Alternatively, they may be taken as one standard deviation below the MCE level of spectra. The basis for DBE is in reality economical and a probabilistic approach is typically more suited to evaluate this level of ground motion, say 50% probability of not exceeding in 100 years. There should be no significant damage to a dam and the appurtenant structures, and equipments should remain functional and damage easily repairable during and after the occurrence of this level of ground motion.

2.4 Illustrative example of DSHA methodology

To illustrate the application of the foregoing DSHA methodology, Fig. 2 shows an area of 6º Lat × 6º Lon around a project site shown by solid triangle in seismically active Himalayan region. The tectonic features in the region of the site are shaped by the collision of the Indian plate with the Eurasian plate under the framework of plate tectonics (Molnar, 1988). With the continued movement of the Indian plate in NNE direction, large scale folding, crustal shortening and vertical crustal movements have taken place, forming the great Himalayas and other subsidiary faults in the region. The region which can be divided longitudinally into five major crustal formation zones identified from south to north as (i) outer zone of the fore-deep, (ii) inner zone of the fore-deep forming the Himalayan foot-hills (iii) Lesser Himalaya formed by superposition of a series of tectonic nappes and probably thrusted over the fore-deep, (iv) the High Himalaya and (v) the Indus-Tsangpo Suture zone. The first four of these zones are separated from each other by large thrust faults. The Main Frontal Thrust (MFT) runs at the boundary between the outer and the inner zones of the fore deep. The Main Boundary Thrust (MBT) separates the napped-folded complex of the Lesser Himalaya from the Himalayan foothills. A thick stratum of crystalline rock comprising the High Himalaya is thrusted over the metamorphosed deposits of the Lesser Himalaya along the MCT. The belt north of High Himalaya and bound by Indus-Tsangpo Suture is known as Tethys Himalaya, which consists of fossiliferrous sedimentary rocks.

The Main Frontal Thrust (MFT) runs at the boundary between the outer and the inner zones of the fore-deep. The Main Boundary Thrust (MBT) separates the napped-folded complex of the Lesser Himalaya (mainly composed of deposits of Proterozoic and Precambrian age) from the Himalayan foot-hills made up essentially of Tertiary and Quaternary sediments. A thick strata of crystalline rock comprising the High Himalaya is thrusted over the metamorphosed deposits of the Lesser Himalaya along the MCT. Thrust formation processes characteristic of Himalayan tectogenesis phase were mainly completed in the MCT zone by the end of Pliocene period (3 m.y.), and no distinct sign of their activity has been found in the recent time. The belt north of High Himalaya and bound by Indus-Tsangpo Suture is known as Tethys Himalaya, which consists of fossiliferrous sedimentary rocks. The trans-Himalayan zone includes the Kailas and Ladakh ranges and continues up to the Tibetan plateau.

In addition to the major structural discontinuities of the Himalayan region, there are a number of other faults/lineaments in the region. A number of transverse faults also exist within the Himalayan ranges as well as to the south in the Indo-Gangetic planes, which are associated with varying levels of seismicity. To get an idea about the association of past earthquakes with various tectonic features in the region, the correlation of the epicenters of past earthquakes superimposed on the major tectonic features is also shown in Fig. 2. A seismic source is defined as an individual fault or an area of diffused seismicity with distinctly different seismogenic potential in terms of the maximum magnitude as well as the occurrence rate of earthquakes in different magnitude ranges. Considering the spatial distribution and correlation of seismic activity with the tectonic features, five broad seismic source zones have been delineated in the study region as listed below:

1. Main Himalayan Thrusts (MHT) area of MCT, MBT, etc.
2. Kaurik fault system (KFS)
3. Himalayan Fold (HF) area between ISZ and Karakoram Fault.

4. Trans-Himalayan (TRH) area towards north of Karakoram Fault.
5. Aravalli Fold Belt (AFB) area in the Indo-Gangetic Plains.

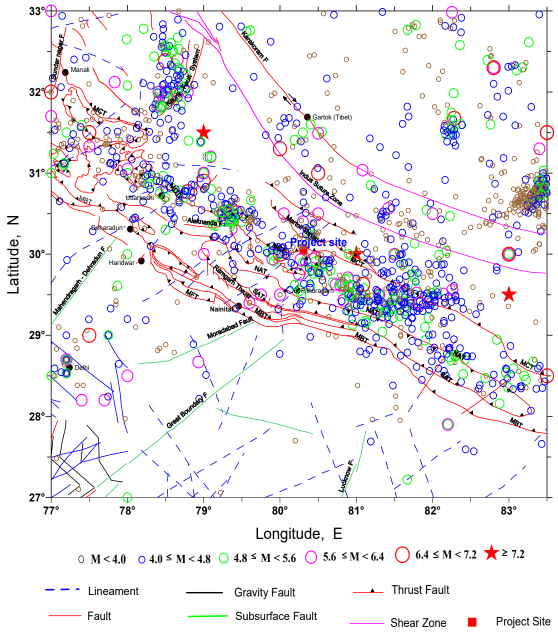


Fig.2. Correlation of the epicenters of past earthquakes with major tectonic features in the region of project site.

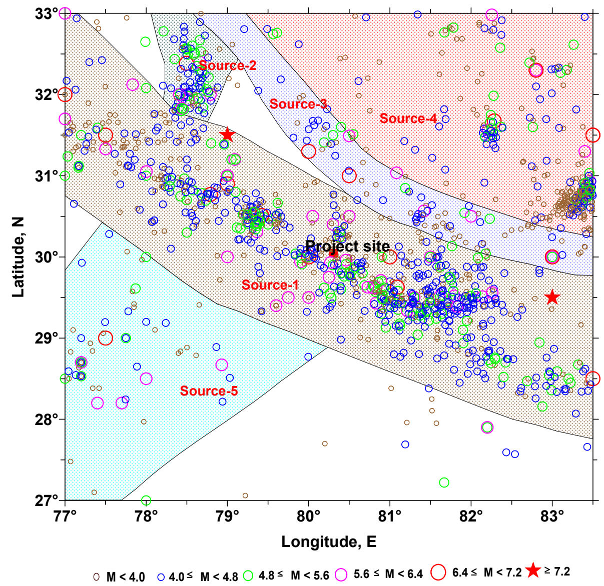


Fig.3. Seismic source zones in the region of project site along with the epicenters of available data on past earthquakes

These source zones are shown demarcated by thick solid lines in Fig. 3 along with the epicenters of the past earthquakes. There is a marked concentration of epicenters along the Kaurik Fault system (KFS), which has been therefore defined by a separate source (source-2). The other main sources are also characterized by some concentrations of epicenters, which have been suitably taken into account in estimating the ground motion by the probabilistic approach.

The magnitude of MCE for a tectonic feature or seismic source can be estimated by different methods, which range from the use of empirical relations in terms of the fault rupture parameters, actually recorded largest historical earthquake with or without some increment, extrapolation of magnitude recurrence relationship, and extreme-value statistics (Gupta, 2002). The maximum value of the length of a fault expected to rupture during a single earthquake can be used to estimate the magnitude of MCE (Wells and Coppersmith, 1994). However, the reliability of this method cannot be considered very good, because the fault rupture parameters for a future earthquake cannot be defined with confidence. It is observed from Fig.2 that the tectonic features closest to the site are the MCT, MBT, NAT and Martoli thrust (MT).

The project site is located closest to the MCT at an epicentral distance of 3.68 km. The largest magnitude closest to the MCT is that of the 1916 event which is 7.1, hence MCE magnitude of 7.5 is conservatively assigned to the MCT. A magnitude 7.0 has been assigned to the Martoli thrust. A magnitude of 6.5 is considered the potential of the North Almora Thrust. A magnitude of 8.0 is assigned to the MBT. For MBT and MCT the MCEs are assumed to occur at a closest distance to fault rupture plane from the site of the dam which is the distance from the site to the detachment surface. The magnitudes and the closest distances to fault rupture plane for the various tectonic features considered for the project site are listed below:

Sr. No.	Tectonic Feature	Type of Faulting	Magnitude of MCE	Closest distance from the site (Rrup) km
1.	Main Boundary Thrust (MBT)	Thrust	8.0	21
2.	Martoli Thrust (MT)	Thrust	7.0	19.4
3.	Main Central Thrust (MCT)	Thrust	7.5	31
4.	North Almora Thrust (NAT)	Thrust	6.5	51.6

The 5 % damped MCE level of horizontal and vertical response spectra for the various MCE magnitude and distance combinations have been evaluated using the frequency-dependent attenuation relation due to Abrahamson and Silva (1997) with a confidence level of 0.84, which corresponds to the mean plus one standard deviation values of the response spectra. The plot showing the comparison between the spectra for different tectonic features is shown in Fig.4. From the figure, it is discerned that the spectral amplitudes for MBT are the highest and hence taken to be the deterministic target spectra for horizontal component of ground motion.

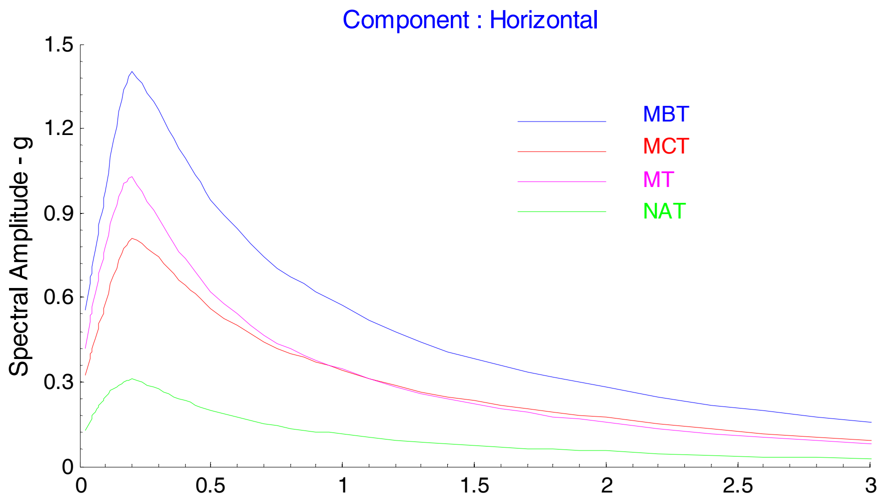


Fig.4. Comparison of the mean plus one standard deviation deterministic response spectra with damping ratio of 5 % for various MCE magnitudes and distance combinations.

3. PROBABILISTIC SEISMIC HAZARD ANALYSIS (PSHA)

By maximizing the magnitude and minimizing the distance, the deterministic approach intends to get an upper limit for the MCE level of ground motion, which may sometimes be unduly conservative. More rational lower levels of ground motion are thus required to be used in practical applications. In this regard, the ICOLD (2010) and CBIP (2007) suggests the use of Safety Evaluation Earthquake (SEE) or Maximum Design Earthquake (MDE) level of ground motion defined probabilistically with different recurrence periods, depending upon the risk class associated with the failure of a dam. For dams whose failure would present a great social hazard (i.e., extreme and high risk class) the MDE/ SEE level of ground motion is defined with return period of about 10,000 years. The return period for dams with moderate risk class may be 3,000 years and with low risk class as 1,000 years. The probabilistic seismic hazard analysis (PSHA) approach provides a powerful tool to obtain such estimates of the design ground motion.

The PSHA approach is based on an altogether different philosophy and differs from the DSHA approach in two major aspects. First, a single scenario earthquake in DSHA is not able to provide a true picture of the seismic hazard at a site, because different combinations of magnitude and distance contribute more significantly in different frequency bands. Small magnitude local earthquakes often dominate the high frequency spectral amplitudes, whereas large magnitude earthquakes at even very large distances may dominate the low frequency amplitudes. The PSHA approach takes into account the effect of all the earthquakes between specified lower and upper bound magnitudes distributed appropriately over the entire area around a project site to provide all expected combinations of magnitude and distance. Second, the DSHA defines the MCE level of target response spectra with 84th percentile level without any consideration for the recurrence period of the MCE magnitude. Thus the return period of the deterministic spectra remains undefined. The PSHA, on the other hand, considers the annual occurrence rate of all possible magnitude and distance combinations, along with all possible ground motion probability levels (a range of the number of standard deviations above or below the median). The probabilistic ground motion is then defined for a specified return period, such that it is not exceeded due to any of the expected earthquakes.

The early history and evolution of PSHA approach is described in a paper by McGuire (2007). Most of the developments in PSHA are based on the classical paper of Cornell (1968), who considered peak ground acceleration as the ground motion parameter. Cornell's original formulation has not considered the randomness in the ground motion attenuation relationship, which was shown to be a significant source of uncertainty in the results of the hazard analysis by DerKiurghian (1977). To have a uniformly conservative estimate of the hazard at all the frequencies, McGuire (1974, 1977) performed the PSHA for response spectrum amplitudes at different frequencies, with the randomness in spectral amplitudes considered by a lognormal distribution. Anderson and Trifunac (1977,1978) generalized the PSHA formulation by modeling the seismicity in more realistic way. Details on the currently used PSHA approach can be found in EPRI (1986), Reiter (1990), SSHAC (1997), US Army Corps of Engineers (1999), Gupta (2002), McGuire (2004), etc.

The basic PSHA approach is based on computing the following probability distribution of a specified measure of the ground motion amplitude (e.g., the acceleration response spectrum amplitude *SA(T)* at natural period *T)*:

$$P[SA(T)] = 1 - \exp\left\{ -Y \sum_{n} \sum_{j} \sum_{i} q_n[SA(T) \mid M_j, R_i] \cdot \upsilon_n(M_j, R_i) \right\}$$

(2)

In this expression, Y is the exposure time; $\upsilon_n(M_j, R_i)$ is the annual occurrence rate of earthquakes of magnitude M_j at distance R_i in the nth seismic source zones around the project site, and $q_n[SA(T) \mid M_j, R_i]$ is the probability of exceeding the spectral amplitude *SA(T)* due to magnitude and distance combination (M_j, R_i) in the nth source zone. The probability distribution of eqn. (2) can be used to estimate the spectral amplitudes with a desired probability of exceedance due to any of the earthquakes expected to occur anywhere in the region around the project site during a specified exposure period. The combination of the desired probability (P) and the specified exposure period (Y years) is equivalent to a return period T_R (years) = $-Y/\ln(1-P)$ for the occurrence of the estimated ground motion. The probabilistic ground motion can thus be defined for a desired return period by using suitable combination of exposure period Y and the probability of exceedance P. The PSHA can also be used to obtain the return period for a deterministic estimate to facilitate taking appropriate decisions about the design ground motion. The various steps involved in implementation of the PSHA approach are depicted in Fig. 5, and are summarized in the following:

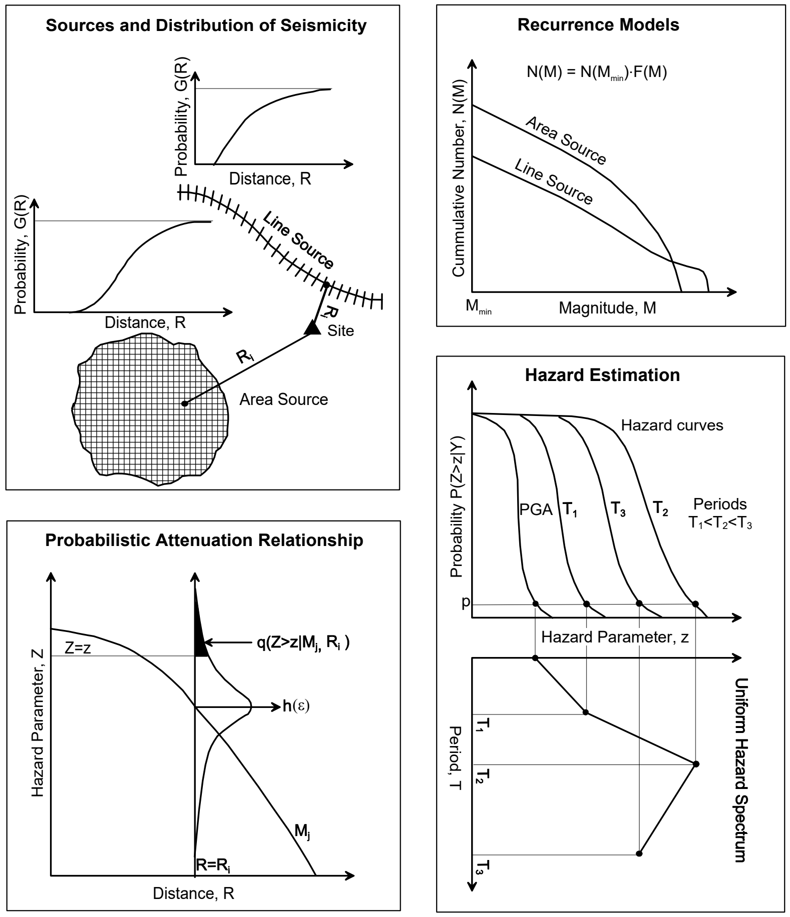


Fig. 5: Illustration of the basic elements of the Probabilistic Methodology

The first step in PSHA is identical to that in the DSHA approach, that is to identify and delineate all the seismic source zones within an area of 6° Lat × 6° Lon with the project site at the center. However, in PSHA approach, each of the sources is further divided into large number of small size elements, and the total expected seismicity in a source is distributed suitably among all the elements. The epicenters of all the expected earthquakes in an element are assumed to occur at its geometric center. The probability distribution function, $G(R)$, of a particular type of source-to-site distance is then defined using the distances to all the elements as illustrated in top left panel of Fig. 5.

In the second step, a suitable earthquake recurrence relationship with lower threshold and upper bound magnitudes is defined for each source zone using available past earthquake data. This is then used to obtain the occurrence rate $n(M_j)$ of earthquakes in magnitude interval $\left(M_j - \delta M_j, M_j + \delta M_j\right)$, which is then distributed among different distance intervals $\left(R_i - \delta R_i, R_i + \delta R_I\right)$, using the distance distribution function $G(R)$, to get the seismicity rate $\upsilon(M_j, R_i)$.

An attenuation relationship suitable for the region of interest is selected in step 3, which describes the response spectral amplitudes in terms of earthquake magnitude, source-to-site distance, and site geologic condition. Such a relation is able to provide the median estimate and the corresponding probability distribution of the residuals for specified earthquake magnitude M_j and source-to-site distance R_i, which can be used readily to estimate the probability $q[SA(T)\,|\,M_j, R_i]$ as illustrated in the bottom left panel of Fig. 5. A single attenuation relation may normally be applicable to all the source zones, but different relations may have to be used in some cases. For example, in the northeast India, if a site is affected simultaneously by shallow crustal and deep subduction zone earthquakes, those are required to be described by different attenuation relations.

By carrying out the summations over all the magnitudes and distances in all the source zones, probability distribution $P[SA(T)]$ of eqn. (2) is computed in the fourth and the final step. The plot of $P[SA(T)]$ versus *SA(T)* is termed as the hazard curve. A complete response spectrum can be obtained by computing the hazard curves for all the natural periods and estimating the spectral amplitude at each period with the same probability of exceedance during a specified exposure period, as shown in the bottom right panel of Fig. 5. Such a spectrum is known as "uniform hazard response spectrum". The MCE level of probabilistic response spectrum is commonly defined for 2% probability of exceedance in 50 years, which represents a return period of about 2500 years for the ground motion.

3.1 Illustrative example of PSHA methodology

To illustrate the application of probabilistic seismic hazard analysis (PSHA) approach, the same example is considered as that for the DSHA method, for which the various seismic source zones are shown in Fig.3. The available data on past earthquakes are used to fit the following Gutenberg-Richter's (1944) recurrence relationship for each of the above seismic sources:

$$\log N(M) = a - bM \tag{3}$$

where, *N(M)* is the annual occurrence rate of earthquakes with magnitude *M* or greater, and *a* and *b* are the constants estimated by regression analysis of the data. To estimate these constants, earthquake data have been converted into moment magnitudes using empirical conversion relations (Chung and Bernreuter, 1981), dependent events (aftershocks) have been removed (Gardner and Knopoff, 1974; Uhrhammer, 1986), and the completeness of different magnitude ranges is accounted using Stepp's (1972) method. The maximum likelihood method of Weichert (1980) has been used to obtain the constants *a* and *b* in the present study. The parameters *a* and *b* obtained for all the sources are as tabulated below.

Sr. No.	Seismic Source zone	a	b	Mmax
1.	Source-1	4.95	0.96	8.6
2.	Source -2	2.71	0.63	7.0
3.	Source -3	2.91	0.74	7.0
4.	Source -4	3.96	0.86	7.5
5.	Source -5	2.67	0.80	6.7

The exponentially decaying recurrence relation is then defined with a threshold magnitude $M_{min} = 5.0$ for each source. Typical example of the fitting of the recurrence relationship for Main Himalayan Thrust (MHT) Source zone is shown in Fig. 6.

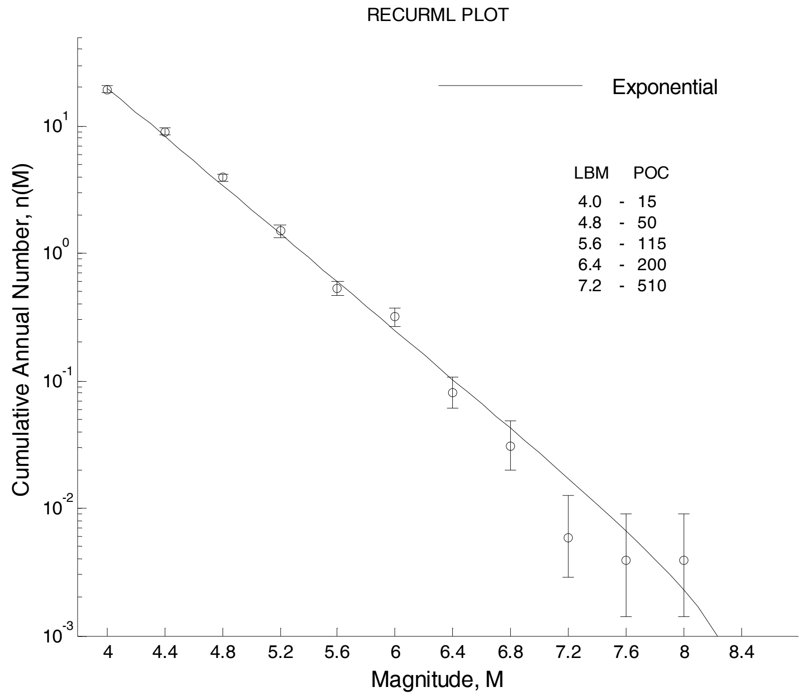


Fig.6. Fitting of exponential recurrence relation for MHT source zone.

From a knowledge of parameters *a* and *b* for a source, the relationship of eqn. (3) can be used to obtain the total number, $N(M_{min})$, of earthquakes above a specified threshold magnitude M_{min}, taken as 5.0 in the present study. For practical applications, it is however necessary to consider an upper bound magnitude M_{max} also. With upper and lower bound magnitudes, the expression for *N(M)* can be written as (Cornell and Van Marcke, 1969)

$$N(M) = N(M_{min}) \frac{exp(-\beta(M - M_{min})) - exp(-\beta(M_{max} - M_{min}))}{1 - exp(-\beta(M_{max} - M_{min}))}$$

(4)

Parameter β in this relation is related to the *b* value as $\beta = b \ln 10$. The relationship of eqn. (4) describes an exponential decay of *N(M)* with increase in *M* up to the maximum magnitude. It may therefore not be suitable to describe the behavior of very large magnitude earthquakes in some cases, where the largest earthquakes occur more periodically and frequently. In such cases, the characteristic earthquake model is found to describe the data better (Youngs and Coppersmith, 1985). The characteristic model is obtained by fitting the relationship of eqn. (4) to the observed data up to some much lower magnitude than M_{max}. In the range, $(M_{max} - \Delta M_c, M_{max})$ of the characteristic earthquake, a uniform occurrence rate is used with the probability density equal to that given by the distribution of eqn. (4) at a still lower magnitude, say one magnitude unit below $(M_{max} - \Delta M_c)$.

From the recurrence relationship for a source zone, the occurrence rate of earthquakes within a small magnitude range $(M_j - \delta M_j,\ M_j + \delta M_j)$ around magnitude M_j can be obtained as

$$n(M_j) = N(M_j - \delta M_j) - N(M_j + \delta M_j) \tag{5}$$

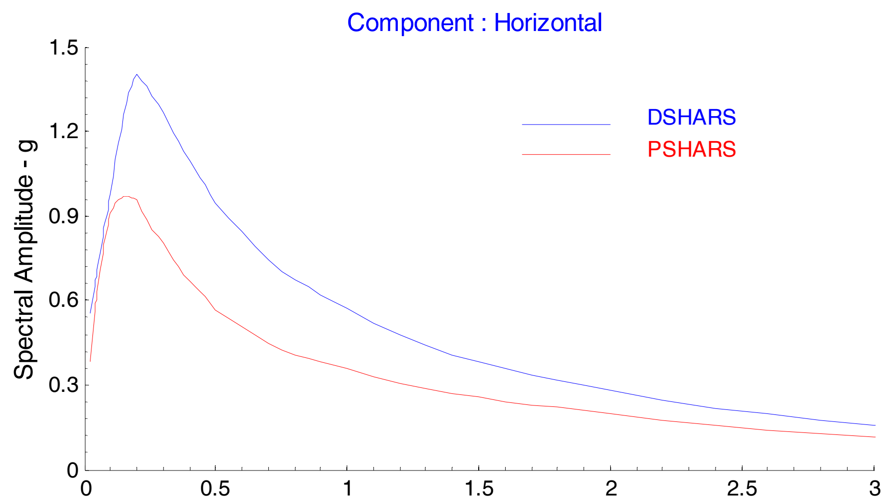


Fig.7. Comparison of the example target response spectra obtained by the DSHA and the PSHA approaches.

the magnitude has been discretized into ten intervals with central magnitude M_j = 5.0, 5.4 5.8 ….., 8.6 and $\delta M_j = 0.4$ for all the intervals. The numbers for various sources are distributed uniformly to get the number within a small distance range $(R_i - \delta R_i, R_i + \delta R_i)$. The total number within the *i*th distance range for all the sources provides the required design seismicity $\upsilon(M_j, R_i)$. By computing the probability distribution of eqn. (2) for each natural period, it is possible to estimate the spectral ordinate $SA(T)$ with a desired confidence level at all the natural periods. The response spectrum thus obtained is known as uniform hazard response spectrum, which forms the basis of the probabilistic design ground motion.

The seismicity rates $\upsilon(M_j, R_i)$ and the probabilities $q(SA(T)|M_j, R_i)$ for all combinations of M_j and R_i in all the seismic source zones are used to compute the probabilistic target response spectra. For this purpose, the magnitude has been discretized into ten intervals with central magnitude M_j = 4.6, 5.0, ….., 8.2 and $\delta M_j = 0.2$ for all the intervals, and the distance has been discretized into 76 intervals distributed uniformly on the logarithmic scale between 0.5 km and 300 km with uniform spacing on logarithmic scale. The MCE level of target response spectra for horizontal and vertical components are estimated with 2% probability of exceedance in an exposure period of 50 years (return period of about 2500 years), which are plotted in Fig. 7 along with the corresponding deterministic spectra.

4 DISCUSSION

Earthquake resistant design seeks to produce structures that can withstand a certain level of shaking without any excessive damage. The level of shaking is described by a design ground motion which can be determined by the help of seismic hazard analysis. The seismic hazard estimation involves quantitative estimation of ground motion characteristics at a project site of interest. Seismic hazard estimation is performed by the DSHA and PSHA methods to arrive at site-specific design target response spectra for a project site of interest. The DSHA method aims at finding the largest possible ground motion by maximizing the magnitude and minimizing the source-to-site distance. The uncertainty in ground motion prediction is considered by using the median plus one standard deviation level of ground motion. The approach "deterministic" in the sense that the ground motion is based on a single set of magnitude, distance, and number of standard deviations. However, the approach is not deterministic in the true sense, because the M_{max} can be determined statistically and the ground motion is also specified probabilistically. Thus the deterministically determined ground motion is, in reality, characterized by a finite probability of exceedance, which remains unquantified.

The PSHA method considers all the expected magnitude and location combinations along with all possible ground motion probabilities. It also specifies the annual occurrence rate of each magnitude-distance combination to estimate the occurrence rate for the ground motion amplitudes. Assuming Poisson probability distribution, the ground motion is then estimated with a specified probability of being exceeded in a given exposure period, which is equivalent to finding the ground motion with a specified return period (reciprocal of the annual occurrence rate). If the deterministic design ground motion is also defined with a consistent return period, then both the methods are expected to provide consistent results.

Thus, the deterministic design ground motion obtained by selecting suitable number of standard deviations to be added to the median ground motion on the basis of the recurrence period for M_{max} is expected to be more reasonable. From a review of international practice, Mejia et al. (2001) have proposed to use different percentile levels between 50^{th} and 84^{th} percentile for the deterministic target response spectra for dams with different risk levels. The underlying philosophy behind deterministic method is that the selected earthquake scenario is reasonable in terms of M and R as well as the recurrence period.

If the difference between the results of the deterministic and probabilistic methods is very wide, the magnitude and distance of the deterministic earthquake need to be rationalized by deaggregation of the probabilistic hazard analysis. The deaggregation is concerned with identifying the respective contributions of different M – R combinations to the total probabilistic hazard (McGuire, 1995). The predominant earthquakes can then be identified in specific source zones or on specific active faults, which can be used to take appropriate decision on the magnitude and distance of the deterministic scenario. Also, the probabilistic method should include all plausible deterministic scenarios. Most appropriate decisions can thus be arrive at by using both the methodologies in a complementary manner (Anderson, 1997; Anderson et al., 2000). For example, the PSHA can be used to guide the choice of the maximum deterministic events using deaggregation and DSHA estimates can be used to refine the probabilistic ones.

5 SAFETY EVALUATION OF DAMS

Site specific seismic study report would mention the design approach to be adopted for checking the stability of the dam and also recommends regarding permissible stresses and sliding factors to be adopted. As per section 5.0 of NCSDP Guidelines (2011) the recommended methods of analysis and safety criteria are as follows:

i) Seismic analysis of dams can start with simplified methods using site-specific seismic coefficient. Seismic coefficient may be used to study the seismic stability of the dam. For preliminary design, Chopra's simplified method may be used, by taking into account dam-water-foundation-interaction in a more realistic way, however the damping of the dam & foundation system is to be limited up to 10%.

ii) As per BIS code 1893 the response spectrum method shall be used for design of dams of height over 100 m, for which, site-specific response spectra should be used by working out the natural period of vibration of the dam taking into account the dam-reservoir-foundation interaction effects. Both the seismic coefficient method and response spectrum method are meant only for preliminary design of dams. For final design, detailed dynamic analysis is desirable.

iii) For detailed time-history dynamic analysis, the substructure approach may be used. Normally it is sufficient to conduct stress analysis for the tallest sections (section over deepest foundation level); one section each for non overflow and overflow portions of the dam. However, if the foundation conditions, material properties and geometry of the dam is observed to vary along the length, necessary representative sections may be needed to be analysed.

iv) For simplified method of analysis using seismic coefficient, permissible stresses may be taken as per BIS code 6512: *Criteria for design of Solid Gravity Dams*. The factor of safety for overturning and sliding may be taken to be 1.5 and 1.0 respectively.

v) When the spectra for DBE level of ground motion are used to evaluate the dynamic response of gravity dams by linear elastic response-spectrum method, the DCR are required to be less than or equal to 1.0, i.e. the tensile stress should not exceed the static tensile strength. The demand-capacity ratios (DCR) is defined as the ratio of induced tensile stress to static tensile strength of the concrete. For this purpose, the static tensile strength or capacity of the plain concrete can be obtained from the uni-axial splitting tension tests or from the static compressive strength, fc, using the relation due to Raphael (1984), $ft = 1.7\ fc^{2/3}$ where fc is in psi or $ft = 0.324\ fc^{2/3}$ where fc is in MPa, as recommended in USACE (2003). Under MCE conditions DCR allowable value of 1.5 may be used for evaluation of the linear-elastic response-spectrum analysis, i.e. the tensile stress should not exceed the dynamic tensile strength which corresponds to the 1.5 times the static tensile strength, USACE (2007).

vi) When the detailed dynamic response of gravity dams are conducted using acceleration time-histories of ground motion, under DBE condition, the DCR are required to be less than or equal to 1.0, i.e. the tensile stress should not exceed the static tensile strength. In the case of linear-elastic time-history analysis, under MCE condition, the level of nonlinear response or cracking is considered acceptable if DCR are less than 2.0 (DCR of 2.0 corresponds to the apparent dynamic tensile strength of concrete), and limited to 15 percent of the dam cross-section surface area, and the cumulative duration of stress excursions beyond the tensile strength of the concrete falls below the performance curve given in the NCSDP guidelines (2011).

6 CONCLUSIONS

The design ground motion is site-specific in nature on account of site soil condition and the expected seismicity, defined in terms of a fixed magnitude and distance combination in the deterministic method, and the occurrence rates of all possible earthquake magnitudes in each of the seismic source zones in the probabilistic method. A judicious use of both the approaches may be useful to arrive at the most appropriate decisions in practical applications.

The basic input data required for both the approaches are same which include data on past seismicity, knowledge of the tectonic features, information on site soil condition and the underlying geology, and the attenuation characteristics of the ground motion parameter of interest. The uncertainties in the available data need to be accounted appropriately in both the approaches to arrive at the design ground motion with probability of occurrence comparable to the other hazards that threaten the engineering structures.

The seismic coefficient estimated from the site specific seismic study needs to be taken in to account for checking the stability of the dams. The earthquake performance of gravity dams has to be evaluated on the basis of stresses, sliding factor, demand-capacity ratio and the associated cumulative duration.

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Chapter 6

Data Acquisition System and Wireless data transmission for a Dam structure

Abstract

Dam Instrumentation is done to verify the design parameters, evaluate performance of new technologies used in dam construction. Well instrumented dams can alert the authorities to approaching failure so that preventive measures can be initiated. Choice of proper sensor types, technology, measurement range and their location on the dam is very important to optimise costs and to extract full benefits of instrumentation. Once the sensors are installed the performance of the structure is available with a proper calibrated intelligent data logger. This paper describes a brief idea on the vibrating wire type of sensors used for the dam health assessment and explains in detail the parameters of a data logger and the transmission of the data to different locations. The advantages of using vibrating wire type sensors, is also explained. A typical dam instrumentation scheme is described to give an idea of the number of sensors and their location, cabling arrangement, and data acquisition system required to instrument a typical multi-span box section road bridge over a river.

1.0 Introduction

In order to study various static and dynamic behaviors and learn from them, geotechnical sensors are carefully located at relevant points on the body of the dam and around it. Geotechnical instrumentation is used in dam construction for one or more of the following objectives:

- To verify design parameters and to evaluate performance of new materials and technologies used in the construction.
- To verify and control the construction process.
- For performance monitoring and verification.
- For monitoring condition of the dam and to alert responsible officials in the case of approaching failure.

Advanced technology is available nowadays to perform the above monitoring and verification functions, to improve understanding of the actual loading pattern and to study the corresponding dam response. It is possible to monitor how effect of loading changes with time as the dam ages and deteriorates. Geotechnical instruments are usually hidden from view; but the benefits are never out of sight. The techniques vary from the static measurement of stress, strain, temperature and deformation to several complex dynamic measurements. Static tests can always be performed with known loading or hydraulic jacks to confirm monitoring results and to calibrate instruments used in different sections of the dam. Sensors for geotechnical and structural engineering should ideally have a lifetime comparable with the serviceable age of the structure, which is generally of the order of a few decades. World wide experience with resistive sensing elements, being used earlier, have shown that they are very easily affected by changes in external factors like temperature, humidity, cable length, nearby magnetic or electric fields, etc. The performance of the resistive sensing elements degrade over time and their output leads are prone to noise pick up from nearby electrical activity. These factors make it difficult to get stable and accurate readings from resistive sensors, especially over long periods, though they are very well suited for laboratory applications. Nowadays most sensors for geotechnical and structural engineering applications are based on the vibrating wire principle. The output from a vibrating wire sensor is in the form of an AC voltage signal, where the frequency is proportional to the measured parameter. These sensors are very rugged, simple and have a very long service life. Their signal can be transmitted over long distances unaffected by influences that render resistive sensors unusable. The following chapters will give an idea of the Vibrating wire technology, Cables, junction boxes, data acquisition system, and the data transmission technology.

2.0 Vibrating wire sensor

A sensor is an instrument that responds to a physical stimulus (such as Temperature, Pressure, velocity, Flow, Humidity, Speed and Direction, Light intensity, Water level, Heat, Light, Sound, Motion.) It collects and measures data regarding some property of a phenomenon, object, or material. As explained the latest reliable technology available in market is vibrating wire type sensors shown in figure below. Here the actual sensing element uses the elongation (deformation) caused by the force to change the preload of a vibrating wire (VW). The elongation/force-dependent frequency signal obtained can be easily transmitted. It is free from any interference and does not need any analogue-digital conversion for further processing. The following sketch explains this unique technology:

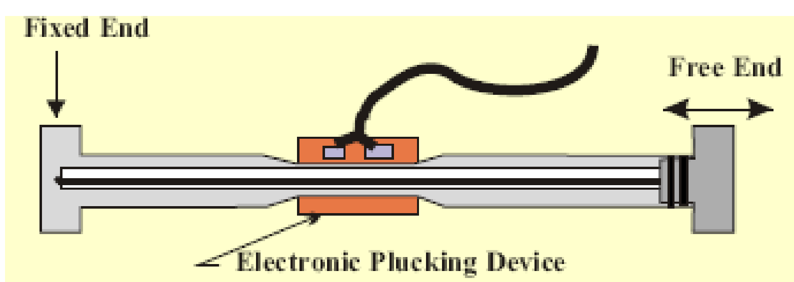


Fig1. Vibrating Wire technology

A wire is stretched between two points, a fixed end and an end which is free to move. An electrical signal is sent to the sensor which vibrates the wire. The resultant resonant vibration of the wire can be measured by a reader or data logger thus accurately determining the extent of stretching or relaxation of the wire. The excitation electronics, directly fixed to the sensor, supplies the wire with the required energy for the vibration, amplifies the frequency signal and converts it from a sinusoidal into a square-wave (TTL) signal. Such a signal can be transmitted interference-free and most microprocessors can directly accept it for further processing. The high signal output, about 20% at nominal load (a strain gauge sensor has about 0.2%) is amongst other things responsible for the exceptional precision and stability. Once the technology is embedded into the sensor all the sensors look similar and only the installation procedure differs as per the measuring parameter. As the sensors are passive in nature, the excitation signal, measurement conversion will be taken up at the data logger unit. Hence the data logger is considered to be the most intelligence part in the instrumentation activity.

The advantages of vibrating wire type sensors are many. In dam instrumentation activity, the distance between the sensor and the DAS is high for a normal voltage signal to travel, and hence the frequency signal is used. Also noise interference is relatively less. Figure below shows some of the VW type sensors used for measuring different parameters.

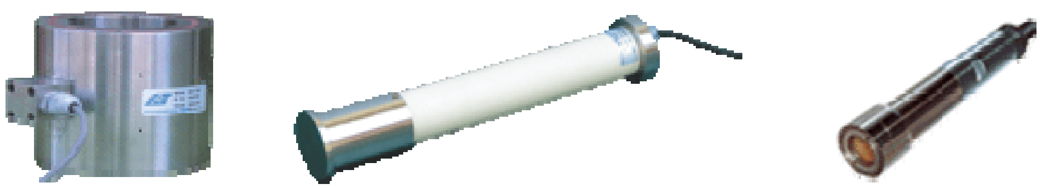


Fig.2. VW type load cell, joint meter and uplift piezometer

The sensor reacts to the excitation as per the material in which it is clamped or fixed. This information is to be taken through a good cabling arrangement to the data logger/ automatic data acquisition system.

3.0 Cables

Sensitive instruments used to measure the performance and safety of structures requires secure connections between the sensors and the readout locations. It is essential that adequate consideration is given to the connecting cables, particularly when they are to be buried within the structure or exposed where they could be accidentally damaged.

Any current-carrying conductor, including a cable, radiates an electromagnetic field. Likewise, any conductor or cable will pick up energy from any existing electromagnetic field around it. These effects are often undesirable, in the first case amounting to unwanted transmission of energy which may adversely affect nearby equipment or other parts of the same piece of equipment; and in the second case, unwanted pickup of noise which may mask the desired signal being carried by the cable, or, if the cable is carrying power supply or control voltages, pollute them to such an extent as to cause equipment malfunction. Vibrating Wire transducers do not require the use of specialist instrument cables to link them to the readout location since the signals they generate are in the form of a current and not a voltage. Even then, in order to overcome any undesirable signal loss / quality, these cables are armoured (especially in earth dams) with a high resistance to tensile loading; PVC sheathing provides waterproofing (although other jackets are available) and shielded pairs with individual drain wires provide for electrical noise protection. Most of the sensors used in structural health monitoring are four wire technology of which two wires are for the excitation and plucking the response. Other two wires are for temperature sensing as all the sensors are equipped with thermistors. Normally the cables are specified in terms of cable diameter with number of core, color with the type of shielding along with working temperature and weight.

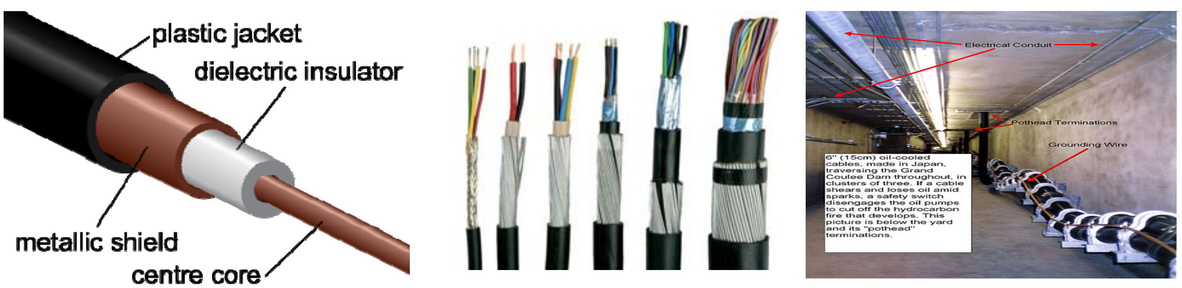


Fig 3. Shielded Armored Cables used in Dam Instrumentation

The Cables need to be tagged with proper identification, else the sensor identity is lost and the data become unusable. This could be overcome by programming the sensor with its own location, which will also be transmitted along with measured data, every time the sensor is made active.

4.0 Junction Boxes

The junction boxes are used for connecting and switching the input through one cable to another cable. This is done for easy accessibility of the sensor reading, servicing and to use the hand held data logger to monitor the data. This is normally specified by number of input connections which will be routed out through one output cable. A junction box for receiving electrical cables and for housing electrical components has a base member and a lid member positionable over the base member to form an enclosure. The lid and base members are hinged on one side. On the side opposite the hinges is a clasp assembly via which a user may lock and unlock the base from the lid by two independent locking mechanisms. Lower and upper concentric flanges slidably engage cable conduits. Knock-out elements may be removed from within the concentric flanges to form apertures through which cables may pass from an engaged cable conduit into the enclosure. A mounting flange may be used to install the junction box within a recess of a building, between wall layers. A skirt flange may project from the junction box in the exterior direction past the external wall layer to convey moisture away from the building recess.

Fig 4. Typical View of a Junction Box.

5.0 Data Logger

This main focus of this topic is to give an insight into data acquisition techniques. Data acquisition is the process of measuring an electrical or physical phenomenon such as voltage, current, temperature, pressure, or sound. PC-based data acquisition uses a combination of modular hardware and flexible software to transform the standard laptop or desktop computer into a user- defined measurement or control system. While each data acquisition system has unique functionality to serve application- specific requirements, all systems share common components that include signals, sensors, signal conditioning, DAQ hardware, and a computer with software. In the front end the VW sensor (or transducer) converts a physical phenomenon into a measurable electrical signal, such as voltage or current. Transducers convert physical phenomena into measurable signals, however, different signals need to be measured in different ways. The figure below shows the logic of Data Logging.

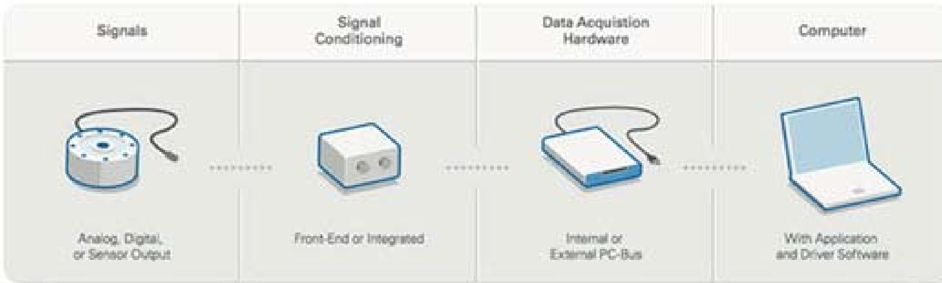
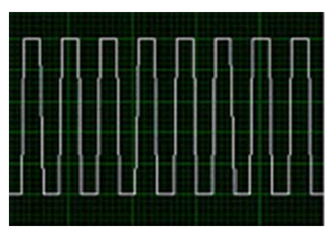


Fig 5. Data acquisition Flow

To select a data logger, it is important to understand the different types of signals and their corresponding attributes. Signals can be categorized into two groups: analog and digital. An analog signal can exist at any value with respect to time. A few examples of analog signals include voltage, temperature, pressure, sound, and load. The three primary characteristics of an analog signal are level, shape, and frequency. All analog signals can be categorized by their frequencies. Unlike the level or shape of the signal, we cannot directly measure frequency. You must analyze the signal using software to determine the frequency information. This analysis is usually done using an algorithm known as the Fourier transform. A digital signal cannot take on any value with respect to time. Instead, a digital signal has two possible levels: high and low. The useful information that one can measure from a digital signal includes the state and the rate.

	
Fig6.a Analog Signal	Fig 6 b.Digital Signal

The measurement typically carried out in a dam site is analog in nature. This requires highly sophisticated data logger. As already told the hand held unit will have provision to excite the sensor when the cable is connected and to pluck the response with the noise pickup. Hence a power electronic design in the form of Embedded technology is normally incorporated. A Typical data logger is shown in the figure below, which can measure the frequency of the current signal in terms of period generated by a VW sensor. It also displays the frequency, frequency squared value, which is the actual value of the measured parameter in proper Engineering unit. The frequency to engineering unit conversion is done inside the data logger. Normally the initial measurement of the sensor, at the time of installation is fed for all the units in the memory of the data logger. This value when multiplied with the gauge factor, which is derived during calibration of the sensor at the lab before installation gives the actual engineering unit measurement. Normally more than 5000 readings with date and time for around 500 sensors can be accommodated. The data logged in this unit can be downloaded through serial/ USB ports into a computer for further analysis. Present day data loggers allow bidirectional data streams over USB, Ethernet, and Wi-Fi (IEEE 802.11). The new technology helps engineers achieve high-performance applications on these external buses, which previously was possible only on an internal bus such as PCI.

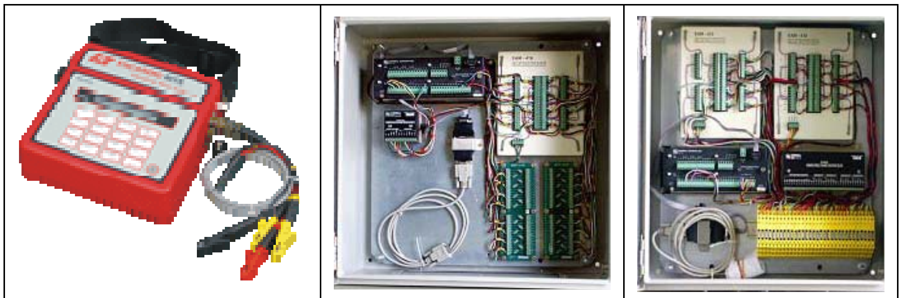


Fig 7a. Handheld data logger   Fig7b. DAQ internal wiring

6.0 Automatic Data Acquisition and Wireless Transmission

An automatic data acquisition system shown in fig 8 below typically does not need any human intervention as it is programmed to log the data at stipulated interval and store it in its memory for further processing. Typical specifications of a data logger/DAQ is their channel handling capacity, operating speed, download capability, accommodating the type of inputs, operating temperature etc. Automated Data Acquisition and Alarm Reporting System (ADDAARS) was designed to obtain, monitor and analyze, in real-time, critical safety parameters such as inflows, outflows, gate openings and lake elevations for 29 principal reservoirs. This system allows dams to be operated more safely and emergency plans to be more effectively coordinated and implemented with the emergency management agencies. ADDAARS provides information to decision makers in realtime through a combination of radio and satellite telemetry, microwave, fiber optic and internet technology in order to Improve the distribution and use of manpower, Provide real-time data for safe dam and reservoir operation, Reduce emergency response time and to Provide a precise and consistent data collection system. The logical connection is shown in figure below.

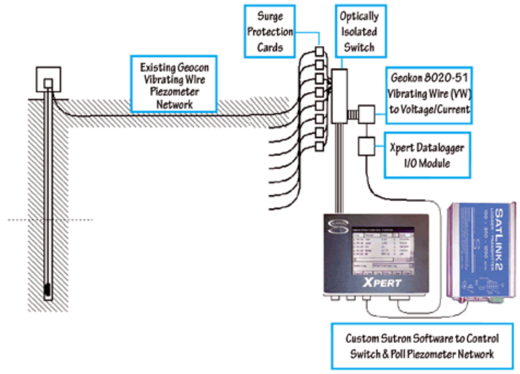


Fig 8. A customized automatic data logger Logic ad the system

The system shown here can scan the data channels from a few times per second to once every few hours. The instruction set includes 44 measurement, 52 processing/math, and 18 program control instructions. Data and programs are stored either in non-volatile flash memory or battery-backed RAM. The standard memory stores 26,000 data points in final storage area. This system's digital output ports in combination with suitable device drivers can interface many control devices available commercially such as valve actuators, motors, solenoids, relays, etc. They can also be used to control external alarm annunciators like lamps and buzzers. Multiplexers allow a number of sensors to be measured by a single controller. The multiplexer module sequentially multiplexes 16 groups of four lines or 32 groups of two lines at a time or even more. Compatible sensors include vibrating wire, thermisters, thermocouples, potentiometers, load cells, strain gauge etc. Several multiplexers can be controlled by a single processor. Once the data acquisition is done automatically in a programmed interval the data is to be stored and transmitted forward for analysis and interpretation. The DCP collects, stores, and prepares data for transmission via Satellite and Radio. The DCP manages all scheduling of sensor readings and organization of sensor data. The Satellite Transmitter receives data from the DCP and transmits it to the Satellite. Satellite communications are one-way from the station to the receive site. After the DCP is programmed, nothing can be done during an event without reprogramming the DCP. Since all transmissions are scheduled, this is a near real-time system. During an event, the DCP will provide all the data needed. All scheduling of satellite transmissions is handled by the DCP and the satellite transmitter. In case of emergency the system will give alarm or operate the gates if programmed to do so. The transmission technology is schematically shown in Fig 9 below. The control activities are efficiently managed with the SCADA system in most of the foreign countries. Knowledge based algorithms are used in prediction of the worst case scenario and to take efficient decisions. The data logging, plotting and control is done in a secured system operating with SCADA software. Real-time data is presented here in graphical formats, enabling us to visualize and interact with information in a wide range of ways such as numeric and historical trend data, application reports and remote connections and many others. Precise analysis and a quick response time result in fewer losses and higher quality.

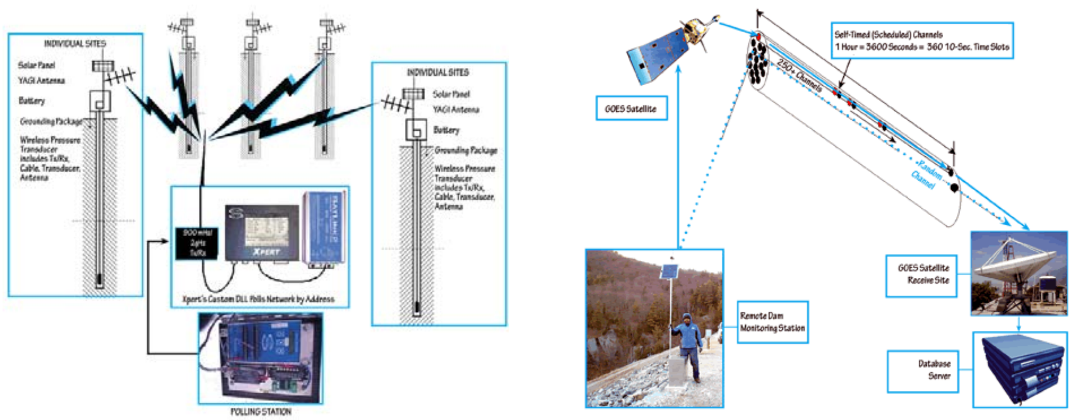


Fig 9. Transmission technology

7.0 Case Study

CWPRS was associated in one of the very important and successful project at Madhya Pradesh, where the dam is fully automated. This multi purpose project is constructed across the river Narmada in Khandwa Distt of Madhya Pradesh. The concrete gravity dam has been constructed with 27 blocks, which consist of 07 Nos. of non-over flow (NOF) blocks and 20 Nos. overflow (OF) blocks. Instrumentation has been carried out for one overflow and one non overflow blocks since 1996 in phased manner depending upon rising of the blocks. The instruments, which have been installed, are of Vibrating Wire types. A total of 182 instruments have been installed at various levels and chainages of the dam. These data are logged on a versatile Central Data Acquisition in regular and preprogrammed interval and used for analysis and interpretation. In this project few sensor data was not included as their identity is lost. The data analysis and interpretation was also carried out at CWPRS.

7.0 CONCLUSION

The objective of making the paper is to make the user understand the present day technology and its capability towards application in a dam structure. Decision making in such applications are very crucial, which needs through understanding of the techniques. It is found, that at several cases in Indian dams, instrumentation carried out are not effective, due to improper selection of sensors, routing of the cables, selection of data acquisition system and the associated software. Though this topic discussed looks very simple, practical difficulties are expected in the cable routing and bringing the data till the junction box and to the data logger unit. Once data logging is assured, the transmission, analysis and control is very much possible with the modern Graphical User Interface control technology. Since the actual instrumentation scheme, which involves the choice of instrument, cable, location, type and measurement range is a very specialized subject, it is best left to the experts in the field.

8.0 ACKNOWLEDGEMENT

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Annexure-I

The Following are the instruments installed at Omkareshwar Dam built across Narmada River, Madhya Pradesh.

The instrumentation was provided by M/s AIMIL Ltd is a leading ISO: 9001:2008 certified company, with an all India network of ten offices, staffed and managed by over 500 professionals. It was established in 1932 .We provide state of art technology for generating eco-friendly and sustainable solution in the field of Civil Engineering, Electronics, Analytical and Tele-communication.

The following are the instrumentation carried out at Omkareshwar Project:

Salient Features of Project :

Type of Dam
Height of Dam
Purpose of Dam
Total Power Generation
Reservoir Area
Total Storage Capacity
No of Overflow Blocks
No of Non Overflow Blocks

Annexure-1

Details of Instruments Installed :

	Instrument
	Froup of Five strain meter with Rosette & Junction Box
	Strain Gauges
	No Stress Strain Meter
	Stress Meter
	Pore Pressure Meter
	VW Temperature Meter
	VW Joint Meter

	Uplift Pressure Meter
	Normal Plumb Line
	Inverted Plumb Line
	Tilt Meter
	V Notch
	Single Point Bore Hole Extensometer
	Strong Motion Accelerograph
	Seismograph
	Automatic Data Acquisition System
	Digital Read Out Unit
	4 Core Steel Armoured Jelly Filled Cable
	20 Core Steel Armoured Jelly Filled Cable
	40 Core Steel Armoured Jelly Filled Cable
	10 position Switch Cum Junction Box

A. Group of Instruments (Strain Meters,No stress strain meter , Pore pressure meter, Stress Meter and Temperature Meter)

1. The above instruments are installed in block no.5,14 and 23 exactly as per instrumentation drawings issued by department.
2. A few piezometers are installed in other blocks also whose readings are satisfactory.

3. The Switch boxes are installed in gallery and protected from metallic box properly.

4.The cables from each instrument are reaching in gallery and suitably terminated in switch cum junction box.

5.Multi Core (40 core)cables are laid from switch boxes to control room located on the top of dam.
The connection of each instrument with multicore cable is done properly.

6.The data from group of instruments are being recorded periodically and they are satisfactory.

S/N	Instrument	Total Qty

1	Froup of Five strain meter with Rosette & Junction Box	15
2	Strain Gauges	16
3	No Stress Strain Meter	15
4	Stress Meter	15
5	Pore Pressure Meter	64
6	VW Temperature Meter	4
7	VW Joint Meter	16
8	Uplift Pressure Meter	14
9	Normal Plumb Line	3
10	Inverted Plumb Line	3
11	Tilt Meter	3
12	V Notch	6
13	Single Point Bore Hole Extensometer	3
14	Strong Motion Accelerograph	1
15	Seismograph	1
16	Automatic Data Acquisition System	1
17	Digital Read Out Unit	3
18	4 Core Steel Armoured Jelly Filled Cable	2000Rmt
19	20 Core Steel Armoured Jelly Filled Cable	500Rmt
20	40 Core Steel Armoured Jelly Filled Cable	3300Rmt
21	10 position Switch Cum Junction Box	36 No.

Chapter – 7

Safety Review and Rehabilitation of Concrete and Masonry Dams

INTRODUCTION

Most existing dams during earlier days have been designed by very simplified methods that are now considered simplistic and inaccurate. These dams are more than 40 years old. Moreover, due to ageing effect and old technologies employed in the construction of dams during that period, many dams have started showing distresses in the form of cracks, large deformations, seepages, bulging of faces of dams and galleries, loss of mortar in joints of masonry dams and dislodging of concrete from faces etc. Under these distresses, the condition of the dam looks very scary and apprehensions are created about the structural safety of dam. Further, the damages are sustained by some dams that have been subjected to intense ground motions and growing concern of seismic safety has led to considerable interest in evaluating structural safety of the dams against static and earthquake forces. The distresses developed in dams are monitored through dam instrumentation and visual inspection from time to time. Based on initial assessment, the need of detailed assessment of safety is considered before taking any detailed remedial measures. Several methodologies such as geophysical tomography, nuclear logging, tracer techniques and mathematical modelling are used to evaluate the structural safety of the existing gravity dams.

VARIOUS CAUSES OF DISTRESSES IN GRAVITY DAMS

There are many causes of distresses in the dams. Some causes of distresses in dams such as deficiency in structural design, lack of proper foundation investigation, under estimation of hydraulic parameters, deficiency in quality control of material during construction, deterioration of dam material, damages during earthquake, unexpected high flood, high pore pressure in dam body, leaching of lime due to alkali aggregate reaction, development of high uplift pressure in toe region, malfunctioning of spillway/Undersluice gates, Seepage through dam body or foundation, mining and blasting activity in the vicinity of dam, decay of material due to ageing effect, sabotaging by criminal activity etc., are responsible for distresses.

METHODS FOR ASSESSMENT OF DAM SAFETY

Different methods of assessment of safety and rehabilitation of dam are as follows:

1. Physical inspection of the dam
2. Assessment of weak zones in the body of the dam by Seismic tomography
3. In-situ determination of properties of the dam material by nuclear and sonic logging.
4. Identification of seepage route by tracer technique
5. Laboratory determination of engineering properties of the dam material
6. In-situ determination of material properties by Flat Jack Test
7. Stability analysis of the dam by Finite Element Method
8. Selection of suitable repair material for rehabilitation
9. Repair and rehabilitation of dam

Physical inspection of the dam

Dams are regularly physically inspected by project authorities and experts for monitoring structural health of the dam. Damages in the form of cracks, seepage, tilting of dam, opening of joints settlement etc., are generally observed at several locations such as operation and foundation galleries, upstream and downstream faces of the dam. The possible causes of distresses are identified. The analysis of dam instrumentation data is also studied to get the idea about the beginning and locating the location of the distresses. The physical inspection gives preliminary idea about the overall health of the dam. The type of further investigations to be carried out, is decided based on findings of physical inspection of the dam.

Assessment of weak zones in the body of the dam by Seismic tomography

The quality of in-situ material of dam is an important parameter for the safety of the dam. Geophysical Tomography is a means of making a picture of a slice of the material of dam. Tomography is a type of inverse problem where measurements are first made of some energy which has propagated through a medium. The received character of this energy (e.g. amplitude, travel time or potential difference) is then used to infer the values of medium through which it has propagated. The seismic velocity and attenuating properties of the dam material can be related to the observed travel time and amplitude of seismic wave by a line integral along a ray path.

In tomography testing of the dam, P-waves are generated on the upstream face of the dam at two meter interval by using a hammer source and with a sparker in the water covered portion. These seismic waves are recorded using vertical geophones having 14 Hz natural frequency fixed at two meter interval on the downstream face of the dam. For each seismic traverse consisting of 24 geophones for recording seismic waves, 24 hammer shots are used for their generation thus providing 576 ray paths for each plane starting from upstream to downstream. The plane separating the source and receiver holes is divided into a mesh of grid cells known as finite elements. Each element in the mesh is assigned a starting velocity and the synthetic travel time for the portion of each ray path passing through is calculated. In this way the total travel time for each ray path is calculated and then compared with the measured travel time. The velocities assigned to the various elements are then adjusted iteratively until the calculated and measured travel times for the ray paths are the same. The resulting velocity image is termed a tomogram and enables identification of anomalous velocity zones lying between the upstream and downstream face of the dam.

This technique was recently applied at Murbadi dam situated in Thane district of Maharashtra. The travel time data was collected in nine planes (Fig. 1). The travel time data was analysed by seismic ray tomography using algorithm of Jackson et.al (1992) which enables imaging of the velocity distribution within the sampled area. Results for all the planes are presented in the form of velocity distribution contours, out of which two significant planes are shown in Fig. 2. The tomogram revealed that the velocity through the masonry varies from 2500m/sec to 4000m/sec, which is an indication of its good quality. However, for the plane 4-4 at elevation RL 107.8m, large zones having, velocities less than 2500 m/sec are deciphered. This indicates the masonry to be of inferior quality. Similar low velocities are obtained for inclined zone from RL107.8m to RL 104.6m. This inclined zone of poor quality masonry might have to be treated by selective grouting for arresting the seepage and for safety of the dam.

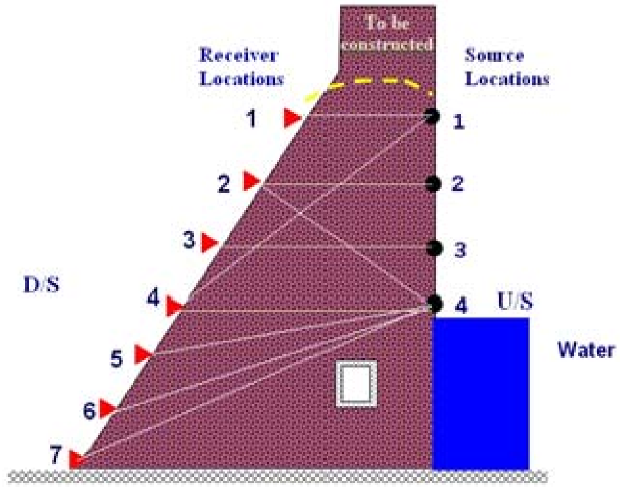


Fig. 1: Planes covered for seismic tomography of the dam

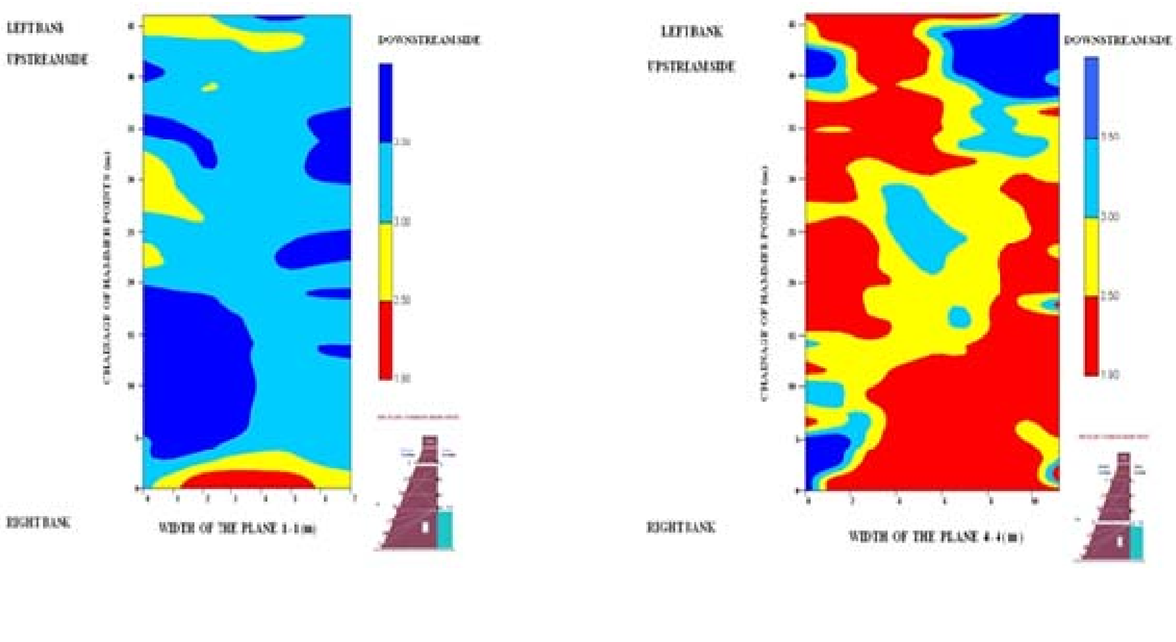


Fig. 2: P-Wave Velocity (Km/sec) distribution along Plane 1–1 and Plane 2-2

In-situ determination of properties of the dam material by nuclear and sonic logging

Nuclear and sonic borehole logging techniques are being effectively used to investigate weak zones in dams and for determination of in situ properties of body mass of dam. The determination of physical properties provides information about the health of a hydraulic structure. Nuclear logging either utilizes the radiation emanating naturally from the surrounding media or the radiation induced by interaction of the media with neutrons or photons or both. In acoustic logging devices they generally contain one or two transmitters that convert electrical energy into acoustic energy, which is transmitted through the environment as an acoustic wave. The receivers then convert the acoustic energy back into electrical energy for transmission through the cable. One such site study using bore hole logging has been carried out at Kolkewadi dam. The objective behind the study was to evaluate the dynamic properties of masonry required for strengthening of the dam. Borehole logging with gamma-gamma, caliper and sonic probes were employed at selected sites (Nx size boreholes) within the dam body to determine in situ physical properties. Figure.3 shows a plot of the dynamic physical properties of the dam body, obtained from nuclear and sonic logging at Kolkewadi dam.

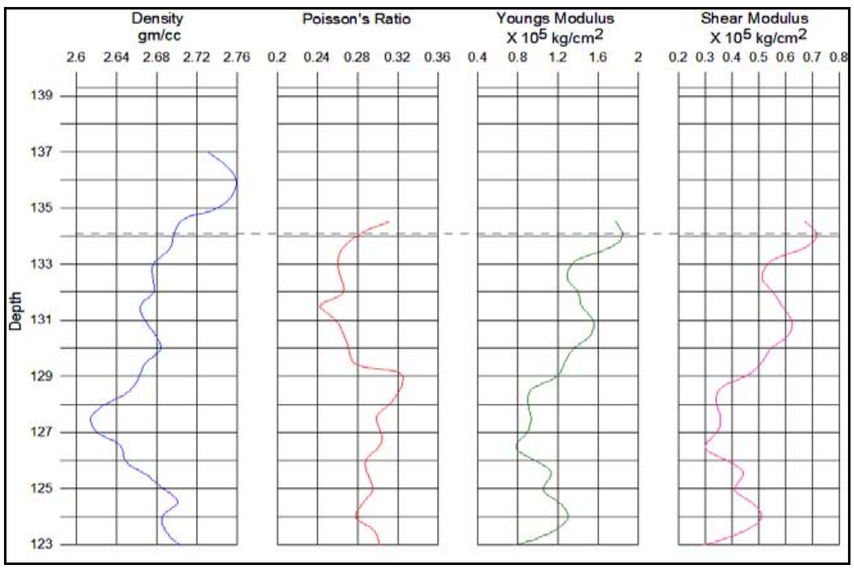


Fig. 3: Dynamic physical properties obtained from nuclear and sonic logging

Based on the above studies it was possible to evaluate in-situ bulk density, shear and compressional wave velocities, Poisson's ratio, shear and Young's modulus of masonry in the dam. The studies also revealed minimum and maximum values of dynamic properties for different proportion of Un-coarse Rubble Masonry (UCR) in the borehole. The density, shear and compressive wave velocities of masonry dam obtained by nuclear density and sonic logging and computed values of Poisson's Ratio, Young's and Shear Modulus were utilized for adopting strengthening of the dam.

Identification of seepage route by tracer technique.

The tracer method is widely used as an advanced and cost effective technique to detect seepage zones, seepage path, seepage loss, movement of water etc. qualitatively and quantitatively. The major objectives of using tracer techniques in geotechnical studies are to determine seepage in dams, location of seepage entry zones, delineating seepage path, assessing the efficiency of remedial measures, examination of soundness of bedrock etc. Tracers are broadly classified into two groups: (i) Conventional tracers and (ii) Isotope tracers including stable isotopes viz. ^{13}C, ^{2}H, ^{18}O, environmental isotopes viz. tritium etc. and unstable isotopes like ^{3}H, ^{51}Cr, ^{60}Co etc. (Gaspar et. al, 1972). The choice of tracer is important as it must behave like the material to be traced but be distinguishable from it for the purpose of detection.

Dye tracers like sodium fluorescence generally used for dam seepage analysis are either injected in the bore wells or in the reservoirs. One such site where this methodology was adopted is at Bhama – Asked Dam, Maharashtra. Tracer test was used to delineate the path of the seepage from the damaged portion of the tailrace channel. The results of tracer study after laboratory analysis is shown as tracer break through curves in Fig.4 (R.K.Kamble et.al, 2011).

The tracer study could establish the interconnection between the oozing of water in the tailrace channel and the permeable zone formed by the weak red Breccias in the foundation. Based on the results, suitable recommendations were made for proper drainage holes in the tailrace channel area to relieve uplift pressures in order to reduce or stop the leakage.

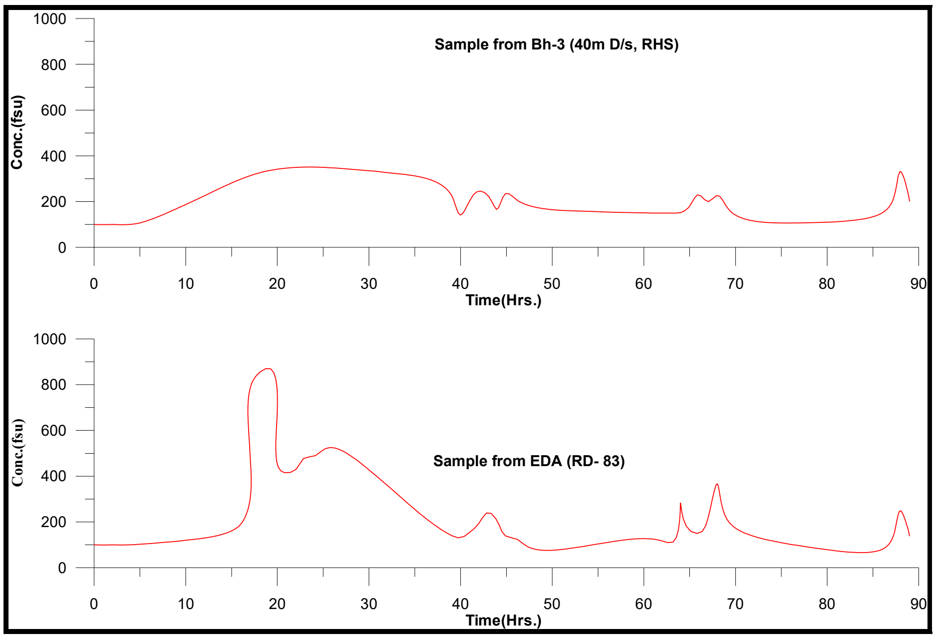


Fig. 4: Tracer breakthrough curves from dam site Bhama-Askhed.

Laboratory determination of engineering properties of the dam material

The engineering properties of concrete/masonry dam such as Young's modulus of elasticity 'E', Poisson's ratio, Crushing and Tensile strengths are used as an input parameters for stability analysis. For the determination of these properties, cylindrical cores of diameter from 150 to 200mm from important locations are collected by drilling in the dam body at top, upstream and downstream faces. Usually, minimum 3 samples or more are collected from each location. Also samples are prepared in the laboratory by collecting ingredients of dam material from the site / quarry. Collected samples are tested on Universal Testing Machine (UTM) in the laboratory under controlled conditions to determine engineering properties under static conditions. Dynamic Modulus of Elasticity is determined by Resonant frequency method.

Recently, such laboratory studies were conducted for Kolkewadi Dam. Kolkewadi dam constructed across Boladwadi stream in Maharasthra lies in high seismic zone III near to Koyna Dam. The total length of the dam is 497 meters and maximum height above the foundation is 66.30 meters. The dam is constructed in concrete and UCR masonry with cement mortar proportion of 1:3, 1:4 and 1:5. The material properties of Kolkewadi dam was determined in the laboratory by taking cores from the body the dam to cover all types of masonry / concrete forms to be used as input parameters for carrying out dynamic analysis of dam. It was observed that there is a large variation in the properties due to varying proportion of mortar and stone samples and also due to change in the orientation of stones in the cores.

In-situ determination of material properties by Flat Jack Test

The flat jack method consists of cutting a thin slot approximately elliptical in shape into the concrete/rock surface after fixing two/four pins at a fixed distance, by drilling a series of overlapping holes. The distance between pins is measured with accuracy up to 0.0001”. A view of the slot cut by jack hammer drilling after fixing of pins is shown in Fig.5. This process relieves the concrete/rock surface of the stress originally existing across it. Because of the stress relief, the sides of the slot converge.
The amount of convergence depends upon the stresses existing at that point and its elastic properties. The convergence after making of slot is measured between two/four reference points fixed at a known distance on either side of the slot prior to cutting of the slot. A view of the measurement of distance by deformeter between four references pins after the slot is made is shown vide Fig.6. The flat jack is then embedded tightly in the slot by quick setting mortar, for filling the gap between the jack and the slot.

Fig.5- Horizontal Slot alongwith fixed pins

Fig.6- Distance by Deformeter

The flat jack is then pressurized by a hydraulic pump until the displacement between the pins which took place after the slot is made is cancelled and the pressure required to cancel the displacement after making of the slot is called the cancellation pressure. The cancellation pressure is very nearly equal to the pressure/stress existed in the normal to the
plane of slot before the slot is cut, provided the length of the slot and the jack are same. A view of Flat Jack test in progress is shown vide Fig.**7**. In flat jack test the stress displacement envelope obtained during stressing of the slot for measuring cancellation pressure is used to evaluate deformation modulus, E_m.

Fig.7- Flat Jack Test in progress

Recently, Flat Jack Method has been conducted at Bhakra dam. Bhakra Dam and its two contiguous Power Plants form the principal features of the overall Bhakra Nangal Project which in addition consists of Nangal Dam. Dam Safety Committee of Bhakra Beas Management Board during its inspection of 1990 has desired to carry out Dynamic Analysis of Bhakra Dam due to increase in seismic activities in Himalayan region. As such, the dynamic analysis of the dam, besides a review of the structural stability of the dam for the normal static loading conditions, have to be carried out, by taking into consideration the in-situ properties of materials (such as, rock and concrete Modulii, etc). The different portions of dam have been constructed with different concrete strength combinations. For determining the realistic values of the different constituents of the dam the normal practice followed is laboratory testing of extracted concrete cores. But the Dam Safety Committee did not allow disturbing any part of the dam by extracting the concrete cores throughout depth of concrete. So insitu properties of dam concrete and surrounding rock mass are determined by flat jack method, which comes in the category of non-destructive test (Fig.8). The flat jack method is used to determine deformation modulus, cancellation pressure and Poisson's ratio.

Fig. 8: Location of Flat Jack test at D/S (R.H.S.) Face of Bhakra Dam.

Stability analysis of the dam using Finite Element Method

For assessing the stability of gravity dams, 2D/3D stress analysis is carried out by Finite Element Method. Finite element models are used for linear elastic static, pseudo-dynamic and dynamic analysis and for nonlinear analyses that account for interaction of the dam and foundation. The finite element method provides the capability of modelling complex geometries and wide variations in material properties. The complicated foundations involving various materials, weak joints on seams and fracturing can be readily modeled using finite element method. Two dimensional, finite element analysis is reasonably appropriate for long conventional dams with transverse contraction joints and without keyed joints. The dams located in narrow valleys between steep abutments, multiple openings, spillway blocks and dams with varying rock moduli across the valley are conditions that necessitate three dimensional modelling. Sometimes in addition to static and pseudo-dynamic, dynamic analysis is also required to be carried out using site- dependent earthquake ground motions.
The stress analysis is carried out by FEM using general purpose finite element programs such as ANSYS, SOLVIA, LUSAS etc., by taking actual material properties and site specific seismic parameters under seven load combinations as per IS:6512-1984. The value of seismic coefficients in vertical direction is taken as 1/2 to 2/3 of horizontal seismic coefficient and the value of seismic coefficients is taken as 1.5 times at top of the dam reducing linearly to zero at base of the dam as per IS1893-1984, 2002. After carrying out stress analysis, stability of the dam is evaluated against overturning, sliding and allowable stresses based on IS: 6512-1984 recommendations. Characteristic locations within the dam in which a stability criteria check should be considered include planes where there are dam section changes and high concentrated loads. Large galleries and openings within the structure and upstream slope transitions are specific areas for considerations. It is presumed that if stresses are within

allowable limits, then overturning will not take place. The factor of safety against shear / sliding along the dam foundation interface is evaluated using following equation:

$$F = \frac{\dfrac{(w-u)\tan\phi}{F\phi} + \dfrac{CA}{F_c}}{P} \qquad (1)$$

Where,

w= Total weight of dam, u=Total uplift force

tan ϕ=Coefficient of internal friction of the material P= Total horizontal force,

C=Cohesion of the material at the plane considered A=Area under consideration

F_ϕ and F_c =Partial Factor of safety in respect of friction and Cohesion

The average of factor of safety should be greater than one at every plane of computation under all load combinations. Recently,2D stability analysis using a two-dimensional finite element model of 26.50 m high Overflow block of an existing dam has been carried out by discretizing the dam foundation system into 456 nine noded isoparametric elements using 1474 nodes under above listed seven load combinations by taking seismic coefficients ($\alpha_{Horizontal}$ = 0.108, $\alpha_{Vertical}$=0.0504) for seismic zone II. The input parameters include mass density of dam material (2.4×10^{-3} Kg/cm^3), 'E' for dam and foundation (2.20×10^5 , 2.5×10^5 Kg/cm^2) and poisson's ratios for dam and foundation (0.20, 0.15). The seismic coefficients have been applied as 1.5 times of values at top of the dam and gradually reduced to zero at dam base. Results are obtained in the form of principal stresses as shown in Fig.5, direct stresses and deflections at nodes.

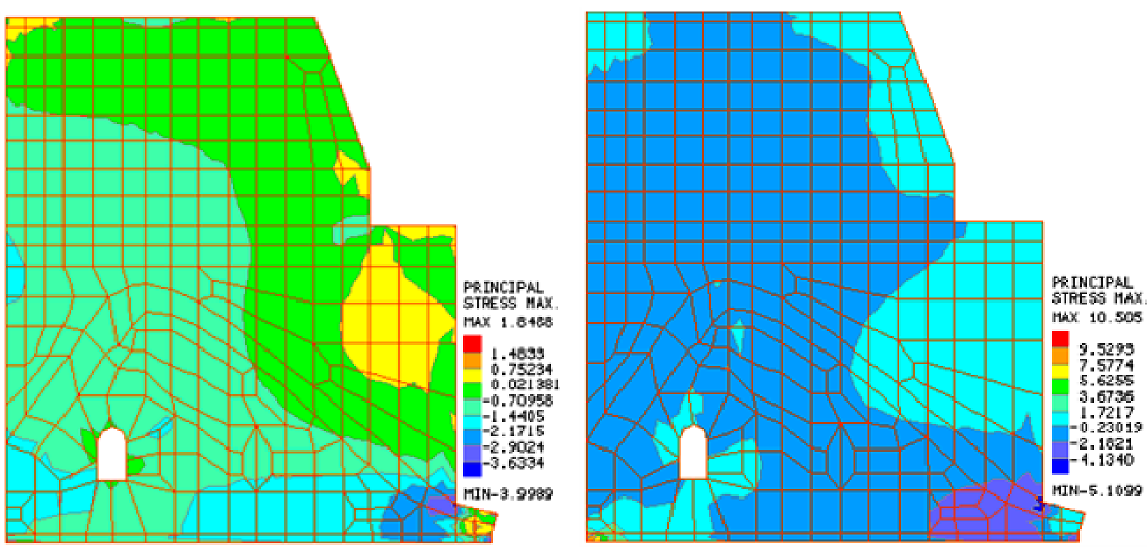


Fig. 5: Maximum Principal Stress Contours for Load Combination B & E

Based on results of analysis, factor of safety against sliding has been calculated at the interface of dam and foundation using above mentioned equation and plotted with respect to base width of the dam at the selected plane (Fig.6). The average factor of safety is calculated under all seven load combinations and stability of the dam is assessed as per above given criteria. Maximum tensile stress increases tremendously under earthquake loads as compared to static load combinations but still remain within allowable limits. Average factor of safety under all load combinations is also found to be greater one. Hence the dam has been found safe under structural stability analysis.

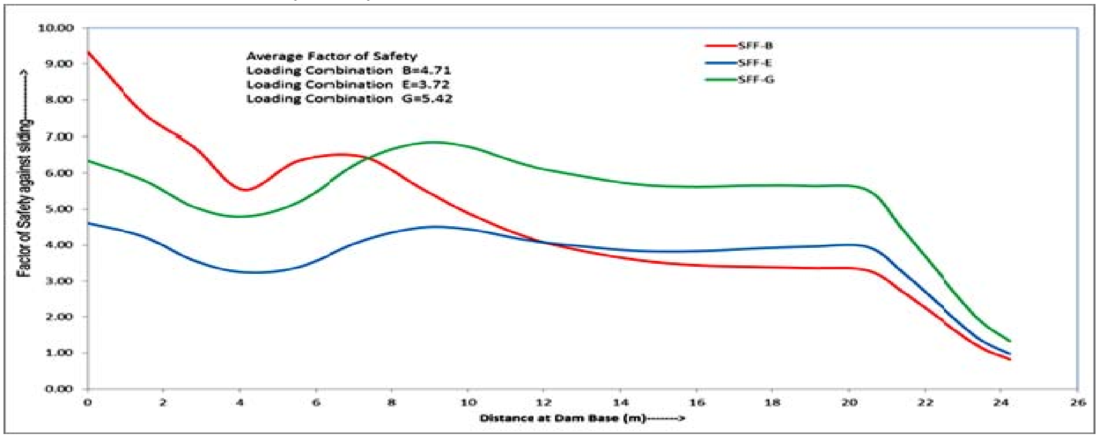


Fig 6: Variation of Factor of Safety against Sliding at Dam base Foundation Interface

Selection of suitable repair material for rehabilitation

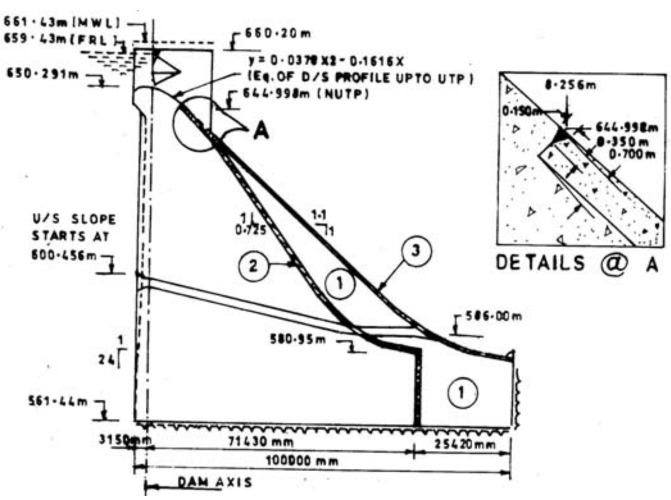
There are innumerable combinations/systems of repair materials to suit different requirements. An ideal repair system would be the one which possesses the properties for maximum penetration with longer pot life, ability to develop bond with concrete/substrate material, ability to harden to a mass stronger than adjoining concrete, ability to withstand normal variation in ambient temperature without losing its properties to a harmful degree and durability. Laboratory studies therefore form an important part in determining the suitability of repair material for the particular application. Following laboratory tests are important for selection of a suitable system/repair material for its application at site.
For Grout Systems - Mix viscosity, pot life, compressive/ tensile strength, Penetrability, bond strength etc.
For sealing system - Usable time, Pressure bearing capacity.
For cementitious/ epoxy mortar/concrete material – workability, density, strength, elasticity, bond strength, abrasion resistance etc.

Recently, this technique was applied at Murum Silli composite gravity dam. The Murum Silli dam is constructed during 1914 to 1923 in Chittisgarh across Silauria River, one of the tributaries of the Mahanadi to supplement the flow in the river during the periods of low supply. The siphon spillway of the dam was found damaged after passing of the flood discharge for several years. For strengthening the damaged siphon spillway for its intended use, laboratory studies on two selected epoxy compounds namely Sikadur-31 & Sikadur-41 for evaluating their suitability in repairs/strengthening of damaged siphon spillway, were conducted. The physical properties such as Compressive Strength, Tensile strength, Modulus of Elasticity, bond strength with concrete in direct tension under dry condition and Abrasive resistance for both the systems were determined after casting samples and testing after seven days.

Repair and rehabilitation of dam

The objective of repairs is to reinstate the structural integrity of the element, improve durability, prevent the ingress of corrosion promoting materials, restore impermeability and improve the appearance of the concrete surface. Whereas, the purpose of rehabilitation of the distressed structures is to remove the weak zones and replace them or repair them with stronger materials to make monolithic structure, reintroduce a protective and durable environment around the reinforcement and prevent further action of deteriorating agents from the external environment. If the cause of distress is understood, then an appropriate repair system will be selected and consequently, the repair will be successful with a maximum life.

The repair process generally consists of removal of deteriorated or damaged concrete, surface preparation, installation of repair materials and implementation of specified repair techniques. The proper repair techniques and material selection and installation are essential for a successful project that is capable of providing the serviceability and durability of any repair. In case of distressed concrete / masonry hydraulic structures for making them serviceable, measures such as grouting of cracks on upstream face, sluice barrels, galleries both dry exposed and underwater surfaces, sealing of joints, gunitting u/s face, jacketing and use of geomembrane /geofabrics, treating u/s face with polyurethane coating or by epoxy mortar, repairs and upgradation of hydraulic gates by epoxy compounds and by providing additional spillways, strengthening of dam by buttresses or by continuous backing at downstream as shown in Fig.7 and bonding of old to fresh concrete with polymer base materials , use of colgrouts, strengthening masonry dam by providing concrete lamina on U/s and D/s faces, concrete capping, by cable anchoring, improving drainage in dam body, restoration of stilling basin by repairing the eroded portion, grouting of foundation and structure, are usually adopted.



a) Section of Strengthened Spillway b) A view of strengthening of Spillway

Fig.7: Strengthening of spillway by adding continuous backing of concrete

Conclusions:

Due to ageing of dams decay, deterioration and distresses are occurred. An assessment of safety of such dams is essential for taking remedial measures to rehabilitate the dams in order to safeguard the safety of people and property. The multidisciplinary approach for assessment of safety of dam is the need of hour. An application of many methodologies provides physical properties of body of the dam, which in turn give better assessment of status and safety of dams. The physical inspection of dam provides damages in the form of cracks, seepage, tilting, opening of joints etc. Geophysical tomography of dam provides status and health of the dam by identifying weak zones and soundness of body mass of the dam. The cores drilled from the weak zones and laboratory determination of properties provides insight of quality of the material. Nuclear and sonic logging in the boreholes gives dynamic properties of material. These properties are used as input to assess the safety of the dam by applying FEM. Tracer technique delineates the path of seepage in the body of the dam. After assessing the safety of dam, suitable repair materials and repair methodologies are decided. The related case studies have proved utility and potential of an integrated approach for assessing safety and rehabilitation of dams in systematic and economical manner.

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Chapter – 8

Safety, Stability And Rehabilitation Of Earth / Rockfill Dams

1.0 INTRODUCTION

Dams have been built across rivers by mankind right from the civilization for storing the river flow during rainy season and releasing it during the remaining part of the year for the purpose of domestic, irrigation, generating electricity, flood control etc. Dams constitute perhaps the largest and the most complex of structures being built by engineers. The dams are built to last from 100 to 300 years depending upon merits of each case.

Dams are designed to withstand all the possible destabilizing forces with a certain factor of safety. It has been an indicator of a factor of ignorance or lack of knowledge of various response processes of materials used in construction, the stresses caused, the strains experienced and finally the failure mechanism. Design of large dams has to be extra safe that there is a minimum probability of their failure. Sudden release of storage can cause disproportionate flooding and losses to the human habitats in the downstream. Failures of dam by overtopping and seepage/ piping constitutes 25 – 30 % each, Slope slides and leakages cause 13-15 % each, remaining such as damage to slope paving, Unknown etc constitutes 5 – 7 %. It is a fact that the number of earth dams is more than the masonary and gravity dams. The earth dams being constructed with locally available soils, their performance on seepage aspect is relatively poor. Many earth dams require to be rehabilitated so as to prevent the loss of precious water. The overall safety and stability of dams require various aspects to be addressed. The lecture note covers types of soil required for embankment construction, slope stability analysis, dynamic analysis of earth dams and liquefaction potential of foundation against site specific earthquake, remedial measures in case of likely failure and instrumentation required for monitoring the behavior of dam for safe operation etc.

2.0 SUITABILITY OF SOIL FOR CONSTRUCTION OF EMBANKMENT DAMS

The embankment dams are constructed with locally available soil. These dams are suitable for any type of foundation; rock in the foundation is not necessary; height of dam can be raised easily; relatively safe in earthquake areas. The embankment dams are classified as i) Homogenous embankment ii) Zoned embankment iii) Rockfill dams with central clay core, and iv) Rockfill dams with upstream Face membranes.

In **Homogenous embankments** the dam section consists almost entirely of one type of material. This type is adopted due to compulsions of material availability within a reasonable distance at site. Usually this type of section is made of low permeability material and requires flatter slopes than a zoned section.

Zoned embankment uses two or more types of materials, depending on their availability, utility and costs. There is an impervious zone called the Core inside the section (Fig 1). The outer zones on both sides, called casing are of pervious materials. These zones are separated by filters. If the casing material is not pervious enough, it may still be necessary to provide internal drainage. Zoned dams have greater stability during rapid drawdown. They are suitable for large heights.

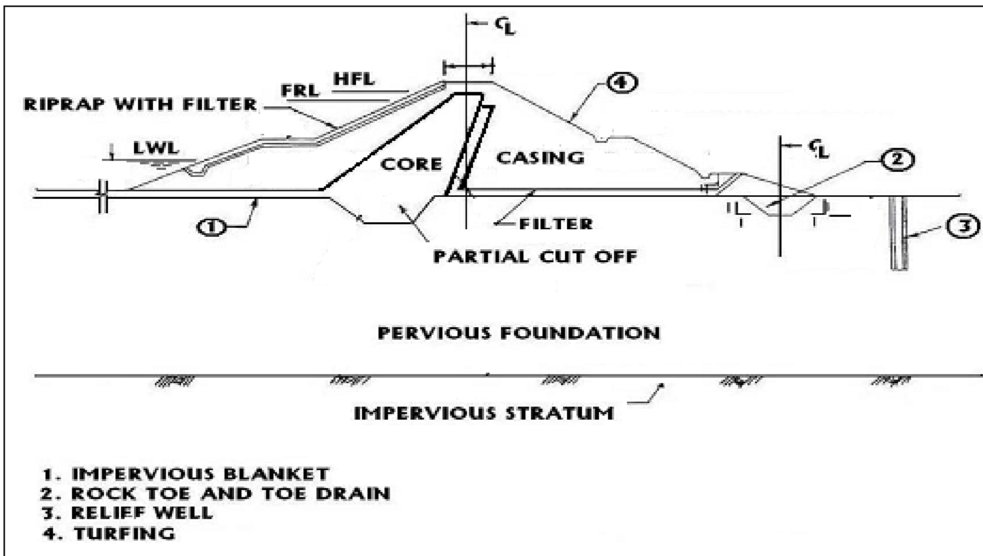

Fig. 1 : Components of Zoned Earth Dam

Rock fill dams with central core, have rock fill zones on both sides, with an impervious zone in the middle, and transition zones and/or filters in-between. Rock fills are fragmented rock, either natural boulder deposits. Good quality rock fill provides free drainage and high shear strength. **Rockfill dams with upstream face membrane** have concrete on upstream slope of the rockfill dam function as water barrier.

2.1 Types of Soil for Embankment Construction

The different components of the embankment dams are i) Core ii) Casing iii) Cut-off, iv) Internal Drainage system and foundations v) Slope protection vi) Surface drainage vii) Impervious blanket viii) Relief well etc. Suitability of soils for construction of earth dams as per Indian Standard is shown in Table 1.

Table -1 Suitability of soils for construction of earth dam:

Relative suitability	Homogenous Dykes	Zoned earth dam		Impervious Blanket
		Impervious	Previous	
Very suitable	Gravelly Clay **(GC)**	**GC**	Well graded Sand or Gravel **(SW,GW)**	**GC**
Suitable	Intermediate & Low Plasticity Clay **(CI,CL)**	**CL,CI**	Silty Gravel **(GM)**	**CL,CI**
Fairly suitable	Poorly graded sand **(SP),** Silty sand **(SM),** High plasticity Clay **(CH)**	**OM,OC, SM,** Clayey sand **(SC) ,CH**	**SP,OP**	**CH,SM, SC,OC**

Silty soil such as Ml,MH, ML and all soil containing organic contents are not suitable.

2.1.1 Core

The core provides impermeable barrier within the body of the dam. Impervious soils are generally suitable for the core. However, soils having high compressibility, high liquid limit and having organic content are avoided. They are prone to swelling and formation of cracks. The core may be located either centrally or inclined upstream. The locations will depend mainly on the availability of materials, topography of site, foundation conditions, diversion consideration, etc.

The main advantage of a central core is that it provides higher pressure at the contact between the core and foundation reducing the possibility of leakage and piping. On the other hand, inclined core reduce the pore pressure in the downstream part of the dam and thereby increases its safety. The section with an inclined core allows the use of restively large volume of random material on the downstream side. The practical considerations that govern the thickness of the core are i) Availability of suitable impervious material ii) Resistance to piping iii) Permissible seepage through dam iv) Availability of other materials for casing, filter etc v) Minimum width that will permit proper construction. The top level of the core generally should be fixed at 0.5 m above design MWL.

Suitability of soils for construction of core in earthquake zones depends upon particle size gradation, plasticity of the clay etc. The type of soil which is a well graded, coarse mixtures of sand, gravel and fines having $D_{85} > 60$mm ,$D_{50} > 8$mm and cohesionless fines (0.075 mm) < 20%, are **most suitable** as core. The soil is considered **good suitability** for core if a well graded mixture of sand, gravel and clayey fines with $D_{85} > 25$ mm and having plastic index fines > 12. The soil is graded **fairly suitable** as core when fairly well graded , gravelly, medium to coarse sand with cohesion less fines having $D_{85} > 19$mm , D_{50} between 0.5mm and 3.0mm, fines < 25% and PI > 25. Clay of low plasticity (PI = 5 to 8, LL > 25) or Silts of medium to high plasticity (PI > 10) with little coarse fraction, are **poorly suitable** as core. Finally, the soil that are **not to be used** as core are the Fine, uniform, cohesion less silty sand having $D_{85} < 0.3$mm. The expensive, dispersive soil shall not be used in the construction of the embankment dam.

2.1.2 Casing

The function of casing is to impart structural stability of the dam and protect the core. The relatively pervious materials which are not subjected to cracking on direct exposure to the atmosphere are suitable for casing. Upstream slope protection measures are Hand place riprap, Dumped riprap, Cement concrete facing etc. Downstream slope protection measures are Turfing, Stone pitching, Network for open paved drains, Geonet etc

2.1.3 Cut-Off

It is used to reduce loss of stored water through foundations and abutments and thereby prevent subsurface erosion by piping. Cut off may be in the form of sheet piling, cement bound curtain, diaphragm wall of bentonite, concrete of other impervious materials. The type of cut-off is decided based on the detailed geological investigation. A cutoff which completely penetrates and seals the pervious foundation strata is called **positive cutoff**. It should be keyed at least to a depth of 400 mm into continuous impervious sub stratum. The alignment of the cut-off should be fixed in such a way that it's central line should be within the base of the impervious core. Minimum width of COT is 4 m for 10 to 30 % of Head.

Where it is not possible to provide positive cut-off, partial cut-off with or without upstream impervious blanket may be provided. The partial cut off is specially suited for the horizontally stratified foundation with relatively more pervious layer near top. The depth of the partial cut-off in deep pervious alluvium will be governed by i) Permeability of substrata, ii) Relative economic of depth of excavation governed usually by cost of dewatering iii) various length of upstream impervious blanket.

2.1.4 Internal Drainage System

Internal drainage system comprises of i) inclined or vertical filter, ii) a horizontal filter, iii) a rock toe and iv) toe drain. The design of filter takes into account the particle size distribution and the shape of the sand/gravel. Moreover, the stability of the base soil adjacent to a given filter depends on its resistance to drag forces. Inclined or vertical filter is to be provided as specially to protect the core material from migration. Filters also help in controlling the leak through crack in the core. A transition filter zone between the core and the downstream shell would be necessary in case of rock fill dams. It is required to provide adequate toe protection.

2.1.5 Slope Protection

The upstream slope protection is ensured by providing riprap. The riprap can be placed on the slope either by hand or simply dumped. The thickness of riprap is not less than 300 mm. The downstream slope protection is ensured by providing riprap or turfing. If the annual rainfall is less than 200 cm, it is usual practice to protect the downstream slope from rain cuts by providing suitable turfing on the entire slope. In case if annual rainfall is more than 200 cm, a riprap of 300 mm thick is provided. Details of downstream slope protection such as prevention from erosion by Rain-wash, Prevention from erosion by tail water etc. are given in IS:8237-1985.

2.1.6 Impervious Blanket

The horizontal upstream impervious blanket is provided to increase the path of seepage, when full cut-off is not practicable on pervious foundations. It may be provided either with or without partial cut-off. It shall be connected to core of the dam. Its permeability should be far less than the foundation soil. The use of soil with high plasticity for blanket will lead to formation of cracks. In order to prevent cracking due to exposure to atmosphere, spreading of a layer of random material of 300 mm thick, over the blanket is recommended. The details of design of blanket and suitability of soils for its construction are given in IS: 8414 and IS 1498 respectively. The general design guideline is to have a blanket of minimum thickness of 1.0 m and a minimum length which is 5 times the maximum water head.

2.1.7 Relief Wells

It is used to reduce the pore pressure developed in the foundation. It consists of small drainage well (45 to 90 cm in dia) sunk near the downstream toe of an earth dam. It has a slotted pipe (about 10 to 15 cm in dia) placed in the centre which is surrounded by graded sand filter media. This arrangement permits the ingress of seepage water into the well, allowing it to rise to the outfall (relief) level, where the pressure gets relieved. They are installed to ensure safely in cases where seepage control is depended only on partial cut-off or upstream blanket. Its use is imperative if top strata is impervious followed by pervious strata. A system of relief wells suitably spaced are installed, to reduce the intensity of the under seepage pressure. The seepage water is safely and conveniently led to a natural drainage channel.

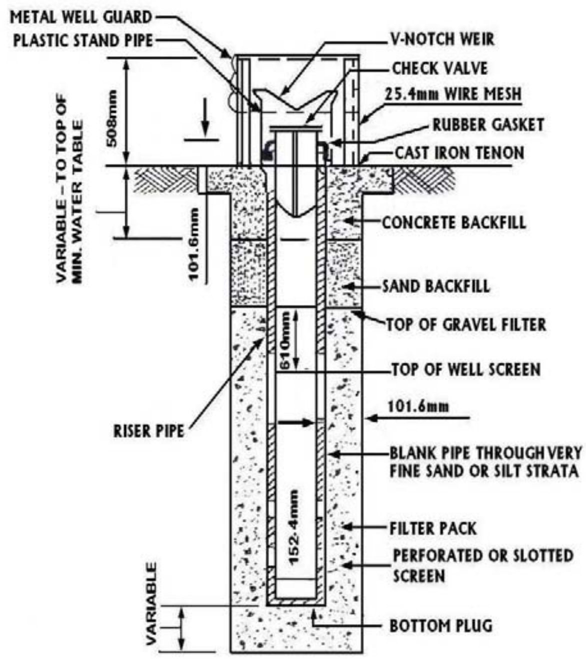

Fig. 2 Typical Relief Well

3.0 SAFETY REQUIREMENTS

The basic requirements for design of embankment dam are to ensure Safety against overtopping, Slope stability, Safety against internal erosion, Phreatic line within downstream face, Safety against wave action.

3.1 Safety Against Overtopping

Sufficient spillway and outlet capacity should be provided to prevent overtopping of earth embankment during and after construction. The freeboard should be sufficient to prevent overtopping by waves and should take into account the settlement of embankment and foundation. In case of unyielding foundation, the amount of settlement for the embankment should be restricted to 1.0 percent of the height of dam. In case of compressible foundation, the settlement should be computed based on laboratory test results and same should be provided by increasing the height of dam correspondingly. Longitudinal camber should be provided on the top of dam along the dam axis to provide for settlement. The camber varies from zero height at the abutments to maximum at the central section in the valley where maximum settlement is anticipated. Guidelines for free board requirements in

Embankment dams are explained in IS:10635-1993. There should be no risk of over topping of the dam section. The most important aspect of this criteria is estimation of the design flood and provision of adequate spillway capacity to pass that flood with required net freeboard to protect the dam crest against wave splash.

3.2 Safety Against Slope Stability

The purpose of slope stability analysis is to provide a quantitative measure of the stability. The slopes of the embankment should be stable under all loading conditions. It is expressed as the Factor of Safety (FS) against failure. It is defined as the ratio of Restoring forces to the Disturbing forces, for an assumed potential failure surface. FS for several failure surfaces are computed and the minimum value of FS is considered for stability. The value of FS less than 1.0 indicates failure of the slope. There are different analytical methods available to carry out stability of earth / rockfill dams such as Swedish slip circle, Bishop's Simplified method, Spencer method, Janbu's rigorous method, Morgensten-Price, etc. These methods are called limit equilibrium methods and do not give the deformation of the failed slope. Now a days, Numerical methods software are available which give FS as well as the likely deformation of the failed slope. Embankment slopes are to be designed in accordance with the provisions contained in IS:7894-1975.

The critical issues in the analysis are i) potential failure mechanism, ii) geometry of sliding, iii) pore pressure and iv) shear strength of soils. The dams have to be assessed for following conditions :

i. Downstream slope for steady state seepage
ii. Upstream slope for rapid drawdown
iii. Construction condition for both side slopes.
iv. Earthquake loading

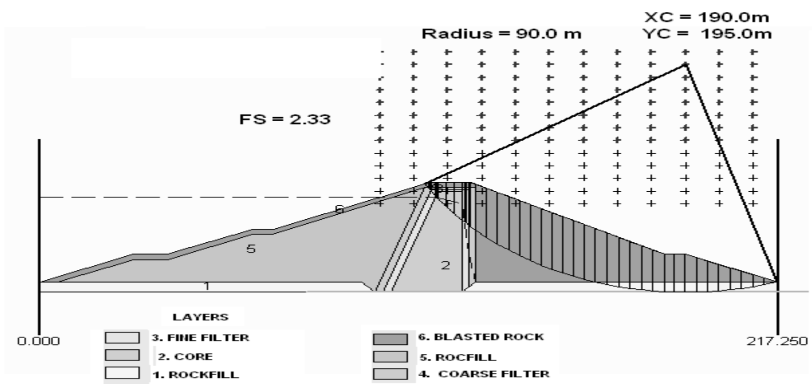

Fig. 3: Slope Stability Analysis by Slip Circle Method

3.3 Safety Against Internal Erosion

The seepage through the embankment and foundation should be such as to control piping, erosion and sloughing and excessive loss of water. Seepage control measures are required to control seepage through dam and seepage through foundation. Design for control of seepage through dam shall be made in accordance with provisions contained in 'Indian Standard drainage systems of earth and rock fill dams.' Design for control of seepage through foundation may be made in accordance with provisions contained in IS: 8414-1977.

The phreatic/ seepage line should be well within the downstream face of the dam section. If the dam section is homogeneous and no drainage arrangements are made, any seepage is going to emerge on the downstream face. This results in "sloughing" or softening of the downstream face and may lead to local toe failure, which may progressively develop upwards. This can be safeguarded against by providing a free draining zone on the downstream face or by intercepting the seepage inside the dam section by internal drainage.

4.0 DYNAMIC ANALYSIS OF EARTH DAM

Deformation of earth dam due to an site specific earthquake is carried out by the method proposed Prof.B.Seed and F Makdisi, which is based on the concept of Prof. Newmark. The method assumes that failure occurs on a well defined slip surface and that the material behaves elastically at stress levels below failure but develops a perfectly plastic behavior above yield. Seed and Makdisi carried out finite element dynamic analysis on selected earthen dams and they prepared empirical relations in the form design chart. The earthquake induced deformation can be determined by using these charts. The steps involved in the method are given below:

I. Conduct **slope stability analysis** with different of horizontal seismic coefficient to determine **Yield acceleration (K_y).**

II. Conduct **Dynamic Response Analysis** of the dam for a given Maximum credible earthquake (MCE) by finite element method, to determine **Maximum Crest acceleration ($\ddot{U}_{max}$)**. Using this, evaluate **acceleration of the Sliding mass (K_{max})** from first empirical relation.

III. Using values of K_{max} and K_y, evaluate the **crest displacement (U)** for different magnitudes of Earthquakes, from second empirical relation.

4.1 Yield Acceleration (K_y)

The yield acceleration, K_y, is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass and causes it to experience permanent displacement. It is determined from slope stability analysis with different values of horizontal acceleration. The value of horizontal acceleration for which FoS reduces to 1.0, is called **Yield acceleration.** It is the threshold limit between stability and instability.

4.2 Maximum Crest Acceleration (Ü)

Earth / Rockfill dam section is idealized by Finite Element mesh as shown in Fig 4. Mesh contains 863 nodes and 840 elements. The dynamic properties of the soil such as Shear modulus (G) and Damping ratio (D), which are shear strain (γ) dependent, are determined from **Resonant column test** in laboratory. These non-linear properties are shown in Fig 5. Site specific earthquake in the form of acceleration-time history is applied to the base of the dam. Fig. 6 gives the Acceleration – time history of Maximum Credible Earthquake (MCE) record. Dynamic response analysis of the dam is carried out using computer software such as QUAD-4, FLAC-2D etc. The analysis solves the dynamic equation of motion:

$$\mathbf{M\ddot{x} + D\dot{x} + Kx = F(t)} \qquad \textbf{(1)}$$

Where,

M, D and K	Mass, Damping and Stiffness matrix of assembly of the dam
F(t)	Site specific earthquake
$\ddot{x}$, $\dot{x}$, x	Nodal acceleration, nodal velocity and nodal displacement

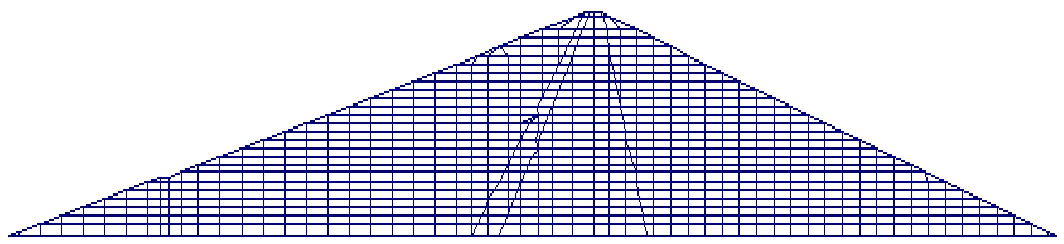

Fig. 4 Finite Element Idealization of Rockfill Dam Section

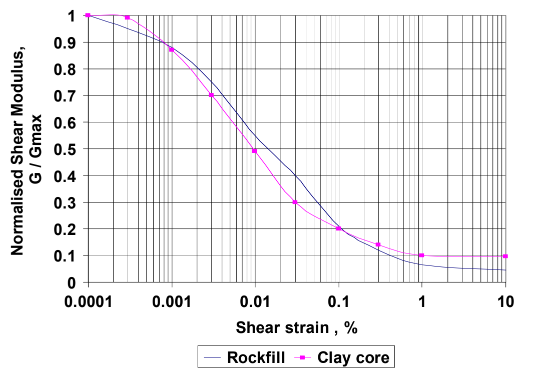

Fig. 5a : Shear Modulus Vs Shear Strain

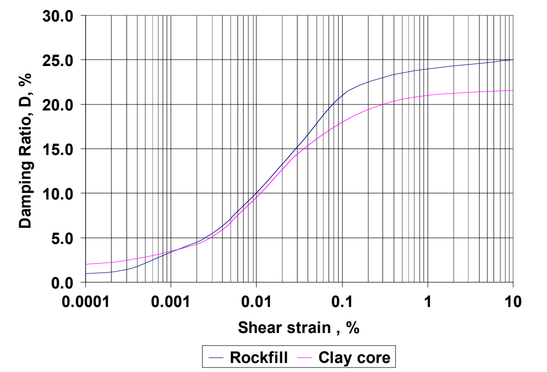

Fig. 5b : Damping Ratio Vs Shear Strain

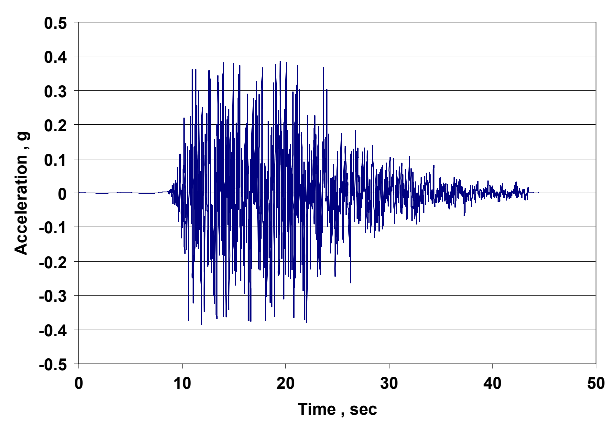

Fig. 6 : Horizontal Acceleration-Time History of Maximum Credible Earthquake

The dynamic response analysis gives maximum horizontal and vertical acceleration that are likely to be induced at each nodal point. The Fundamental period (T_o) of the dam was found to be 1.33 sec. The first design curve of Prof. Seed is shown in Figure 7.

4.3 Crest Displacement (U)

Prof. Seed's second design curve is shown in Figure 8. This curve gives variation of the ratio **(K_y / K_{max})** with 'Normalized permanent displacement' (U / K_{max} .g .T_o) for different magnitudes of earthquakes (6 ¼,7 ½ and 8 ¼), where **'U'** is the actual displacement. It is mentioned that permanent displacement of the sliding mass occur only if the ratio **(K_y / K_{max})** is less than 1.0. For a given ratio of (K_y/K_{max}), normalized displacement can be found out for a given magnitude of earthquake. Actual crest displacement (U) of the sliding mass that is likely to undergo is determined from the value normalized displacement.

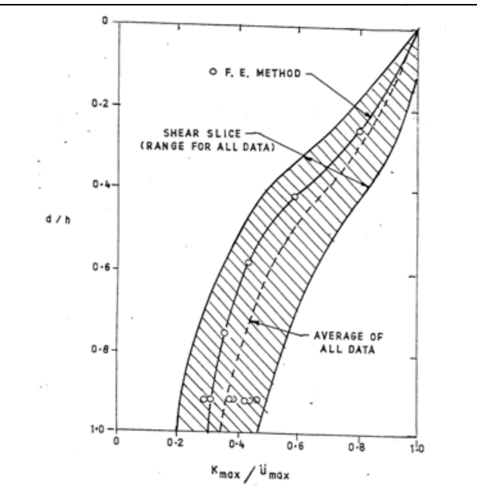

Fig 7 : Seed's Empirical Curve-1

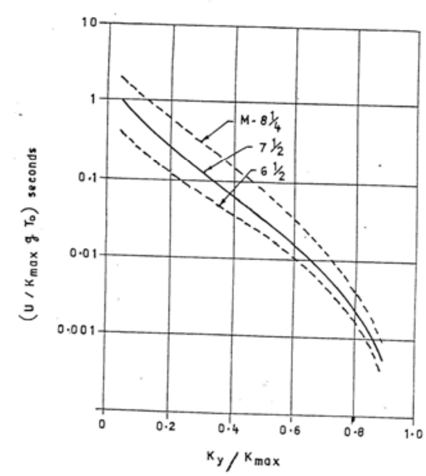

Fig 8 : Seed's Empirical Curve-2

5.0 FOUNDATION LIQUEFACTION

Liquefaction is a phenomenon in which a saturated silty-sand foundation stratum behaves like a liquid when subjected to an earthquake. In liquefaction, the superstructure sinks in to the ground because of reduction in the effective stress of the foundation. Many structures had been damaged due to liquefaction during Bhuj earthquake. The pore water pressure increase in foundation soil during the earthquake causes reduction of effective stress. The liquefaction resistance of an element of soil depends on how close the initial state of the soil is to the state corresponding to "failure" and on the nature of the loading required to move it from the initial state to the failure state. Characterization of liquefaction resistance developed along two lines: methods based on the results of laboratory tests, and methods based on in situ tests, such as Standard Penetration tests, Cone Penetration test, Shear wave velocity etc.

Steps for determining the Zone of liquefaction in the field from laboratory tests are as follows:

i. Design earthquake is established
ii. Shear stress time history induced at various depths in the foundation are determined from the dynamic response analysis for the given design basis earthquake.
iii. The shear stress time histories are converted to number of equivalent cycles.(N)
iv. Using laboratory tests, determine the magnitude of the cyclic shear stress required to cause initial liquefaction in the field in N cycles at various depths. These can be plotted with depth.

The zone in which the cyclic shear stress levels required to cause initial liquefaction are equal or less than the equivalent cyclic shear stresses in induced by an earthquake is the zone of possible liquefaction.

5.1 Liquefaction from Laboratory Tests

Cyclic shear stress will be imposed on the soil element due to ground shaking. This situation is simulated in laboratory tests. The most commonly used laboratory test procedures are Cyclic
triaxial shear test and Simple shear test.

Cyclic triaxial tests or simple shear tests are conducted to determine liquefaction failure in sand. The liquefaction failure is defined as the point at which the pore pressure reaches the confining pressure or cyclic strain amplitude (5 %) is reached. The number of loading cycles required to produce liquefaction failure (N_L) decreases with increasing shear stress amplitude and with decreasing density. Fig. 9 shows Test record of cyclic shear tests and cyclic deviator stress versus Number of loading cycles under confining pressure.

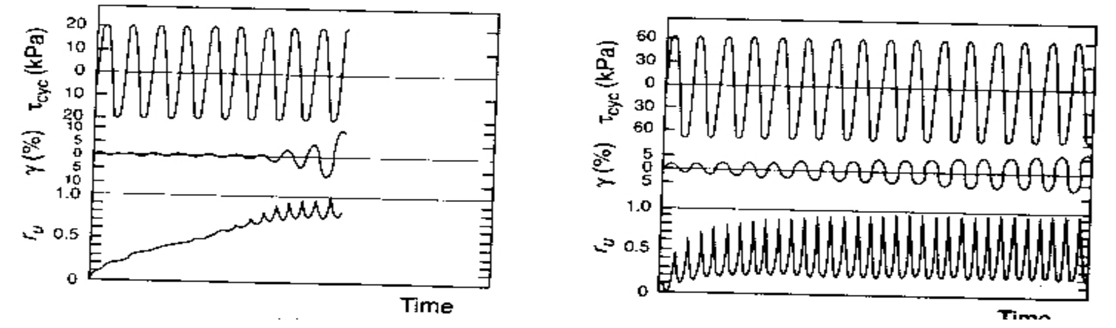

Fig. 9: Results of Cyclic Shear Tests

A. Loose sand wit 47% relative density, liquefies at 10th loading cycle
B. Dense sand with 75% relative density with high cyclic shear stress not liquefied.

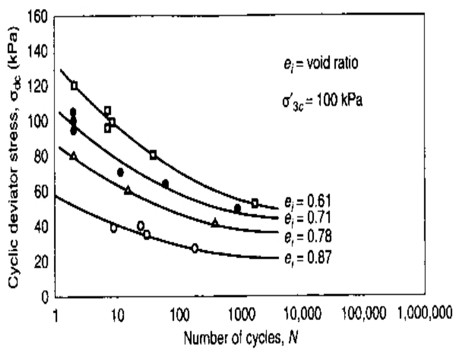

Fig. 10 : Cyclic Deviator Stresses versus Number of Cycles

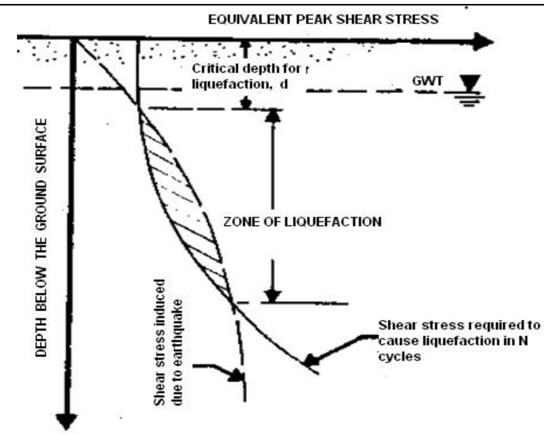

Fig. 11 : Zone of Liquefaction in Field

Cyclic Stress Ratio (CSR), is the cyclic strength of soil normalized by the initial effective overburden pressure, for different void ratio. (Fig. 10)

5.2 Liquefaction from Field Tests

Liquefaction of foundation is determined from Simplified Procedure proposed by Seed and Idriss (1971). As the seismic loading is excited at the base of the soil column, the shear wave propagates to the ground surface and shear stress is generated in the soil column. The shear stress time history during earthquake is random in nature. An average of shear stress (τ_{ave}) is normalized with the initial effective overburden pressure (σ_0'), is called Cyclic Stress Ratio (CSR) and is given as: $\boldsymbol{CSR = \frac{\tau_{ave}}{\sigma_0'} = 0.65 * \frac{\sigma_0}{\sigma_0'} * \frac{a_{\max}}{g} * r_d}$

(2)

τ_{av}	=	maximum shear stress for rigid body
σ_0	=	total overburden pressure
a_{max}	=	peak horizontal acceleration on the ground surface
g	=	acceleration of gravity
r_d	=	the stress reduction coefficient (1.0 at the top and 0.9 at 15.0m below)

The CSR is the seismic demand of a soil layer and determined for each depth.

Next, the capacity of the soil to resist liquefaction, called Cyclic Resistance Ratio (CRR) is determined. It is computed from Penetration resistance value (N_m = blow counts per 300mm of penetration), of Standard Penetration Test (SPT). Procedures are also available based on Cone Penetration Tests (CPT), shear wave velocity measurement, and Becker Penetration Test (Youd, et al., 2001).

The SPT blow counts, N_m, are corrected. The correction factors are applied for Overburden (C_N), hammer energy (C_E), bore hole diameter (C_B), rod length (C_R) and sampler size (C_S). The corrected SPT blow count $(N1)_{60}$ is determined as follows:

$$\left(\mathbf{N_1}\right)_{60} = \mathbf{N_m\ C_N\ C_E\ C_B\ C_R\ C_S} \tag{3}$$

The value of SPT blow counts for soil with fines content can be adjusted to the equivalent clean sand value of $(N_1)_{60CS}$. This can be done by applying constants, α and β, that are functions of fines content. Hence, the effect of fines content (FC) on the value of CRR is included as

$$\left(\mathbf{N_1}\right)_{60CS} = \boldsymbol{\alpha} + \boldsymbol{\beta}\left(\mathbf{N_1}\right)_{60} \tag{4}$$

Where α and β can be determined for Fines Content (percent of soil having size < 0.075 mm) in the soil.

Prof. Seed's base curve for CRR versus SPT is shown in Fig 12. This curve is for earthquake with magnitude of 7.5. For other earthquake magnitudes, a magnitude-scaling factor (MSF) should be applied.

The Simplified Procedure applies for level to gently slope sites and for depths less than 15 meters. Therefore, the value of CRR should be corrected for greater depths that are for high overburden stresses. The K_σ is equal to unity for effective overburden pressure less than 1 tsf and then decreases with increasing effective overburden pressure.

$$\mathbf{CRR = CRR_{M=7.5} * MSF * K_\sigma} \qquad \textbf{(5)}$$

The Factor of safety against liquefaction can be written as :

$$\mathbf{FS_L = \frac{CRR}{CSR}} \qquad \textbf{(6)}$$

The value of $FS_L < 1.0$ indicates occurrence of liquefaction and $FS_L >= 1.0$ indicates occurrence of 'No Liquefaction'.

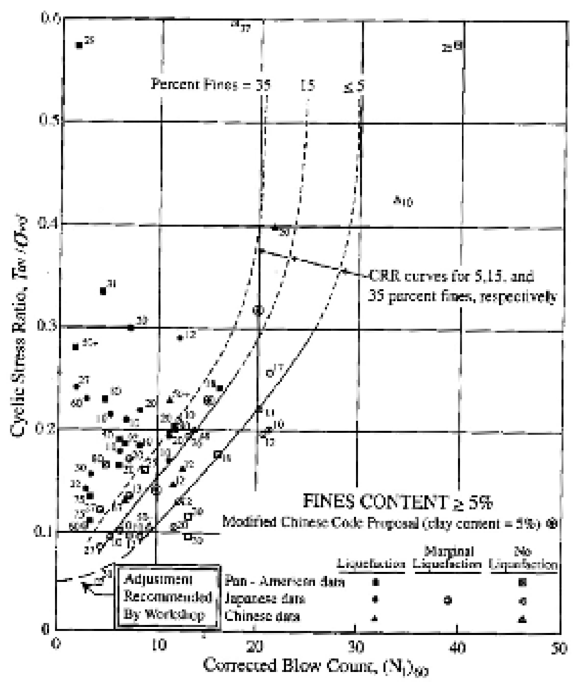

Fig. 12 Simplified Base Curve for CRR from SPT data(Seed 1985)

6.0 INSTRUMENTATION OF EMBANKMENT DAMS

All embankment dams should have an adequate level of instrumentation to enable design engineers to monitor and evaluate the safe performance of the structures during the construction period and under all operating conditions. This includes all appurtenant structures and facilities whose failure or malfunction would cause or contribute to loss of life, severe property damage, or loss of function or interruption of authorized mission. Instrumentation is not a substitute for an inadequate design. It is a tool to monitor and verify the performance of the design as constructed.

In view of concerns for dam safety, it has become increasingly important to provide sufficient instrumentation in earth and rock-fill dams for monitoring the performance of the structure during construction, and for all anticipated stress conditions throughout the operational life of the project. Visual observations and the interpretation of instrumentation data from the embankment, foundations, abutments, and appurtenant features provide the primary means for engineers to evaluate dam safety. In recent years, technology of devices for measuring seepage, stresses, and movements in dams has improved significantly with respect to accuracy, reliability, and economics. These technologies should be used to the extent necessary to acquire sufficient information within the required timeframe to assure the thorough understanding of dam performance.

6.1 Instrumentation Plan and Records

The planning, design, and layout of an instrumentation program are integral parts of the project design. Instrument data are an extremely valuable asset that supplies an insight into the actual behavior of the structure relative to design intent for all operating conditions, establishes performance that is uniquely characteristic to the dam, and provides a basis for predicting future behavior. As structures age and new design criteria are developed, the historical data provide most of the information necessary to evaluate the safety of the structure with respect to current standards and criteria. Older structures may require additional instrumentation to gain a satisfactory level of confidence in assessing safe performance. Instrument data can be of benefit only if the instruments consistently function reliably and the data values are compared to the documented design limits and historical behavior. Automation of dam safety instrumentation is a proven, reliable approach to obtaining instrument data and other related condition information.

6.1.1 System Design

The design and construction of new projects as well as the rehabilitation, dam safety modifications, and normal maintenance of older projects present opportunities to prepare for the future engineering analyses of structural performance. As a minimum, the parameters that are critical to satisfactory performance will dictate the selection of instrument types. Generally, the types of measurements are (1) Horizontal and vertical movement, (2) Alignment and tilt, (3) Stresses and strains in soil and rock fill, (4) Pore pressure, (5) Uplift pressure, (6) Phreatic surfaces, (7) Seepage clarity and quantity.

In all circumstances, background information that may affect the validity of the data or the analysis of the performance (such as hydrologic or weather conditions) is documented and baseline instrument data for each type of measurement is obtained for future comparison.

6.1.2 Installation and Maintenance

Instrumentation for a project should be included in the design phase, during construction, and throughout the operational life of the project as conditions warrant. After a project has been operational for several years, appropriate maintenance, repair, and replacement of instrumentation must be accomplished during the normal operation to assure continued data acquisition and analyses of critical performance parameters. Specialized expertise are required to install and maintain automated instrumentation.

6.1.3 Data Collection, Interpretation, and Evaluation

The frequency with which instrumentation data are obtained must be tailored to the monitoring purpose, period of construction, investigation, or other interest, and project operating conditions. In all cases, sufficient calibration must be performed and background data must be obtained to ensure that a valid and reliable database is developed, maintained, and available to facilitate subsequent comparisons. After a baseline of performance is established, the subsequent reading of instruments during construction and operating conditions should be based on an anticipated rate of loading or changes in reservoir levels. The evaluation of the data should follow immediately. As a minimum, all data should be plotted as instrument response with respect to time, as well as reservoir level or other range of loading.

6.1.4 Documentation

Information relative to instrumentation systems is an invaluable resource that is necessary to evaluate instrument and system performance, as well as influence the assessment of dam performance and should be preserved and readily accessible.

6.2 Types of Instrumentation

The type, number, and location of required instrumentation depend on the layout of the project and the construction techniques employed. Devices may consist of the following: i) piezometers (open tube, Casagrande type, electrical, vibrating wire, or occasionally closed systems) located in the foundation abutment and/or embankment, ii) surface monuments, iii) settlement plates within the embankment, iv) inclinometers, movement indicators (at conduit joints, outlet works, and intake tower), v) internal vertical and horizontal movement and strain indicators, vi) earth pressure cells, and vii) accelerographs (in areas of seismic activity).

6.2.1 Piezometers

The safety of a dam is affected by hydrostatic pressures that develop in the embankment, foundation, and abutments. Periodic piezometer observations furnish data on porewater pressures within the embankment, foundation, and abutments, which indicate the characteristics of seepage conditions, effectiveness of seepage cutoff, and the performance of the drainage system. At each cross section that piezometers are placed, some should extend into the foundation and abutments and be located at intervals between the upstream toe and the downstream toe, as well as being placed at selected depths in the embankment. Two of the more important items in piezometer installation are the provision of a proper seal above the screen tip and the water tightness of the joints and connections of the riser pipe or leads.

6.2.2 Surface Monuments

Permanent surface monuments to measure both vertical and horizontal alignment should be placed in the crest of the dam and on the upstream and downstream slopes. Monuments should be embedded in the embankment by means of a brass or steel rod encased in concrete to a depth regionally appropriate to avoid frost action. Guidance on spacing is as follows: 50-ft intervals for crest lengths up to 500 ft, l00-ft intervals for crest lengths to 1,000 ft, and 200- to 400-ft intervals for longer embankments. These monuments should be installed as early as possible during construction and readings obtained on a regular basis.

6.2.3 Inclinometers

Inclinometers should be installed in one or more cross sections of high dams, dams on weak deformable foundations, and dams composed at least in part of relatively wet, fine-grained soils. Inclinometers should be installed particularly where dams are located in deep and narrow valleys where embankment movements are both parallel and perpendicular to the dam axis. Inclinometers should span the suspected zone of concern. It is essential that these devices be installed and observed during construction as well as during the operational life of the project.

6.2.4. Movement Indicators

Various types of instrumentation may be installed to measure horizontal spreading of the embankment (particularly when the foundation is compressible), movements adjacent to buried structures, foundation settlement, and internal strains. Strain measurements are particularly significant adjacent to abutments and below the crest to detect cracking of the core. Where there is a possibility of axial extension, as near steep abutments, surface monuments should be placed on the crest at 10.0 m intervals to permit measurement of deformations along the axis.

6.2.5 Pressure Cells

The need for reliable pressure cells for measuring earth pressures in embankments has long been recognized, and much research has been done toward their development. Although many pressure cells now installed in earth dams have not proved to be entirely satisfactory, newer types are proving to be satisfactory and increased usage is recommended. Some types of pressure cells installed at the interface of concrete structures and earth fill have performed very well.

6.2.6 Accelerographs

It is desirable to install strong motion, self-triggering recording accelerographs to record the response of the dam to the earthquake motion. Digital accelerographs are recommended. The digital units record and provide fundamental event information on a near real-time basis and should be incorporated into dam safety monitoring programs.

6.2.7 Weirs for Seepage Measurements

The seepage flow through and under a dam produces both surface and subsurface flow downstream from the dam. The portion of the total seepage that emerges from the ground, or is discharged from drains in the dam, its foundation, or abutments, is the only part that can be measured directly. An estimate of the quantity of subsurface flow from flow net studies may be based on assumed values of permeability. The portion of the seepage that appears at the ground surface may be collected by ditches or pipe drains and measured by means of weirs or other devices.

6.3 Automated Data Acquisition Systems

It is possible to install and operate automated instrumentation systems that provide cost-effective real-time data collection from earth and rockfill dams. Installation of these computer-based automated data acquisition systems (ADAS) provides for more accurate and timely acquisition, reduction, processing, and presentation of instrumentation data for review and evaluation by geotechnical engineers. Consideration should be given to providing an ADAS for all new dam projects, dam safety modifications to existing dams, and monitoring system rehabilitation that are necessary to assure appropriate data acquisition.

7.0 REMEDIAL MEASURES / REHABILITATION

In spite of construction of dam as per the codal practices, the actual behavior of the dam differs during its operation due to uncertain / unexpected loads such as earthquake. It is true that using existing engineering technology, engineers today are capable of designing and constructing new dams that will behave acceptably during the design earthquake. If, for example, there are loose alluvial sands in the foundation, you simply remove them. On the other hand, engineers are faced with existing dams, founded on alluvial material that is potentially liquefiable. It is the seismic retrofit of these existing dams that are the key concern facing geotechnical earthquake engineers. In order to rehabilitate a deficient embankment dam to prevent potential seismic instability, one must either change the engineering properties of the dam and/or foundation; modify the geometry of the existing dam, or both. If predicted permanent deformations are estimated to be small and tolerable, then the dam is safe. On the other hand, if the deformations are intolerable and the dam is not to be taken out of service, then seismic remediation are required. There are a wide variety of treatments for rehabilitating dams are in use.

7.1 Methods of Remediation

The present methods available for engineered remediation of seismically deficient earth dams are explained in the following paragraphs.

7.1.1 Berms and Buttresses

Upstream and downstream berms and buttresses are used to increase the effective overburden pressure on the problem material and thus increase its liquefaction resistance. This increase in overburden also causes a small amount of consolidation and thus increases the density. Berms and buttresses are also used to increase the length of the failure surface, provide a counterweight to limit movement, and maintain a remnant section. The effectiveness of a berm is generally limited to a zone that is about as deep as the berm is thick. A berm or buttress can not reduce the factor of safety during day-to-day operation, and its presence is obviously verifiable. If coarse-grained soil or rock is available, berms and buttresses can, with some difficulty, be constructed on the upstream shell without lowering the pool.

7.1.2 Excavate and Replace

This method assures that the problem material is removed and replaced with a non-liquefiable material. Excavation and replacement offers the advantage of providing relative assurance that what was designed has actually been constructed in the field. It is often expensive and operationally difficult. Dewatering is almost always required and in many cases the reservoir must be lowered significantly. This approach is most useful when the problem material is near the ground surface. In addition to excavating liquefaction prone material, the excavation-and-replace method can also be used after an earthquake to remediate shallow cracking.

7.1.3 In-situ Densification

When excavation and replacement are ruled out for some reason, in-situ densification can sometimes be used to decrease the potential for liquefaction by decreasing the void ratio of the problem material. The method includes vibro-techniques, dynamic compaction, compaction grouting, and displacement techniques. In-situ densification is most effective when the material to be improved is close to the ground surface and has limited fines content. To date this approach has not been used under an existing dam except in cases where most of the embankment over the foundation zone to be densified has been temporarily removed. Foundation densification will not be uniform and could adversely change the dam-foundation interface. Cracking might occur which could increase the risk of piping. Verification of the amount of improvement and of the spatial variability of the improvement is required.

7.1.4 In-situ Strengthening

While somewhat similar to in-situ densification, in-situ strengthening forms a composite material that is strong enough to ensure stability. Soil nailing, stone columns, and methods of deep soil mixing are examples. Some of these methods may also cause consolidation and increase the strength of the soil around the feature, but this increase in strength is generally ignored in stability analyses. In-situ strengthening is generally most effective when the potentially liquefiable material is confined to a relatively thin layer but it can be implemented for thick deposits in the case of deep soil mixing. The method is to be applied with caution, if it is used under an existing structure. Conventional grouting of the foundation through the embankment has not been used for the purpose of strengthening for two reasons. One is the possibility of hydraulic fracture in the embankment. The other is that it is not easy to determine to what extent the grout has penetrated the zones needing improvement.

7.1.5 Increase Freeboard

An increase in freeboard may be used when the seismic analysis indicates that the dam is marginally stable and/or only small earthquake-induced deformations are probable. Obviously, this approach decreases the probability of overtopping associated with settlement or slumping of the crest.

7.1.6 Drainage

This approach provides for relief of seismically induced pore water pressure. Techniques include strip drains, stone columns, and gravel trenches. Gravel trenches can be used to intercept migrating elevated pore pressure plumes. Analysis of drains is problematic , because accurate and reliable in situ permeability are extremely difficult to obtain. Care must be used in placement of stone columns and gravel trenches so that stone crushing is minimized and permeability remains high. Stone column spacings should be sufficiently small to dissipate pore pressures to a low level during the earthquake, and to prevent the occurrence of high hydraulic gradients that could carry large amounts of fines into the gravel drains. Potentially, stone columns could be flushed periodically to maintain their performance. It is inevitable that extra drains added for remediation will reduce the length of flow lines and thus will increase the seepage gradients under static pool conditions. Hence, even if the drains are designed as filters with respect to the adjacent material, the static safety of the dam will be somewhat reduced and more water will filter through the dam.

7.2 Validation of Rehabilitated Dam

Good judgment requires that absolutely nothing be done to increase the likelihood of failure during normal static operating conditions such as increasing the likelihood of failure by piping, or hydraulic fracturing. One must also verify that any intended densification and/or drainage are achieved. Verification of densification is usually achieved by the use of a test section where before-and-after SPT, CPT, and/or shear wave velocity measurements are used to assess the effects of the densification technique attempted. If the technique is found successful in the test section, the same before-and-after index tests are used to verify densification of the improved zone.

8.0 CWPRS EXPERIENCES

CWPRS has been associated in solving cost-effective solutions to the safety, stability and rehabilitation aspects of embankment dams for many projects. One case study is described below. Moreover, a case history of rehabilitation of Chang earth dam after upstream slope failure due to Bhuj earthquake is also described.

8.1 Seepage and Stability Analysis of Dudhawa Earthen Dam

Dudhawa dam comprises a 2.9 km long and 24.7 m high earthen dam across Mahanadi river. The dam had shown signs of distress by way of leakages, sand boils since its first impoundment in 1962. This resulted in under utilization of its full capacity. Temporary remedial measures, such as relief wells and toe loading, were being carried out by the authorities as per the recommendation of the Dam safety Panel. Finite element seepage analysis were carried out for two sections, one at Chainage 40-42 km and another at 83-84 km, based on the prototype piezometric data.

The classification soil from the different zones of the dam are given in the Table 2.

Table 2 : Zones of the Earthen Dam

Sr. No	Zone	Classification as per BIS	Permeability cm/ sec
1	Casing soil	SC	7.06×10^{-6}
2	Core soil	CH	1.0×10^{-8}
3	Foundation soil : Chainage 40 to 42 Km	SP	2.88×10^{-5}
	: Chainage 83 to 84 Km	CH	1.0×10^{-8}

Two dimensional seepage analysis was carried out for the two sections using Finite Element method (FEM) using computer software SOLVIA-TEMP. Figure No.4 shows the finite element idealization on the dam section. Analysis used 8 noded isoparametric elements. The soil permeability values as determined in the laboratory were used in the analysis is shown in Table III.

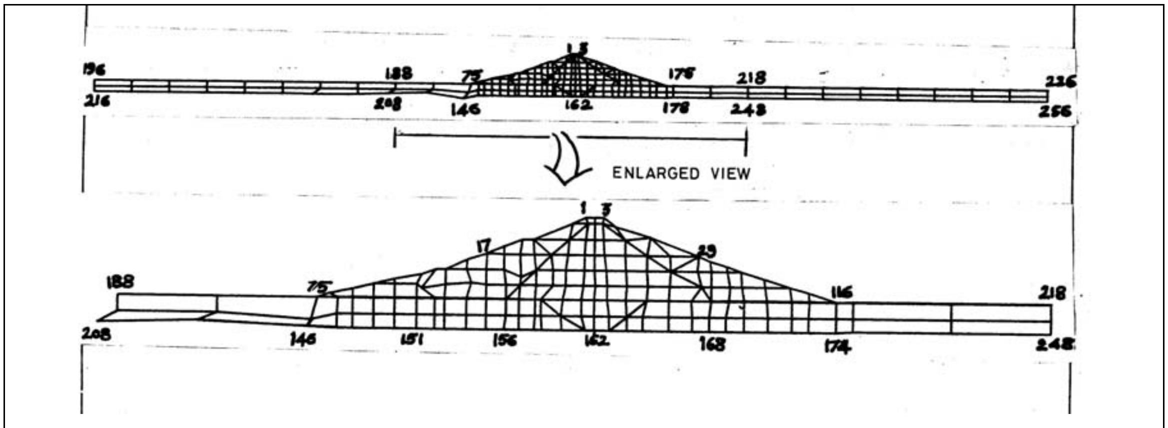

Fig. 13: Finite Element Idealisation of Dam Section at Ch 40-42

Boundary conditions for the analysis were maximum reservoir water level RL 423.76 m), tail water level (or piezometric head at toe). The analysis yielded values of the total head the nodal points and the water pressure values at the gauss points of the elements. Initially, analysis were carried out with considering the freely draining filter zone. The results of the analysis, however, indicated that the pressure heads on the down stream slope did not match with the actual observed piezometric data. Analysis was repeated with different lower values of the permeability of the filter/rock toe (i.e. not freely draining filter) such that the total head values of the analysis matched with the actual observed piezometric data. The contours of the pressure lines were drawn. The phreatic line, which is the zero pressure line, was identified from the pressure lines. It was observed that the phreatic line emerged close to the down stream slope of the earthen dam, which was confirmed from the field observations. These pressure lines data were used as input subsequently in the slope stability analysis (STABR).

Slope stability analysis using Bishop's modified method was used to check the Factor of safety (FoS) of the slope of the dam. The program has a provision for incorporating the equipressure lines in the analysis. The equipressure lines as evaluated from the seepage analysis was used in this analysis. The FoS of 1.26 and 1.12 were evaluated for the sections at Ch 40-42 and Ch 83-83 respectively. Figure No. 6 shows critical slip circle for the section. These values of FoS are well below the FoS of 1.5 recommended by BIS. The high pore pressure in the downstream portion of the dam is due to the fact that the filter and rock toe do not function as required. This has resulted in the lower FoS in the stability analysis.

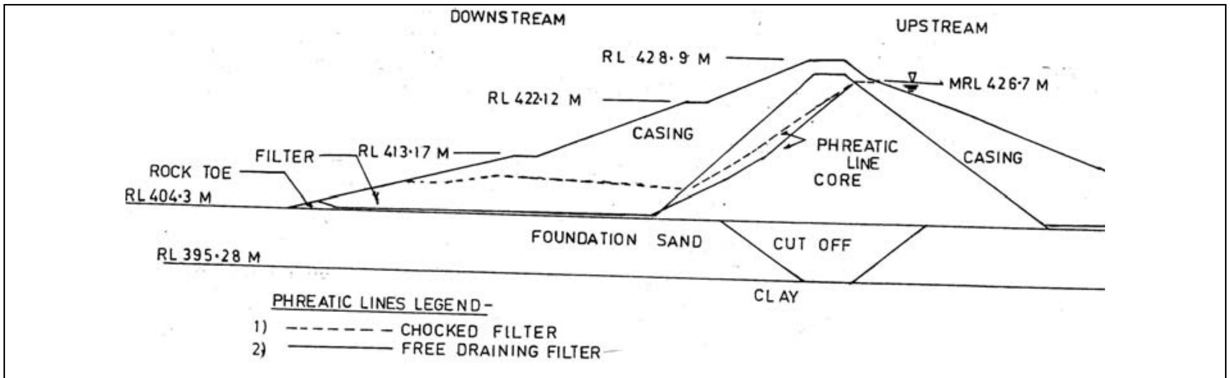

Fig. 14 : Phreatic Line for Free Draining and Chocked Filter from Seepage Analysis for Ch 40-42

Remedial measures in the form of rubble fill loading on the downstream slope from RL 404.42m to RL 413.4 m for section between chainage 40-42 and from RL 411.50 m to RL 417.5m for section between Ch 83-84. The thickness of 1.5m rubble fill was computed from the exit hydraulic gradient from the seepage analysis. The large berm will counter balance the actuating forces and increase stabilizing forces and provide adequate overburden to improve stabilizing effective normal stresses. Figure No.7 shows the proposed remedial measures. A properly constructed drainage system must be provided below the loading berm to let all the seepage flow safely to the downstream.

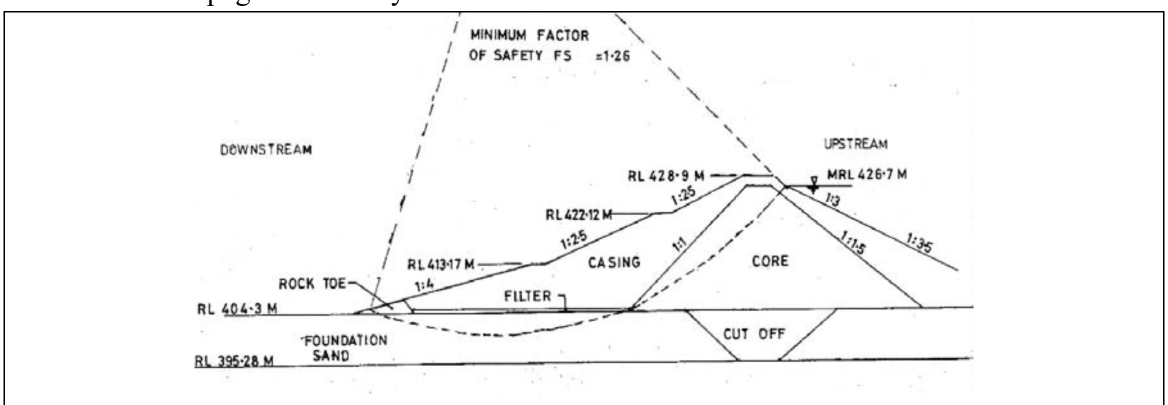

Fig. 15: Stability Analysis Ch 40-42

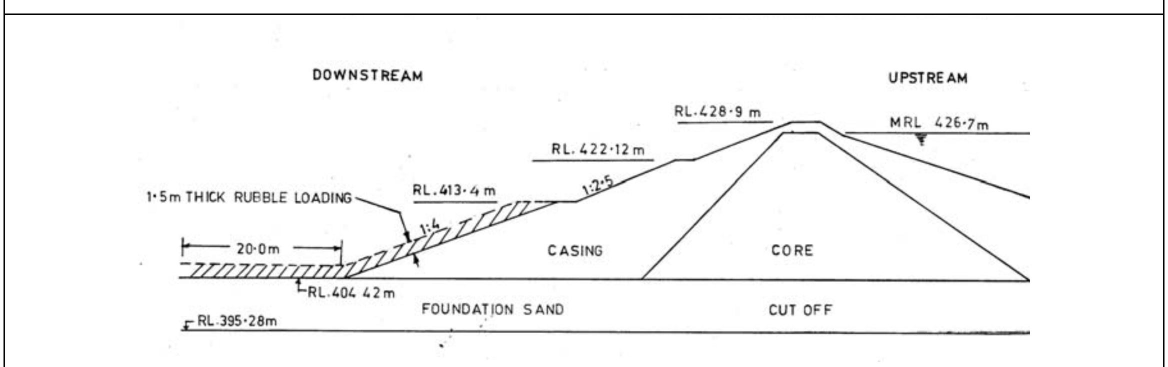

Fig. 16: Remedial Measures

8.2 Stability Analysis and Strengthening of Chang Earth Dam, Gujarat

The Chang dam site was constructed across river Chang in 1963, near village Kakarva, 24 km away from Bhachau, Gujarat. The dam was designed in the 50's, without accounting for the seismic stability and foundation liquefaction failure. It is a zoned earthen dam having an uncoursed rubble masonry core wall in lime mortar. The upstream and downstream shells are semi pervious soil, such as silty sand, sandy silts, and sandy clays. The maximum height of the dam is 15.54 m with overall length of 1266 m. The dam was badly damaged during Bhuj earthquake, which required rehabilitation and strengthening.

During the earthquake, significant liquefaction occurred beneath the upstream shell zone, where the low dead water reservoir pool saturated the alluvium and the base of the shell. A large translation slide occurred on the upstream side towards the reservoir, with crest settlement up to 6.5 meters. Large cracks and fissures on the upstream face of the dam were produced. Silty sand boil ejections were observed at and near to the upstream toe. The major longitudinal upstream slippage of 85 meters along dam axis had occurred. The impervious core zone and the upstream slide movement massively disrupted masonry core wall.

The rehabilitation/reconstruction works of the dam included: i) Field Investigations ii) Design aspects iii) Construction. Detailed soil investigations were conducted and soil tests of existing dam section, borrow area soils were carried out. For the design section, the seismic coefficient of 0.16 g with importance factor 2.5 had been considered.

In the Gorge Portion, loose soil was removed up to 3.5 m depth and wooden piles of 15 cm diameter were hammer driven up to rock, to densify the foundation material. The number of piles driven were 4438. In other portion of the earth dam, on u/s side near the toe existing loose soil was removed in 12.5 m wide and 1.0 m depth and replaced by pervious material. Above this compacted layer, loading berm of 11.0 m and on u/s side rock toe was provided as a measure against liquefaction.

The fine-grained soils of Clay core mostly comprise of clay silts in varying percentages. The clay content mostly varied from 18-25% qualifying as CL / CI material. A loose layer was compacted to achieve dry density of 95% of MDD. The Casing material contained 5-10% clay qualifying as SC / SM material.

The hand placed riprap was provided on the u/s and d/s slopes of the dam embankment. Graded filter has been placed underneath the riprap. The riprap material consisted of the most durable rock fragment of approved quality. Riprap had stones weighing more than 25 kg at more than 80% of the area. Headers in M10 concrete of size 150x150x600 mm have been provided normal to slope protruding at least 15 cm above general surface of the riprap to dissipate wave energy of reservoir water. Post-rehabilitation work the dam has been effectively utilized for the purposes for which it was constructed.

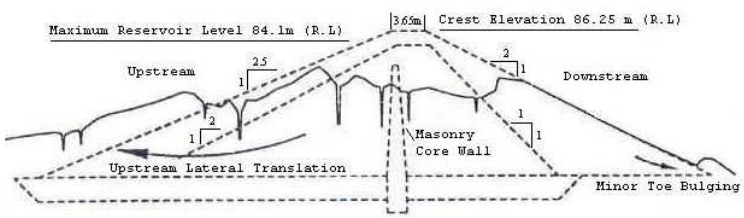

Fig. 17 : Failed Section of Dam , Gujarat

Fig. 18 : Transverse Cracks in Dam

9.0 CONCLUDING REMARKS

The dam construction engineering technology is a well advanced field in view of better understanding of geology, soil mechanics, geophysics, Hydrogeology etc. A safe and stable earth / rock fill dam is constructed by following the guidelines in the Standard codes in respect of selection of soils. The soil properties are determined from field tests as well as laboratory tests. The design section is arrived from stability analysis. Earthquake and liquefaction resistant design from dynamic analysis. Probable uncertainties are a priori visualized and implemented to arrive at an optimum and safe section. In addition, instrumentation with regular monitoring and analyzing the data, will improve the safety of the dams.

However, dams constructed in the past, with knowledge of yester years, needs to be assessed for their stability and safety. The existing dams need to be checked for stability by using the available numerical techniques for different loading conditions. The present properties of the soil can be determined from detailed soil investigation of dam and

Chapter 9

STRENGTHENING AND REPAIRS OF GRAVITY DAMS

1.0 INTRODUCTION

After independence, India has made commendable progress in construction of dams. India is one of the world's most prolific dam-builders. Some of the important dams in India are, (i) Tehri dam, the eighth highest dam in the world (ii) Hirakud dam, the longest dams in the world about 26 km in length (iii) Nagarjunasagar dam, the world's largest masonry dam with a height of 124 meters and is the pride of India being largest man-made lake in the world, (iv) The Grand Anicut located on Cauvery River in Tamil Nadu, is the oldest dam in the world ,(v) Indira Sagar dam is the largest reservoir in India and (vi) 84 m high and 415 long, Ghatghar lower roller Compacted Concrete dam is first of its kind in India.

Dams are good as a long as they are under control, but are a threat to public safety in case of their failure, causing considerable loss of life and property. Nature often interferes with any change caused in the environmental factor in the form of unprecedented floods or unpredictable earthquakes damaging dams. The three basic types of dams are Concrete dams, Masonry dams and Earthen-Rock fill dams. Causes of deterioration in the dams are Design deficiencies, Foundation deficiency, Moisture absorption, Temperature effects, Leaching, Excessive uplift pressures, Constructional defects, Defective construction joints, Floods etc. These damages lead to seepage causing loss of water and also affects structural integrity.

Damages in concrete dams are comparatively less than those in masonry and earthen dams. A massive failure of concrete dam has rarely been occurred. In concrete dams there is only possibility of cracking due to excessive loading, thermal effects or other effects like Alkali Aggregate reactions. These cracks can be repaired by grouting using suitable grouts.

2.1 STRENGHENING OF DAMS

Strengthening of the dams is generally accomplished by following methods.

2.2 Pre-stressing

Pre-stressing is generally considered to be an emergent measure of strengthening of gravity dams. There could be a loss of pre-stress over time. The treatment is therefore to be followed by some alternative permanent measures. The other disadvantages of this method are, pre-stressing cables have to be spaced closely in order to counteract the tension developed due to earthquake loading and pre-stressing towards heel of the dam sometimes induces tension at the downstream toe at low water levels.

The method of cable anchoring is as follows.

a. Drilling of Cable Holes
b. Water Proofing of dill holes by grouting
c. Re-drilling of holes
d. Water Testing of holes
e. Preparation of Cables to be anchored
    i. High Tensile Steel Wire for Cables
    ii. Testing of Wires
    iii. Preparation of Cables
f. Homing of Cables
g. Fixing of Bearings and Stressing Plates
    i. Bearing Plates
    ii. Stressing Plates
h. Tensioning of Cables
i. Final Grouting of Anchor Cables

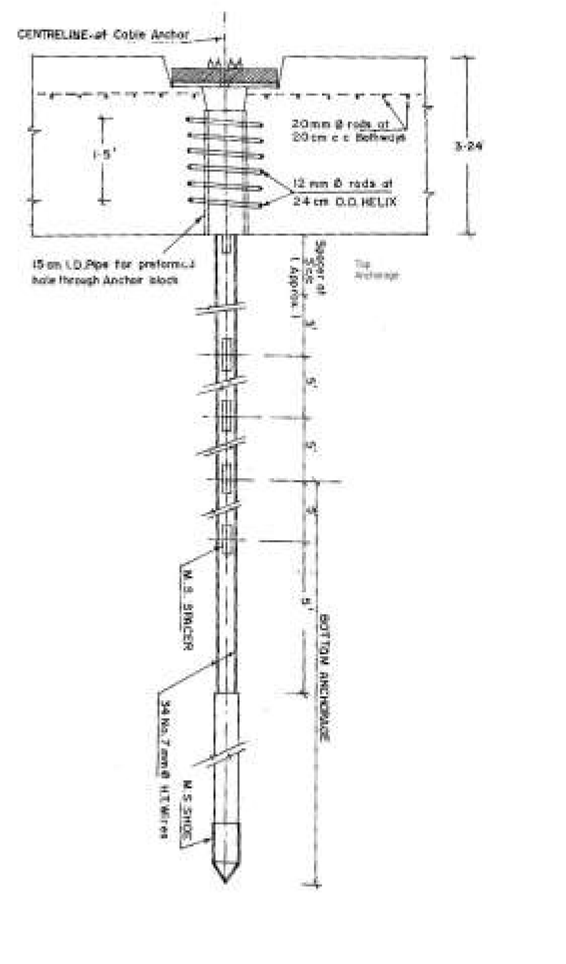

Figure 1 : Details of cable anchoring

The detail sketch of cable anchoring is shown in Figure 1.

2.3 Earth Backing

This is a simple way to strengthen the dam by providing earth backing on downstream face of dam. Possibility of separation between earth and masonry particularly near the top can not be ignored in such type of backing. In case of low water level, earth backing induces tension at the toe. A typical earth backing is shown in Figure 2.

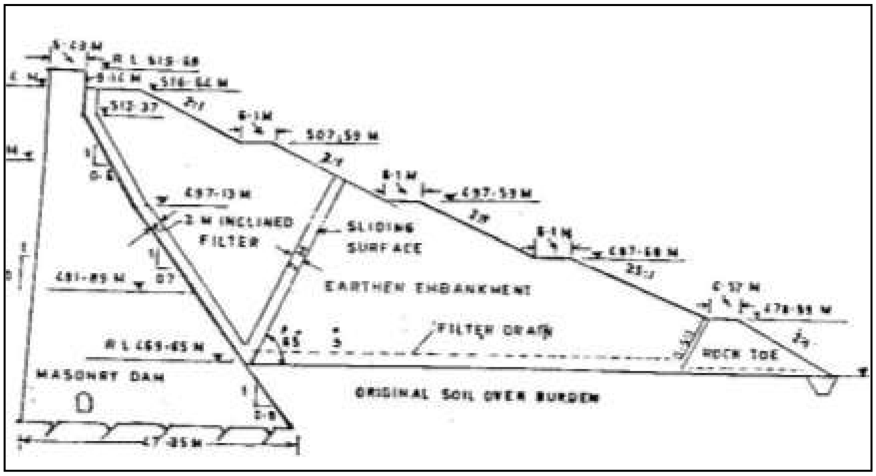

Figure 2 : Typical earth backing of a masonry dam

2.4 Concrete / Masonry Backing

A viable alternative of permanent strengthening measure is by providing buttresses or full masonry or concrete backing. The performances of dams backed using buttresses or full concrete backing has been satisfactory.

2.4.1 Design Aspect

The reservoir water level at which the backing material and the old structure are jointed has an crucial bearing on sharing of the load between the new backing structure and the old structure. These loads change due to variation in water level. A high level of bonding i.e at higher water level permits good flexibility in construction program, but is uneconomical. Lower bonding levels i.e at low water level put constraints on the availability of time for construction but is good for sharing of load by backing section requiring reduced sections and hence is economical. If the reservoir level is high during bonding, flatter will be the batter with large base width. Hence the reservoir level shall be as low as possible during bonding. Full backing is uneconomical as compared to partial or the buttress backing. Usually the total length of buttresses is not less than the half the length of dam for which strengthening is required. That is, buttresses supports a length of dam twice its width. The width of buttress at every section is generally less than the width of dam at that section.

2.4.2 Method of Construction

The buttress masonry is to be raised in specified manner in three zones (Figure 3). Initially the masonry in zone I for the full length of buttress is taken up after preparation of foundation irrespective of the lake level. Masonry in zone II along with bonding concrete is constructed only when the reservoir level is at bonding level or below. After completion of masonry in Zone II up to the level of masonry in Zone I if sufficient period is available till lake level rises above bond level, the masonry in Zone III for full section is raised to a suitable height.

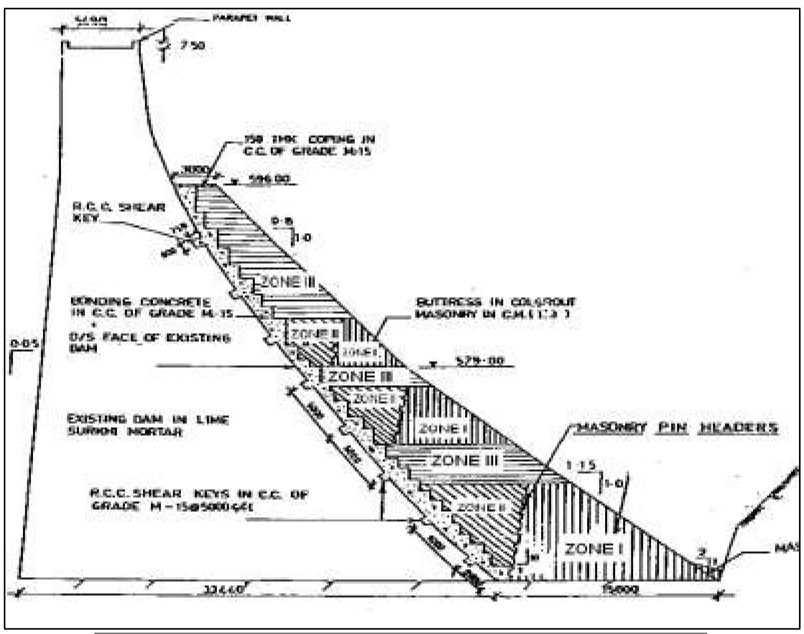

Figure 3: Raising of buttresses

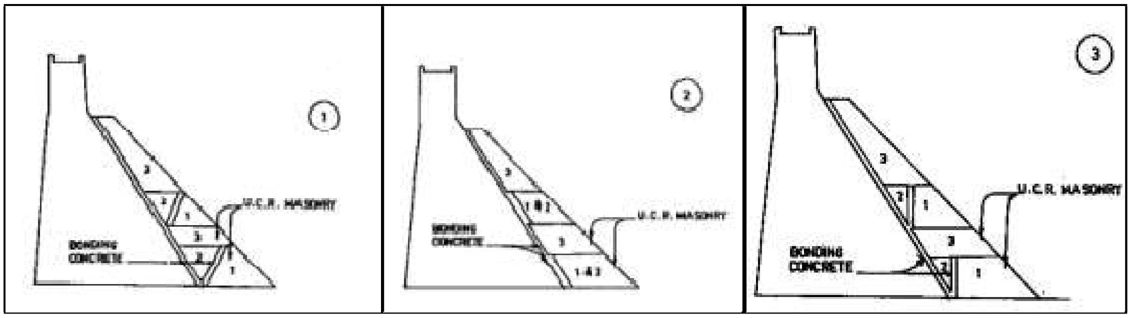

Figure 4 : Altenative methods of construction of buttreses

Alternative methods of construction of buttresses masonry are shown in figure 4. From the construction point view, construction by the alternative 2 is difficult as it involved a reverse slope for masonry during construction. Between the alternatives 3 and 1, the later is found suitable from the point of view of construction and it also reduces the quantity of masonry work. Before construction of buttresses, the face of the existing dam has to be cleaned thoroughly. Any damages or patches on surface to be removed till it give dense appearances. Pockets if any, on surface shall be filled by dense concrete.

Photo 1 – Operation gallery

2.5 CWPRS EXPERIENCES

2.5.1 Konar Dam – Grouting of concrete dams

The Konar concrete dam is located on river Konar about 31 km from its confluence with river Damodar in Hazaribagh district of Bihar. The construction of the dam was started in 1950 and was completed in 1954-55. The dam is provided with 3 galleries, inspection gallery of size 2.13 m x 1.22 m running almost the entire length, operation galley (Photo 1) of size 2.74 m x 1.52 m and drainage galley of size 2.44 m x 2.13 m.

In the year 1968 cracks were observed in all the three galleries. The cracks in the inspection gallery

Photo 2 – Cracks in gallery

were extensive as compared to other two galleries (Photo 2). The depth of the crack was varying between 1.0 m to 3.8 m. Studies indicated that the cracks are active due to a number of reasons such as temperature effects, structural loading and also due to variation in reservoir level. The dial gauges installed across the major cracks indicated relative movements of the two portions upper and lower of the crack line both horizontally and vertically.

Moreover, temperature variation due to heat of hydration and sun facing the downstream face of the dam for most of the day time and dynamic forces such as the seasonal fluctuation in the reservoir level, seasonal variation in temperature gradient and earthquake appeared to be the major causes of cracking causing distresses in dam.

On the basis of laboratory studies, CWPRS recommended the use of epoxy system having a mix viscosity of 360 cps, pot life of 3 hours, bond strength 40 kg/cm^2 in dry condition and 27 kg/cm^2 in wet condition for grouting the cracks. In the year 1970-71, epoxy grouting was carried out through surface nipples as well as through inclined drill holes at different levels intercepting the crack at different depths.

Photo 3 – Grouting of cracks

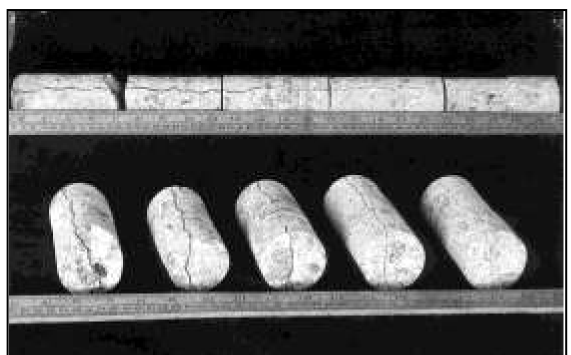

Photo 4 – Cores after grouting

Grouting (Photo 3) was started from middle of the block, proceeding towards the sides. The fine cracks less than 100 microns were grouted using low viscous polyester resin. The consumption of epoxy and polyester were of the order of 3000 kg and 200 kg respectively. Cores taken along the crack (Photo 4) after epoxy grouting, confirmed the efficacy of grouting for perfectly sealing of the cracks as well as restoration of strength.

The cracks reappeared in the year 1976 i.e. after a period of 5 years of treatment on upstream as well as downstream faces of the gallery. However, no seepage was observed through the cracks on upstream face. Thus the treatment though worked satisfactorily, failed to show long term performance. The probable reason for this may be, due to undertaking the repair work when cracks were active. This indicates that the treatment by grouting of the cracks should be necessarily carried out after they become stable/ dormant.

2.5.2 Koyna dam, Maharashtra –Strengthening by backing

Koyna dam comprising block 1 to 17 and 25 to 43 are constructed with rubble concrete / stone masonry (Figure 5). The overflow sections / central spillway comprise of blocks 18 to 24. A strong earthquake with its epicenter close to the dam rocked most of the western peninsular India occurred on 11th December 1967. It was one of the most severe earthquakes in the region. The dam as such was relatively unharmed though shaken severely. The damages caused are summarized as follows:

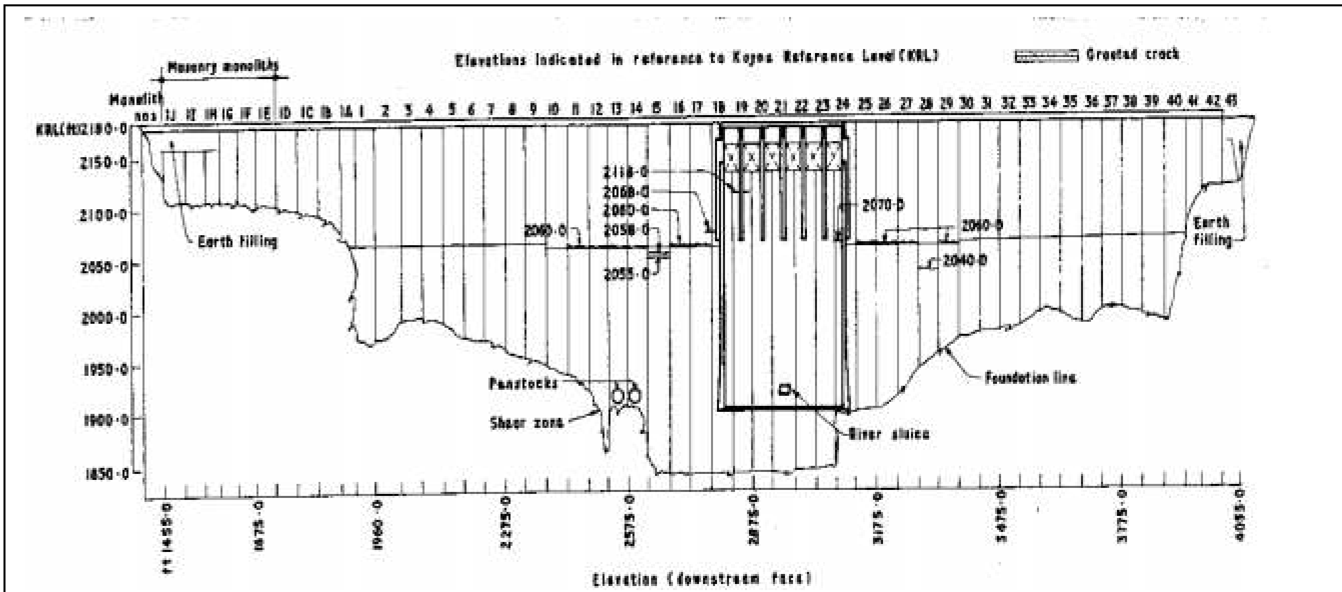

Figure 5: Section of Koyna dam

- Monoliths 1J to 9 and 34 to 43 did not apparently suffer any significant damage.
- Cracks were observed in minoliths10 to 18 and 24 to 33 on the upstream and the downstream sides in the region of EL 622 to 631

- Some horizontal cracks were noticed in the upstream face of the operating gallery in monoliths 19-21 at EL 602
- Cracks were also observed in the deck slab of the spillway bridge.

The remedial measures taken up immediately were, grouting of cracks by epoxy resins, gunitting the area adjoining the cracks in upstream face, drilling additional internal drainage holes from the top of the dam and strengthening of monoliths by pre-stressing. Subsequent to instant remedial measures, it was decided to make an allowance as a measure of safety for the forces that would develop due to earthquake and hence to modify the existing section of the dam suitably. Various alternatives like rock-filling, earth backing, solid concrete backing, and partially buttressed and partially solid backing were studied with respect their merits and demerits. It was then decided to consider the last option of these methods. This method was found suitable because, it has the advantage of not developing high inertia stresses which would have resulted in case of full backing due to large mass at higher elevations. Also in view of the thickness of the buttress section at the higher elevation and the greater exposed area, it would not be necessary to take cooling measures for concrete. The method is also economical.

It was therefore decided to provide full backing up to about EL 600.3 that is about 21.3 m above the river bed as well as buttresses of 7.62 m wide above this level up to EL 653.7 in each of the monoliths from 1A to 17 and 24 to 40 (Figure 6). The design however varied for set of monoliths according to the foundation levels. The backing was so laid that, there was gap of 1.22 m between the old and the new concrete. This was to allow the newly placed concrete to cool and shrink. This also facilitated the advantage of providing adequate working space between the two masses for laying of concrete of the same characteristics against old mass and avoiding high temperature stresses. The closure concrete was placed after a certain period after which the backing concrete had relieved temperature and shrinkage stresses. Shear keys and dowel bars were provided to take up the stresses at bonding interface (Figure 7).

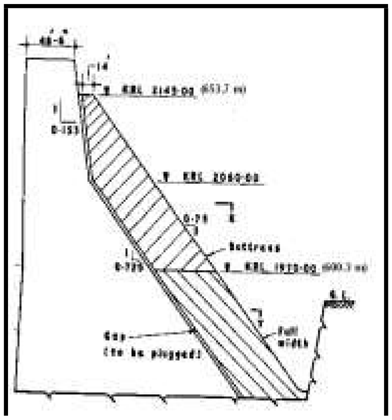

Figure 6 :Section showing backing

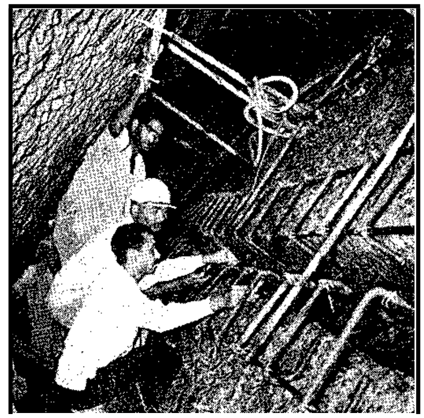

Figure 7 : Fixing of shear keys

The original dam used rubble concrete containing maximum size of aggregates 150 mm. Particularly in zones of full backing, the gap between old dam and the new concrete was maintained which was filled when the strengthening concrete had cooled down suitably relieving shrinkage stresses. Pre-cooling of concrete was also resorted to restrict rise in temperature in concrete. The concrete mix design adopted was 1:3.26:10.33 with a cement content of 172 kg/m^3. The buttresses were constructed in lifts of 0.75 m to 1.5 m. The concrete used for the buttresses was of the same grade as used for strengthening concreting except that the maximum size of aggregates was restricted to 80 mm.

After the Killari earthquake in 1993, it was planned to strengthening of the overflow section of the Koyna dam by full concrete backing as a preventive measure against earthquake. This also would facilitate to build up additional storage of 182.66 mm^3 in the dam using the provision of available flaps to the radial gates.

The scheme of strengthening was to provide full concrete backing of rubble concrete from the level of original dam foundation KRL 561.44 m to the upper tangent point of ogee KRL 645.0 m (figure 8). To overcome the interface problems between the old and the new concrete, it was decided to first cast the strengthening concrete with a gap of 1.2m, allow it to cool and shrink and then join it to the dam body by closure concrete. This was to be done when the reservoir level was at the predetermined level known as Bond RL to account for the locked up stresses in the body of the dam, since the dam could not be emptied during the strengthening work considering the strategic importance of this project in the hydro power requirement scenario of the state.

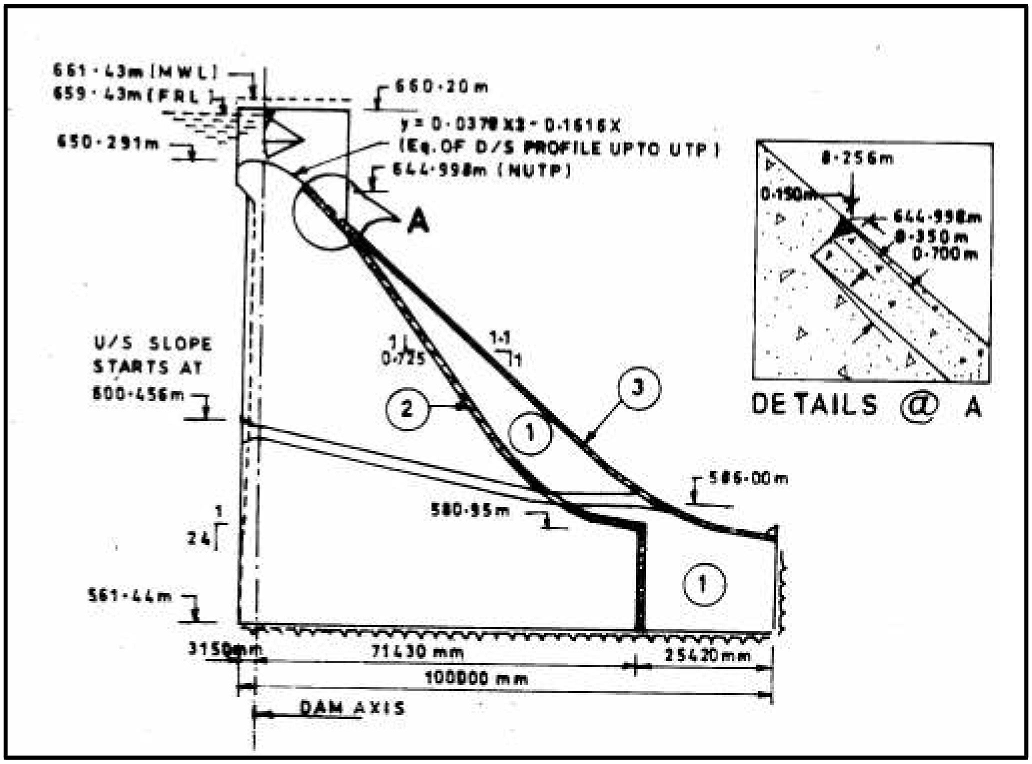

Figure 8 : Scheme of strengthening

1- Strengthening concrete 2- Closure concrete 3- Glacis concrete A- Junction point

Strengthening was done within two years of working season i.e. 2004-05 and 2005-06. The construction program was planned considering constraint of release of water through Power House, floods during the monsoon, constraint of completion of the work in two working seasons, restriction of availability of seven months working period in each season etc. This work involved in all 70,000 m^3 of excavation & 1,60,000 m^3 of concrete. Of the two years program, first year program was planned from foundation (KRL 561.44m) to river bed (KRL 580.90 m) and the second year program was planned strengthening work from river bed (KRL 580.90m) to top (KRL 645 m) and other ancillary works.

The concrete mix proportions were 1:1.66:6.37 (with cement content 177 kg/m^3 and MSA 150 mm) and 1:5.11:7.06 (with cement content 200 kg/m^3 and MSA 80 mm) for the strengthening and the closure concretes respectively. The strength, elastic properties, the ultimate strain capacity and the thermal properties of the concrete mixes were determined in the CWPRS laboratory and are given in Table 1.

Table 1

Sl No.	Properties		Strengthening concrete	Closure concrete
1	Compressive strength (kg/cm^2)	7 days	205	222
		28 days	330	363.7
		90 days	458	495.5
2	Modulus of elasticity (kg/cm^2)	28 days	3.2×10^5	3.6×10^5
		90 days	3.3×10^5	3.7×10^5
3	Adiabatic temperature rise ^0C)		27	30.
4	Diffusivity (m^2/hour)		0.0027	0.0027
5	Coefficient of thermal expansion (mm/mm/^0C)		7.59×10^{-6}	7.61×10^{-6}
6	Ultimate strain capacity		226×10^{-6}	250×10^{-6}

The laboratory studies and computations for estimation of suitable placement temperature and methods to achieve these temperatures were done at CWPRS. The estimated placement temperatures for the strengthening and the closure concrete were in the range of 18←C to 20←C and 16←C to 18←C respectively. The required placement temperatures in concrete were achieved by pre-cooling the concrete ingredients and adding ice to mixing water as a part replacement of water.

Strengthening of old dams by addition of fresh concrete to increases the effective section poses major technical problem at the interface of old structure which is cold, hardened and stressed in comparison to fresh concrete which is warm and unstressed equally. The design required detailed study of the loading condition of the old concrete at the time of joining. The newly placed concrete has to be cooled down to a certain level so as to relieve the shrinkage stresses before it is bonded with old mass. Pre-cooling cools the concrete to avoid excessive tensile stresses inside core mass as a safety measure against thermal cracking. Post cooling of concrete by circulating cold water inside the cast lifts of concrete is done to cool the hardened mass. The rate of post cooling is a important factor, because if the rate is high, tensile stresses develop in the concrete and if it is low the cooling period lengthens, affecting the construction schedule. Cooling parameters have to be decided for a given construction schedule to compromise between the cooling rate and the cooling water temperature so that the construction schedule remain unaffected.

The construction program for strengthening was framed to match the bonding levels and also to complete the work as per schedule. In order to cool the concrete up to predetermined temperature, cooling pipes made of galvanized iron each of 25 mm in diameter were placed in between two lifts, spaced 1m horizontally facing downstream side of the dam. Various stages of strengthening are shown in photos 5 to 8.

Photo 5 : Foundation Excavation

Photo 6 : Placing of cooling pipes

Photo 7 : Backing of oogee

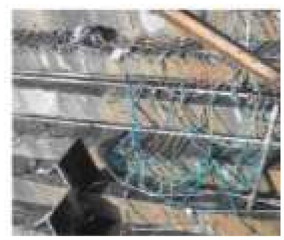

Photo 8: Circulation of water

From the computations the cooling water temperature was estimated to be $20^{\leftarrow}$C to $22^{\leftarrow}$C and the cooling period of about 35 to 40 days These parameters could achieve required rate of fall of temperature of less than $0.55^{\leftarrow}$c per day . During this cooling period, the strengthening concrete was expected to stabilize to $30^{\leftarrow}$C. The closure concrete of 1.22 m width was then placed after a period of 35 to 40 days for bonding of strengthening concrete with old dam. Temperature measured in the strengthening concrete fairly matched with the predicted temperatures.

2.4.3 Hirakud Dam, Orissa

Photo 9 – Hirakud Dam

59 m high Hirakud dam (Photo 9) built in the year 1957-58 across river Mahanadi in Orissa is a composite structure of earth, concrete and masonry. The total length of dam is 4.8 km flanked by earthen dykes of 21 km on left and right sides. The project has installed hydropower generating capacity of 307.5 MW.

The main dam has two concrete spillways having 32 blocks. Left spillway of 402 m length consists of 20 blocks and right spillway of 256 m consists of 12 blocks. They also comprise of 64 low level sluices and 34 radial crest gates. During the course of time, horizontal cracks of various sizes on the upstream face, sluice barrels, operation gallery and around gate shafts and vertical cracks near the right end blocks of spillways were observed. The cumulative length of these cracks was more than 22 km (Fig. 9). The cracks were classified into 2 groups – fines which are less than 5 mm width and wide having size between 5 to 12 mm. About 95% cracks were fine cracks.

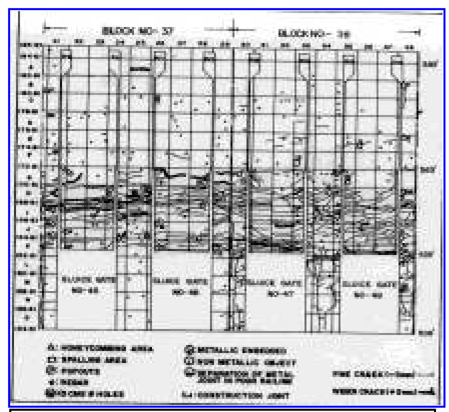

Fig. 9 - Cracking Pattern

It was never thought feasible and economical to deplete the water level and to create dry condition. As reservoir level is operated between RL 630 ft to RL 595 ft. underwater repairs were resorted to for cracks between RL 595 ft. to RL 506 ft. The cracks between RL 595 ft to RL 610 ft were repaired under dry condition.

1. Thermal cracks were developed because of rapid concrete placement
2. Seepage through these cracks causing sufficient ingress of moisture for alkali silica reaction

It was proposed to grout the cracks with epoxy system suitable for underwater treatment. For this purpose, number of epoxy grout system and sealing system were obtained from outside countries and also from Indian market. Necessary tests were conducted for grout and sealing systems by casting the specimen underwater. The injection system having low viscosity of the order of 150 cps and which indicated bond strength of more than 24 kg/cm^2 and 16 kg/cm^2 in direct tension and shear mode respectively was adopted for use. The sealing system selected also indicated pressure bearing capacity of more than 4 kg/cm^2.

Photo 10- Core after grouting

After grouting of the cracks, cores of 65/40 mm dia. and length 5 times dia. were taken from treated crack surface for visual inspection (Photo 10), observation of penetration of epoxy and to check the efficacy of grouting. The cores so obtained were tested under compression indicated failure in concrete away from treated crack plane. The dam was visited recently in Oct 2016 and the views of drainage gallery and spillway portion are as shown in photo Nos. 11 & 12 which proved the efficacy of grouting even after two decades.

Photos 11 & 12 – View of spillway and drainage gallery.

2.4.4 Varasgaon dam, Maharashtra

Photo 13- Varasgaon Dam

Varasgaon dam (Photo 13) is a 63 m high masonry gravity dam built across Mose River during the period 1976 to 1992 in Pune district of Maharashtra. It is one of the four major dams (Khadakwasla, Panshet, Temghar and Varasgaon) which provide water for drinking and irrigation to Pune city and nearby region.

During construction, extensive cement grouting has been done in the dam body to arrest seepage through dam body. Heavy seepage was observed just after first filling, to control seepage through dam body, 50 mm thick cement mortar gunitting has been provided on upstream face during 1986-87. After gunitting seepage has reduced considerably. With the passage of time, seepage has increased through the body of the dam at FRL. For controlling the seepage, project authority has decided to grout dam body using cement fly ash mix grout by adding suitable admixtures.

Accordingly, studies have been taken up at CWPRS. Laboratory studies have been conducted by preparing grout mix using different cement fly ash ratios and water cement ratio. For assessing the suitability of grout various laboratory tests such as Flowability test-Marsh Cone time of afflux, pH value, bleeding potential, jellification time etc. have been carried out.

Also, for assessing suitability of grout in masonry dams, grouting trails have taken up in distressed masonry blocks of size 1m x 1m x 1m by grouting with different percentages of fly ash and water content. To assess the effect of grouting, non destructive test and permeability/water loss tests have also been carried out on distressed blocks at pre and post-grouting stage (Photo Nos. 14 & 15).

Photos 14 & 15 – Grouting in trial blocks and extracted core after grouting

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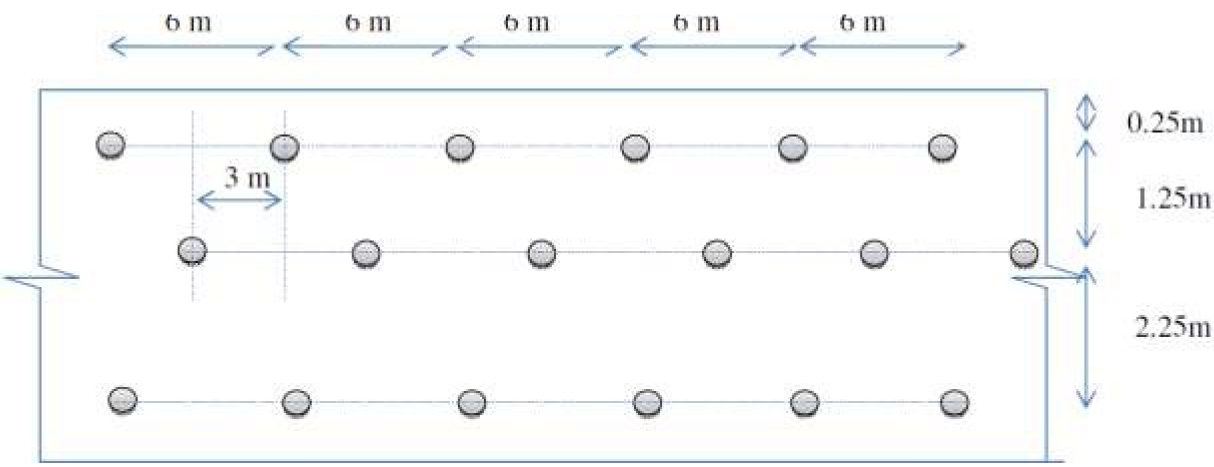


Figure 10. Recommended layout of grout holes in plan

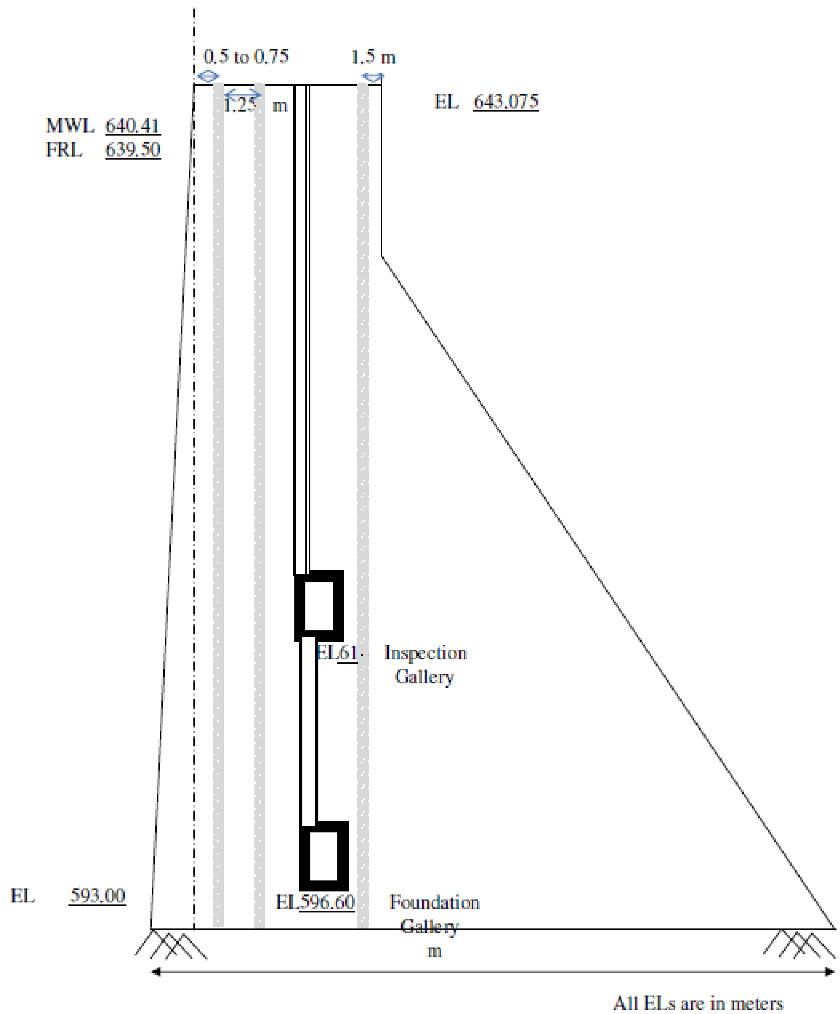


Figure 11. Proposed layout of grout holes in sectional elevation

For Varasgaon dam, stage grouting is recommended and, in descending stages, is suggested. Stage grouting is conducted to permit treatment of various zones individually, by grouting successively increasing depths. As the purpose is to make impermeability of upstream face, the grout holes are arranged in a series of lines to form a curtain approximately perpendicular to the direction of seepage.

3.1 SPILLWAY

3.2 Types

Spillway is a structure used to provide the controlled release of flows from a dam or levee into a downstream area (Photo 16 to 18). Spillways release floods so that the water does not overtop and damage or even destroy the dam. Floodgates and fuse plugs may be designed into spillways to regulate water flow and dam height. There are two main types of spillways, the controlled and the uncontrolled spillway. A controlled spillway has mechanical structures or gates to regulate the rate of flow. An uncontrolled spillway, in contrast, does not have gates; when the water rises above the lip or crest of the spillway it begins to flow from the reservoir. Chute spillways shown below are obvious choice wherever foundation strata poses difficult problems

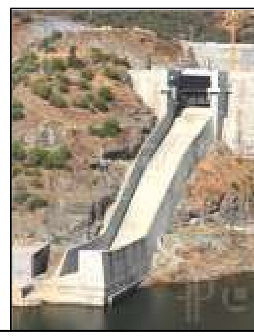


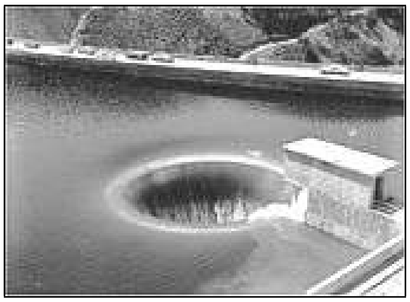
Photo 16	Photo 17	Photo 18

Types of Chute Spillways

Adequate flood release facility to pass the flood flows and the associated safe dissipation of the energy of the flow may also be achieved by the design of a staircase spillway, called stepped spillway. A stepped spillway design increases considerably the rate of kinetic energy dissipation taking place down the spillway channel. In turn, the design eliminates or reduces greatly the needs for a sizable energy dissipater at the toe of the chute in the form of a hydraulic jump stilling basin or flip bucket and plunge pool (Photo 19 to 24).

Photo 19 **Photo 20** **Photo 21**

Stepped spillways



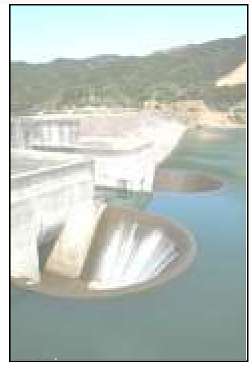


Photo 22 **Photo 23** **Photo 24**

Bell mouth spillways

Bell -Mouth spillways (Photo 28 to 30) are designed like an inverted bell so that water can enter all around the perimeter. These uncontrolled spillway devices are also called morning glory, plughole, glory hole or bell-mouth spillways. In some cases, bell-mouth spillways are gate controlled.

Siphon spillway is enclosed spillway passing over the crest of a dam in which flow is maintained by atmospheric pressure. Siphon Spillway is a closed duct. The design of this type of spillway is done on following assumptions. (1) Hood level is higher than reservoir level. Hence, when flowing full the water level in pipe is higher than the reservoir level and (2) Siphon must be self priming. Typical views of siphon is shown in photos 25, 26 and 27.

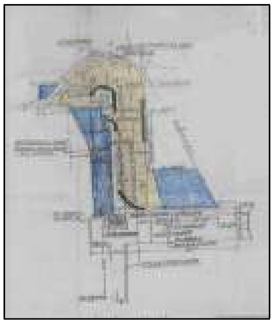


Photo 25 **Photo 26** **Photo 27**

Sketch and views of siphon spillway

Other spillway types include an ogee crest which over-tops a dam, a side channel that wraps around the topography of a dam and a labyrinth which uses a 'zig-zag' design to increase the sill length for a thinner design and increased discharge. There is also a drop inlet which resembles an intake for a hydroelectric power plant but transfers water from behind the dam directly through tunnels to the river downstream.

Energy dissipaters are located at the downstream end, after the fall is completely negotiated and in the vicinity of the natural stream. They may include chute blocks, baffle blocks, stilling basin, end sill and side (training) walls. It is preferable to keep them vertical on water side for the satisfactory formation of hydraulic jump. When the velocity at entry of stilling basin is high, chute and baffle blocks are omitted.

3.3 Damages in spillways

The damages to spillway occur mainly due to, unequal operation of spillway gates, cavitation effect ,abrasion erosion damage, uplift pressure etc. Concrete surface abraded by water borne debris are generally smooth and may contain localized depressions. Most of the debris remaining in the structure will be spherical and smooth. Mechanical abrasion is usually characterized by shallow grooves in the concrete surface and spalling along the monolith joints. One or the other cause above or a combination of all these, damage the spillway and the associated areas of dam. If the damages are not attended timely these damages aggravate making structure unsafe for operation.

Damages in some of the spillways are shown in photos 28 to 32.


Photo 28


Photo 29


Photo 30


Photo 31


Photo 32

3.4 Prevention of damages

The measures should be followed to prevent or minimize abrasion-erosion damage by proper design of spillway to suit flow condition, proper operation of gates for balanced flow into basins and use of suitable materials for construction. The material for repairing the damages in concrete spillway shall have following properties, high compressive strength and tensile strength, good resistance to abrasion, adequate bond with parent surface.

3.5 Case Studies – Repairs of spillway

3.4.1 Sardar Sarovar dam, Gujarat

Sardar Sarovar project comprises of a concrete gravity dam 1200 m long and 165 m high across river Narmada, a riverbed under ground powerhouse of 1200 MW with 6 units of 200 MW each and a canal head powerhouse with 5 units of 50 MW each. The dam has a service spillway of 23 spans and auxiliary spillway of 7 spans each of 18.3 m width separated by 4.7 m thick piers. The energy dissipation arrangement at downstream of each tunnel is provided by way of independent stilling basin each of 72.88 m length and 18.8 m width with an apron portion. Although the concrete mix used to construct the stilling basin was of rich grade of strength 350 kg/cm^2, the surface of the stilling basin got damaged to about a meter depth. Though the exact causes of damages could not be clearly identified, these were anticipated to be due to uplift pressure and micro turbulence phenomenon because of frequent floods passing over the area. Typical damages are shown in Photo 33 and 34. The volume of damaged concrete in five bays was estimated to be of the order of 31,914 cum.

Photo 33

Photo 34

Following methods were suggested for repairing these damages.

- Grouting of the cracks in the damaged surfaces by epoxy or cementitious material depending on the opening of crack.
- Repairs to cavities in stilling basin. .
- Resurfacing damaged portion using impact resistance repair materials
- Grouting of inaccessible deep cavities in concrete substrate.

The materials for repairs for different zones of spillway were identified by conducting laboratory tests at CWPRS to determine strength, elastic and bonding properties on a number of repair materials, such as cementitious mortar, micro-concrete ,epoxy concrete, epoxy mortar and joint sealants. Those found suitable were recommended for use.

3.4.2 Jawahar Sagar dam, Rajasthan

36 m high and 393 m long, Jawahar Sagar dam , is a concrete gravity dam across river Chambal located about 31 km from Kota city. The dam was constructed in the year 1971 and is the third structure in the series of cascade of dams, viz: Gandhisagar, Ranapratapsagar , Jawahar sagar and the Kota barrage. It is provided with ogee shaped gated spillway and a slotted roller bucket with teeth for energy dissipation. The river bed downstream is at a higher elevation than the bucket and is composed of quartzite and shale in horizontal layer.

Floods of the order of 15,500 cumecs passed over the roller bucket in the years 1973 and 1975. These floods caused damages to the teeth and the apron portion (Photo 35 and 36). Though the damages to teeth were not of serious nature, the damages in apron were alarming exposing reinforcement in the concrete at many places. These damages were repaired in the year 1982 bringing the teeth and the apron to desired profile. Epoxy was used as bonding material and concrete for filling the potholes.

Photo 35

Photo 36

After the repairs in 1982, the spillway experienced large floods again during the years 1986 and 1991 of the order of 17,900 cumecs. Due to these floods, damages again occurred destructing the teeth and the apron surface. The damages to the teeth were surfacial with corners and edges broken in few teeth, where as damages to the apron surface and the lip portion of bucket were noticeable. The damaged area including major and minor potholes was estimated to be of the order of 1155 sq.m. These damages in the teeth, apron surface and the lip portion of apron were repaired in early 1994 under the guidance of CWPRS. The methodology adapted was as follows.

1. Deep cavities were filled with epoxy concrete.
2. Shallow cavities were filled with epoxy mortar.
3. The repaired surfaces were coated epoxy mortar to match with adjacent original level.

The epoxy compound used for bonding the surface and for preparing epoxy concrete and epoxy mortar was identified after conducting tests on a number of compounds in CWPRS laboratory. The proportions of epoxy concrete and the epoxy mortar were determined by studying their workability, strength and elastic properties. An area of 1227 sq m was repaired using epoxy concrete and epoxy mortar. The quantities of repair material consumed were, Epoxy mortar: 30.28 cum, Epoxy concrete: 41.12 cum. Epoxy consumed: 22 tonnes, Quartz sand: 67 tonnes and aggregates: 54.5 tonnes

The roller bucket was dewatered for routine inspection in April 2011.It was observed that a major part of the area repaired in the year 1994 was found intact (Photo 37 to 39).

Photo 37

Photo 38

Photo 39

4.1 STUDIES ON STEEL FIBRE REINFORCED CONCRETE (SFRC) AS REPAIR MATERIAL

A broad study of literature proposes the use of steel fiber reinforced concrete (SFRC) to repair stilling basin. An experimental work done by A. N. Nataraja shows an increase of about 32 percent in splitting tensile strength was observed for a reinforcing index of 2.67. An experimental work done by Nguyen Van CHANH shows fibres aligned in the direction of the tensile stress may bring about very large increases in split tensile strength, as high as 133% when only 5% smooth and straight steel fibres are used in conventional concrete.

4.2 Properties of SFRC

Mechanical properties of steel fiber reinforced concrete are influenced by type and percentage of fiber addition, aspect ratio of the fiber, strength of the matrix, and size of the aggregate. Bond between matrix and fiber increases as the length of the fiber increases.

Addition of 1.5% fibers by volume increases direct tensile strength of mortar by about 40%. The increase in splitting tensile strength is somewhat higher, with reported increases of as much as 100%. Influence of steel fibers on flexural strength is much greater than that on tensile strength. Two strength values commonly reported are first-crack strength, where the load-deformation curve departs from linearity, and ultimate flexural strength. Generally, increase up to 150% in first-crack flexural strength for conventional SFRC are reported. Compressive strengths for conventional SFRC vary, sometimes from a loss of strength to a gain as much as a 40% increase depending on type and quantity of fibres used. SFRC can provide considerable improvement in abrasion resistance. The addition of fibers to a conventionally reinforced concrete structure will increase the fatigue life and decrease crack widths. One of the greatest advantages of SFRC is its increased toughness. Toughness is defined as the total energy absorbed in breaking a specimen. It can be quantified by measuring the area under the load-deflection curve obtained from a flexural test .Impact resistance is related to toughness. The addition of steel fibers to concrete can dramatically increase the impact resistance of the concrete. Added fibers will not only increase impact strength but they also help prevent shattering of the composite into fragments after impact.

4.3 Case Study of repairs to stilling basin of Koyna dam using SFRC

In view of above properties of SFRC, studies on steel fibre reinforced concrete were conducted for its use to repair the damages in stilling basin of Koyna dam (Photo 40 to 42) . The properties of SFRC and the discussions are given in following paragraphs.

Photo 40

Photo 41

Photo 42

The quantities in Kg/m^3 of the SFRC mix proposed to be used for repairs are given in Table 2. The steel fibers used were Dramix steel fiber RC-80/60-BN (BEKAERT Make). The observed average density and slump was 2600 Kg/ m^3 and 35-40 mm respectively. The results of the tests conducted on concrete mix with fiber doses ranging from 0 to 40 kg/m^3 are given in Table 3

Table - 2

MIX PROPORTION BY WEIGHT (kg)

53 Grade OPC	**Sand**	**Dramix fiber**	**Coarse Aggregates**		**Water**
			20-10 mm	**10-05 mm**	
1	1.8	0.045	1.7	1.0	0.43
50	90	2.25	85	50	21.5

Table - 3

TEST RESULTS OF SFRC

	Fiber dosage (kg/m^3)				
Properties	**0**	**10**	**20**	**30**	**40**
Compressive Strength (kg/cm^2)	450 – 500	480 – 510	500 – 530	520 – 530	530 – 570

Flexural Strength(kg/cm^2)		38 – 40	41 – 43	43 – 46	45 – 47	49 – 57
Toughness Indices as per ASTM C-1018 – 97	**I_5**	--	2.3-3.9	4.8 – 5.9	5.2 – 7.9	6.1 -8.4
	I_{10}	--	6.4	9.3 – 12.3	9.6 – 12.5	10.5 – 12.9
	I_{20}	--	12.7	22.7	23.7	27.4

5.1 CONCLUSIONS

1. Pre-stressing for dam is an immediate and effective remedial measure particularly when damages occur due to unprecedented floods or earthquakes. The measure however is to be followed by permanent repair method like masonry backing or concrete backing.
2. Temperature control measures for dams discussed in the note are not only necessary but mandatory to estimate suitable placement temperature, cooling water parameters for assessing the shrinkage stresses to decide the bonding period of concrete backing.
3. The conventional remedial measures like filling cavities, pointing and limited grouting in the dam body are economical and serve better to control seepage in masonry dams. If these measures are attempted flawlessly, they would certainly enhance the life of dam and restore functionality.
4. The materials for repairs to spillways and the repair methodology has to be suitably decided depending on causes of damages and the extent of damages
6. Epoxy based material – epoxy concrete / epoxy mortar are best suited for portion of spillway and structures remaining under water
7. High strength, abrasive resistance cementitious mortars having good bonding to concrete can be suitably used to resurface the damaged and pitted surfaces of spillway.

8. There is a considerable improvement in the properties of the concrete for a dosage of 20-25 kg/m^3 of steel fibres, particularly the flexural strength and the toughness index recommending use of SFRC for repairs to spillway structure.

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Chapter 10

IT Tools in Dam Safety Management – Development of Dam Health and Rehabilitation Monitoring Application

1.0 Introduction

Since independence India has been constructing a substantial number of dams to meet the needs of irrigation, drinking water, hydro-power and supplies to municipalities and industries. Based on size and importance, these dams have been categorized as large, medium or small dams. About 4050 large-dams[1] are already in existence, and another 475 are under construction; while full information on medium and small dams – numbering several thousands – is yet to be catalogued. In terms of large dams, India ranks third in the world after China & USA. Although almost every state of India has large dams, the major chunk is situated in Maharashtra (1453), Madhya Pradesh (793) and Gujarat (470).

'Dam Safety' is now considered an inherent feature of planning, design, construction, maintenance and operation of dams; and hence most of the known causes for possible failures are taken care of in case of recent dams. However, dam safety aspects have not been given adequate attention in the old dams, and this has been a cause of serious concern especially in view of the fact that a large number of these dams are aging[2], leading to gradual degeneration. Another aspect in respect of these dams is that they were constructed using the technology, standards and criteria prevalent at that time, which have changed vastly over the years due to experiences gained and the advancement in technology.

Recent time has seen increased awareness as well as positive actions in the direction of dam safety and dam rehabilitation issues. With the establishment of 'Dam Safety Organizations' at central level and in different states, substantial momentum has been gained in the direction of monitoring of the health of ageing dams; and an immense amount of data is being generated and passed on to various state and central bodies. This volume of data is expected to increase many folds with the taking-up of rehabilitation works of old dams. In view of the importance of dam safety issue, there is an urgent need to collect and present the Dam's Health and Rehabilitation related data in an organized manner, and with standard format and nomenclature. With this objective, Central Water Commission (CWC) has presently taken up the task of developing a 'Dam Health and Rehabilitation Monitoring Application', nicknamed 'DHARMA'.

1.1 Present Mechanism for Dam Safety

State governments have autonomy in their list of subjects which includes management of their own water resources within their state. Water being a state subject, states are the owners of the dams within their territories, and hence responsible for the dam safety and any remedial measures. As such, any initiatives by the Central Government would necessarily have to involve the state governments for its proper implementation. Keeping this in view, the matter was broached in the State Irrigation Ministers Conference held in 1975 and, as a follow-up of its recommendations a Dam Safety Organization (DSO) was created at the centre in Central Water Commission (CWC) in 1979.

The objective of this DSO was to perform a coordinative and advisory role for the State Governments and to lay down guidelines, compile technical literature, organize trainings, etc. and in general to take steps to create awareness in the states about dam safety, and thereafter assist in setting up infrastructure for the same. The DSO-CWC[3] took up the initiative by including 12 states (having significant number of large dams) and assisting them in setting up 'Dam Safety Organizations' in their respective States. The DSO of CWC prepared a number of guidelines on dam safety and also compiled the 'National Register of Large Dams'. Besides imparting dam safety related trainings to engineers at Central and state levels, it has also provided (in special cases[4] and on requests from concerned state governments) design related assistance in rehabilitation of distressed dams. With the sustained efforts of DSO-CWC, a total of eighteen[5] Dam Safety Organizations have been established in various states and some of the public sector organizations so far.

As State Governments are the owners of irrigation and multipurpose projects in India, the responsibility of dam safety largely rests with them. As per the mechanism in place for all large dams (irrespective of their age), the safety studies are required to be carried out by the "Dam Safety Organizations (DSO)" of respective states. And, in the consolidated annual report[6] to be brought out by the State DSOs, the detailed information on the health of inspected dams are required to be brought out according to the degree of seriousness of the observed deficiencies under three categories, namely Category-I (major deficiencies which may lead to failure), Category-II (rectifiable deficiencies needing immediate attention), and Category-III (minor or nil deficiencies).

The role of Central Government is confined to monitoring and demand-based advisory services with regard to safety and rehabilitation of such dams; and this task is mainly accomplished through the 'National Committee on Dam Safety (NCDS)'. The NCDS was constituted by Government of India in October 1987 by broad-basing the then existing Standing Committee on Dam Safety. Headed by Chairman of CWC, the NCDS includes representatives of all states having significant number of large dams and some of the agencies/ organizations owning sufficient number of large dams. The NCDS has been entrusted with the task of overseeing dam safety activities in various States/ Organizations, and suggest improvements in line with state-of-art technologies consistent with Indian conditions. In addition, it provides a forum for exchange of views on techniques adopted for remedial measures to relieve distress in old dams. It also monitors follow-up action on the recommendations of the "Report on Dam Safety Procedure[7]" published in July 1986.

1.2 Dam Health and Rehabilitation Monitoring Application

A precise definition of dam failure or a definite quantification of extent of failure may not be possible. Understandably, this is owing to the large variations in type, size and functions of dams, and due to a plethora of reasons that can be attributed for dam failures. Conveniently hence, a converse route is adopted by defining the safe dam; and a safe dam is defined as the one which performs its intended functions without imposing unacceptable risks to the public and society by its presence. Any deviation from this broadly defined position will necessitate an in-depth examination of dam for possible failure, impact assessment of failure, analysis of the causes of failure, and finally implementation of remedial measures for correcting such failure. Evidently, the stated situation calls for periodical inspection of all dams, and a deeper and more frequent inspection of such dams that are perceived as failing in performing their intended functions, or are seen as a risk to the life and property of the people.

Efforts are already being made for updating the database of ‘National Register of Large Dams’, and for creating a ‘Management Information System’ assimilating the historical (engineering) data of large dams. Although the data being sought so far is static in nature, yet its generation is a cumbersome task owing to such factors as: large numbers and variations in types of dams, varied ownerships, geographical spreads, and a vast distribution in their age. In the meanwhile, periodical process of recording the health status of various dams by the Dam Safety Organizations of concerned State/ Public Sector Undertaking (PSU) has also commenced; and an immense amount of data is being generated and passed on to CWC on annual basis. This voluminous and dynamic data needs to be retained as time series; and its quantum is expected to increase many folds with the taking-up of rehabilitation works[8] of old dams. For meaningful realization of all efforts that are being made, there is an urgent need to collect and present the Dam’s Health and Rehabilitation related data in an organized manner, with standard format and nomenclatures. And with this view, Central Water Commission has taken up the task of developing the ‘Dam Health and Rehabilitation Monitoring Application’ Software.

Besides facilitating generation and long-term storage of voluminous data by numerous State/ PSU DSOs, the proposed DHARMA software will enable a systematic presentation and interpretation of data for effective monitoring of the health of dams by the concerned central/ state/ PSU organizations. The software will also help the decision making processes for effecting rehabilitation measures as and when needed. The software development project will be executed by the ‘Software Management Directorate’ of CWC with the help of software professionals outsourced from the market. The project will be taken up in a phased manner, gradually assimilating the available knowledge and database being generated, and by evolving a standardized format for dam-health reporting. Eventually, the fully developed software is expected to allow the larger sharing of database and knowledge-pool, besides bringing the much needed transparency on dam safety and rehabilitation measures.

Some of the key features of the DHARMA development work in hand are as under:

- The State/PSU DSO concerned with a particular dam will be considered as the primary source of pertinent data in DHARMA. For this purpose, the software application - adequately configured for each DSO - will be supplied by CWC along with preliminary data already available in ‘National Register of Large Dams’. All other data will be generated by concerned State/ PSU DSO only, and no data will be keyed twice.
- All dams will be given a unique ‘Dam Identification Code’ (DIC). The ten digit identification alpha-numeric code (XX11XX1111) will carry tags of: state in which dam is situated (first set of two-digit alpha code[9]), concerned DSO (first set of two-digit numeric code), category (second set of two-digit alpha code for large [LD], medium [MD] and small [SD] dams), and the serial number (second set of four-digit numeric code).
- DSO-codes have been assigned for the 18 State/ PSU DSOs registered[10] with CWC, and further codes will be assigned as and when new DSOs will be registered with CWC. Unique codes for dams that are listed under ‘National

Register of Large Dams' have already been generated; and for all other dams, auto generation of DIC will be effected by the concerned Sate/ PSU DSO using their copy of DHARMA.

- The application in general will be divided into six modules. (1) The 'Basic Feature' module will contain such fundamental data as appearing in the 'National Register of Large Dams', besides providing tools for generating DIC in case of new dams. (2) The 'Engineering Feature' module will allow entering of such information as: salient features, design, hydrology, geology, construction history, operation plan, instrumentation, maintenance schedule, earlier studies, safety related events, and known deficiencies etc. (3) The 'Stakeholders' module will record pertinent information about organizations/ agencies responsible as/for owners, operations, beneficiaries, contractors, emergency action etc, besides enlisting the impact of possible dam failure, parties effected, and the emergency action plan. (4) The 'Dam Health' module will record the periodical observations of health inspection teams in a standard format. (5) The 'Dam Rehabilitation' module will catalogue such information on periodical repairs as: type of work, technique involved, cost of work, agencies involved, extent of mitigation etc. (6) The 'Analysis and Report' module will provide varied tools for capturing time-series data of pertinent parameters for the purpose of analysis and report preparation.
- The proposed software will allow multiple-level data processing (data entry, data editing, and data deletion) with adequate password protections. Most of the data will be captured through combo box provisions, thereby enabling standardization of data and limiting the scope of typographical work. The standardization will be made an evolving process with the help of 'Masters' which will be periodically updated by DSO-CWC; and the updated master-table will be passed on to all DSOs. For flexibility of use, copying/ saving of data in multiple files will be permitted. Appropriate tool will also be provided for assimilation of data files received from various sources for creation of a holistic national level database by CWC.
- To the extent digitized data is available in CWC, the basic and engineering data of all large dams will be captured before passing the customized applications to all DSOs. The missing basic/ engineering/ stakeholder data in case of large dams (and full historical data in case of other dams) will be captured by concerned State/ PSU DSO. The health and rehabilitation related data will be captured by State/ PSU DSOs on periodical basis, and such updated data files will be mailed to DSO-CWC on annual basis for further assimilation, analysis and enrichment of 'Masters' .

1.3 Implementation Strategy

Development of DHARMA software will be an evolutionary process and will be carried out with wider consultation with all DSOs. The module-wise development is proposed to be carried out in a phased manner; and even on completion of all modules, the software will be subjected to continuous process of up-gradation. Since capturing of vast historical data will be a fairly time-consuming exercise, it is proposed to be carried out *pari-passu* with the software development. Hence, release of early versions of software will be effected with the module-wise development and with appropriate precautions for data compatibility requirements.

For facilitating wider consultation and for assimilation of practical conditions pertinent to the development and implementation process, it is proposed to create a permanent group under NCDS for carrying out periodical field visits of old dams for meaningful interaction with field and DSO officials. Besides enabling qualitative enhancement of the application, the proposed interactive process will facilitate a wider use and acceptability of the application across all levels of DSOs. Once overall modules of application are developed, the proposed group will assume a role of 'Audit Team' for ensuring uniform and effective implementation of DHARMA across all DSOs.

The proposed group - named as 'Dharma Implementation Group' (DIG) - will have three officials on permanent basis and three officials on two-year's rotation. The DIG will be headed by Chief Engineer (DSO), CWC, with Director (DSM) as member and Director (SMD) as member-secretary, while three officials of the rank of Superintending Engineers will be nominated from State/ PSU DSOs on rotational basis. The DIG will meet at various project locations at least thrice in a year; and needful arrangements for such meetings will be facilitated by the concerned State/ PSU DSO. The periodical reports submitted by DIG will be discussed in the NCDS meetings for reviewing progress of development and implementation of DHARMA

1.4 Conclusion

At this crucial juncture when prevailing water crisis demands construction of many more dams, the existing dams are increasingly getting distressed due to old age and varied other issues. Moreover, with the increased public awareness about environment and safety concerns, the dam health issues are becoming central to almost every debate on water resource development, and in some case even demands for decommissioning of old dams are being raised. Evidently thus, there is an urgent requirement of assimilating health related data of all dams in India, and also analyzing such data for effecting timely remedial measures. Owing to sheer numbers and heterogeneity of dams in India, this is going to be nothing but a Herculean task. Further, such an exercise needs to be carried out jointly by the whole water-resource-engineering-fraternity of our country, without losing any time, and in a transparent manner. The proposed DHARMA software - with its broad-based knowledge pool, standardization, dynamism and scalability – will go a long way in fulfilling the desired objective.

Notes:

1 As per International Commission on Large Dams (ICOLD) Standards, a large dam is one with a maximum height of more than 15 metres from its deepest foundation. A dam having height between 10 and 15 metres is also classified as large dam provided: (a) its length is more than 500 metres, or (b) its reservoir capacity is at least 1 million cubic metres, or (c) the maximum flood discharge through its spillway is greater than 2000 cumec.

2 As per latest information available for 'National Register of Large Dams' there are 98 large dams in India which are more than 100 years old.

3 Presently, a full-fledged Chief Engineer heads the Dam Safety Organization in CWC with five specialized Directorates, namely: (i) Dam Safety Monitoring Directorate, (ii) Dam Safety Rehabilitation Directorate, (iii) Software

Management Directorate, (iv) Instrumentation Directorate, and (v) Foundation Engineering & Special Analysis Directorate.

4 *Few of the important rehabilitation cases handled in CWC include: (i) Mulla Periyar dam, (ii) Konar dam, (iii) Hirakud dam, (iv) Barna dam, (v) Talkalale dam, and (vi) Rihand dam.*

5 *So far, 'Dam Safety Organization' cells have been established in: Andhra Pradesh, Bihar, Chattisgarh, Gujarat, Jharkhand, Karnataka, Kerala, Madhya Pradesh, Maharashtra, Orissa, Rajasthan, Tamil Nadu, Uttar Pradesh, West Bengal, Bhakra Beas Management Board, Damodar Valley Corporation, Kerala State Electricity Board, National Hydroelectric Power Corporation.*

6 *So far, about 13 consolidated reports on 'Dam Health' have been received in CWC from various DSOs for the years 2005 and earlier period.*

7 *The Government of India, in August 1982, had constituted a Standing Committee to review the existing practices of inspection/ maintenance of dams and allied structures in various states, and to evolve standard guidelines for the same. This Standing Committee published its report on "Dam Safety Procedures" in July 1986; and the guidelines laid in the report are required to be followed by all states/ organizations in overseeing the dam safety activities.*

8 *A World Bank assisted project titled "Dam Rehabilitation and Improvement Project" (DRIP) is expected to be taken up shortly targeting 382 large dams at an estimated cost of Rs.2029.73 crore. The Project is proposed to be implemented in the States of Andhra Pradesh, Bihar, Chhattisgarh, Gujarat, Jharkhand, Kerala, Madhya Pradesh, Maharashtra, Orissa, Tamil Nadu, Uttar Pradesh, Uttaranchal and West Bengal.*

9 *The tags used in 'Dam Identification Code' for various states are: ANDHRA PRADESH(AP), ARUNACHAL PRADESH(AR), ASSAM(AS), BIHAR(BR), DELHI(DL), CHATTISGARH(CG), GOA(GA), GUJARAT(GJ), HARYANA(HR), HIMACHAL PRADESH(HP), JAMMU & KASHMIR(JK), JHARKHAND(JH), KARNATAKA(KA), KERALA(KL), MADHYA PRADESH(MP), MAHARASHTRA(MH), MANIPUR(MN), MEGHALAYA(ML), MIZORAM(MZ), NAGALAND(NL), ORISSA(OR), PUNJAB(PB), RAJASTHAN(RJ), SIKKIM(SK), TAMIL NADU(TN), TRIPURA(TR), UTTARAKHAND(UA), UTTAR PRADESH(UP), WEST BENGAL(WB), ANDAMAN & NICOBAR(AN), CHANDIGARH(CH), DADRA & NAGAR HAVELI(DN), DAMAN & DIU(DD), LAKSHADWEEP(LD), PUDUCHERRY(PY)*

10 *The code for already registered DSOs are: Andhra Pradesh(01), Bihar(02), Chattisgarh(03), Gujarat(04), Jharkhand(05), Karnataka(06), Kerala(07), Madhya Pradesh(08), Maharashtra(09), Orissa(10), Rajasthan(11), Tamil Nadu(12), Uttar Pradesh(13), West Bengal(14), Bhakra Beas Management Board (15), Damodar Valley Corporation (16), Kerala State Electricity Board(17), National Hydroelectric Power Corporation (18)*

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Chapter-11
Prioritization of Projects for Rehabilitation

1.0 General

Dams provide immense social and economic benefits; and such benefits include water supply - for drinking, irrigation and industrial uses - flood control, hydroelectric power, navigation, fisheries etc. However, dams also present a risk to public safety, economic infrastructure and the environment.

These risks are associated with dam failures, and sometimes also with its *misoperations*. This degree of risk relates to two factors, namely, the likelihood of a dam failure, and the extent of damage and deaths it would cause. A systematic approach for identification of weak elements of a complex dam system, estimating their vulnerabilities, and assessing the extents of hazards posed by them can help us to make a judicious assessment of the risks associated with each dam.

The risk assessments of projects - with inputs from likelihood of failure as well as from consequences of failure - can help in prioritization of rehabilitation measures for the large numbers of ageing dams in the country. This approach also helps in shifting focus of rehabilitation from purely structural (i.e. engineering) measure to both structural and non-structural measures.

1.1 Ageing of Dams & Need for Prioritization

Although dam failures have been so far infrequent, the factors of age, construction deficiencies, inadequate maintenance, extreme weather, or seismic events can contribute to its likelihood. However, extreme seismic events (exceeding the assumed seismic magnitudes for design of structures) and weather events (leading to larger floods than the assumed ones) are difficult to predict, and hence may not be of much help in assessing the odds of dam failure. On the other hand, age is a leading indicator of dam failure. In particular, the structural integrity and operational effectiveness of dams may deteriorate with age; and in most case, older dams do not comply with the updated dam safety standards.

The reasons for dam failure can be several, and they may get ingrained at any point of time over the prolonged life cycle of dam. Thus, cause of failure may get implanted at investigation stage, design stage, construction stage or the post-construction (operational) stage of the dam. However, the cause may fructify only after a prolonged period; and thus it may get attributed to ageing of the dam.

Even though portrayed as modern-temples of India by Pandit Jawahar Lal Nehru, the dams have been in existence since ancient times. The 24 m high earthen dam of *Thonnur Tank* in Karnataka is over 1000 years old, and it is still in use. Besides, there are 12 other large-dams which are over 200 years old; and altogether 117 large-dams that are over 100 years old. Post independence, a substantial number of dams were added up in the early five-year plan periods, to meet the needs of irrigation, drinking water, hydro-power and supplies to municipalities and industries. While full information on medium and small dams – numbering several thousand – is yet to be catalogued, India has over 5100 large-dams, of which about 74% are more than 20 years old (refer Table 1). These aging dams present a grave threat to the lives and economies of the downstream populations.

Table 1
Age-wise profile of India's large dams

SN	Age-group of Dams	Number of Dams	Cumulative No.	Cumulative %
1	1000 years old	1		
2	200-500 years old	12	13	
3	100-200 years old	104	117	
4	Exact age unknown, but more than 100 years old.	10	127	2.49%
5	80-100 years old	144	271	5.31%
6	60-80 years old	93	364	7.13%
7	40-60 years old	672	1036	20.31%
8	20-40 years old	2512	3548	69.56%
9	Exact age unknown, but more than 20 years old.	202	3750	73.51%
10	5-20 years old	899	4649	91.14%
11	Less than 5 years old, or under construction	452	5101	100.00%
	Total:	5101		

(Data Source: National Register of Large Dams, CWC, 2009)

With the large number of dams becoming old, the likelihood of dam failures in India is understandably on an ascending path. And the likelihood of dam failures has been further aggravated by the fact that the large numbers of ageing dams lacked the supervision and maintenance needed for guaranteeing the structural safety and the operational integrity to prevent possible failures.

To reduce the risk of dam failures, regular health inspections are necessary to identify the deficiencies. And, where ever severe deficiencies are observed, comprehensive rehabilitation measures are required to be taken. However, owing to the large chunk of ageing dams in India, this exercise is skewed with the possibility of spreading the limited financial resources too thinly – with no meaningful results, and perhaps, with detrimental effects. To overcome this situation, there is an urgent need for prioritization of ageing dams for rehabilitation purpose.

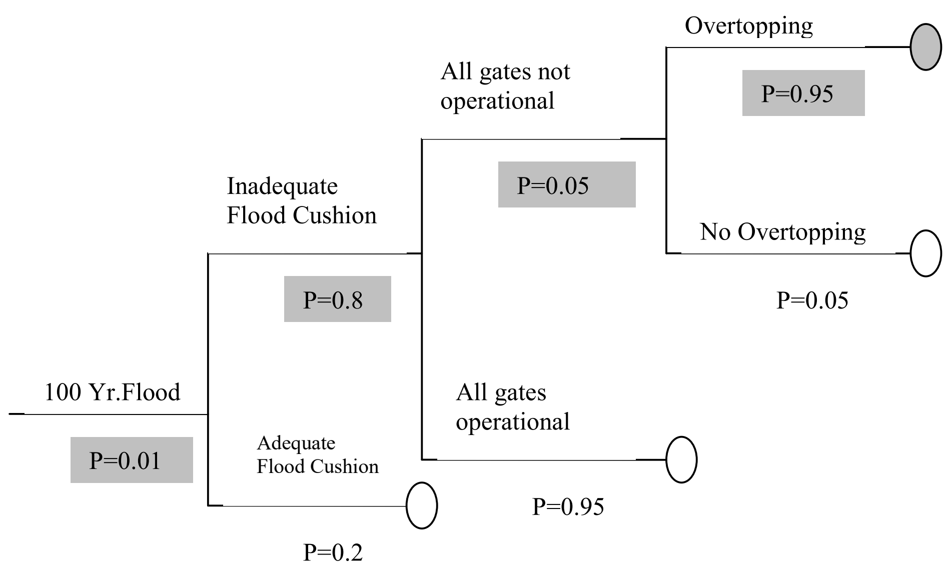
1.2 Likelihood of Dam Failure

There can be many failure modes to which a dam may get subjected to; and some such failure modes are: (a) overtopping and slicing, (b) breaching of dam (c) joint failure, (d) hydro-mechanical failure, (e) internal erosion and piping, (f) slope instability, (g) structural instability due to high uplift pressure etc.

The likelihood of a dam failure can be assessed for different possible failure modes by identifying the series of events leading to such failures, and estimating the occurrence probabilities of each event. For example, the mode of failure in case of a typical earth dam may be the overtopping of dam; and events that may contribute to this failure may include: (a) occurrence of an extraordinary storm, (b) possibility of inadequate flood cushion in reservoir, (c) incidence of one or more gates becoming non-operational, and so on. The overall probability of dam failure - as an outcome of the series of events - will be the product of probabilities (of occurrence) of each event; and this can be expressed in the form of an 'event tree' as shown in Figure below:

Figure -1

Event tree for a typical case of overtopping of dam



1.3 Consequences of dam failure

Consequences of dam failure can be many, but all of them may not be quantifiable or measurable in monitory terms. While damages to the properties and livestock of people and the infrastructure facilities may be quantifiable and measurable in economic terms, the loss to lives of people - even if quantifiable - may not be measurable in economic terms. On the other hand, environmental and social impacts or the political consequences of a dam failure are not at all quantifiable.

The extent of damages and deaths that can be caused by a dam failure will depend upon the size and capacity of the dam, and also on the enormity of population downstream of dam. To quantify the potential harm associated with a dam's failure, the Central Water Commission in its 'Guidelines for Safety Inspection of Dams' (CWC, 1987) has formulated a hazard potential classification involving three hazard categories as given in Table-2. These hazard categories (Low, Significant, and High) do not indicate the likelihood of dam failure; but on the basis of size and location of dam, they reflect the extent and type of damage that the dam failure would cause.

Table 2
Hazard Potential Classification

Category	Loss of Life	Economic Loss
Low	None expected (Non-permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
Significant	Few (No urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than few	Excessive (Extensive community, industry or agriculture)

Though hazard classification is an important and vital parameter for the risk assessment of new as well as ageing dams; this exercise has been somehow overlooked by most of the dam owners and dam safety organizations in India. The disregarding of such vital issue concerning dam safety is happening apparently on three counts. Firstly, there is an apprehension of generating unwarranted fear in the minds of people in the vicinity of dams. The second reason perhaps concerns with the possibility of providing extra ammunition to the ever active anti-dam lobbies. The third reason for the non-implementation of potential hazard classification is apparently the vagueness of the classification itself.

However in present times of increased public awareness, the above apprehensions are rather overtly exaggerated. It is also possible to arrive at a more precise classification of hazard potential, which should also be reviewed periodically. Evidently, the hazard potential will depend upon the size and capacity of dam, and these are practically static factors. However, the hazard potential will also be governed by the population density and extent of developments (urbanization, industrialization, commercial facilities, highways, railroads etc) in the areas affected by a probable dam failure. Since these factors are subjected to progressive change, the hazard potentials of ‘low’ and ‘significant’ classes will require periodical reviews; and these shall be taken up every 10 years.

1.4 Risk Assessment – A Tool for Prioritization

The risk of dam failure is a product of probability of dam failure and the measure of its consequences. It is merely a number with meaning for its relative value, and has no use for its absolute measure.

As discussed earlier, all consequences of failure may not be quantifiable or translatable into economic terms. However for the purpose of risk assessment, an estimation of the damages in terms of loss of lives and loss of property can be made in a systematic manner. While the loss of property can be estimated in economic terms directly, the economic measure of loss of lives pose serious problems. Even here, measures such as estimation of compensation for accidental deaths, or insurance cover for loss of life etc. can be judiciously applied to arrive at a meaningful estimate of consequence of dam failure.

Despite the limitations of risk measurements, the process of 'risk assessment' is an important tool for the dam-safety management with following advantages:

(a) It provides a comparative measure for comparing the risks associated with different dams and their respective modes of failure.
(b) The risk comparison helps in setting the prioritization of dam safety and dam rehabilitation actions.
(c) The systematic process of risk assessment allows for the identification of vulnerable elements of the complex dam system; and this in turn helps in affecting the timely remedial measures.
(d) It gives clear understanding about 'chances of failure' and 'consequences of failure'.

Another advantage of seeking prioritization of projects through risk assessment is that it helps in shifting focus from structural measures for minimizing chances of failure, to the non-structural measures for scaling down consequences. The structural measures, being purely engineering, leads to a high sense of confidence, which is often misleading. Besides, the theoretical analyses are always subjected to assumptions and interpretations with possibilities for grave omissions. The uncertainties associated with structural approach are often hidden under layers of 'Safety Factors', which only jacks up cost of the rehabilitation. In case of ageing dams, this approach further suffers on the ground that we are often dealing with an era whose analysis and design concepts are not in tune with modern concepts, and we have limited knowledge about geological/ foundation features, quality of material, construction techniques etc.

Also, the focus of structural measure is always on the worst case scenario, while less intensive possibilities are ignored even if they are more frequent and cause severe consequences. Thus, with focus on structural measures, a dam may be found to be safe for Probable Maximum Flood (PMF) condition, while there may be recurring flood situations downstream of dam owing to constrained carrying capacity of the river.

Understandably, the 'dam safety' is not merely an engineering problem. It is a managerial problem involving decision making under uncertainties with associated risks. Besides setting the prioritization, the risk assessment approach helps in understanding the explicit tradeoffs of risks, costs and benefits. It helps in understanding the deficiencies in risk based manner, and creates options for such managerial decisions as: (i) accept the dam without major remedial measures; (ii) modify the dam with substantial remedial measures; (iii) alter dam operational parameters; (iv) implement non-structural measures or a combination of structural and non-structural measures; (v) decommission the dam; (vi) reconstruct the dam.

foundation. The analysis of a dam which indicates excessive settlement or liquefaction due to earthquake, the remedial measures can be incorporated and dams are rehabilitated. The case study of Dudhawa dam and the case history of Bhuj earth dam has revealed that safety, stability and strengthening of embankments can be achieved by selecting type of soil, determining its properties, performing seepage, static and dynamic analysis, also by monitoring instrumentation data.

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Chapter - 12

Hydrological Safety Evaluation of Dams

1.0 Introduction

Hydrological inputs play a very vital role in planning of any water resources development project. Hydrological studies are required at all the stages of project formulation, implementation and operation as follows.

- Pre-feasibility stage;
- Preparation of detailed project report (DPR);
- Planning and design;
- Execution of the project; and
- Operation and maintenance of the facility.

Hydrological studies are usually required to cover the following aspects.

- Resource availability i.e. the assessment of quantities of available water and its time variation;
- Safety of project in the event of external flood i.e. estimation of design flood and
- Life of the project i.e., the assessment of the incoming silt load trapped and its distribution in the reservoir for estimating the effect on the live storage and the useful life of the project.

In any river valley project, dams are usually the key structures designed to impound water for irrigation, power generation, water supply, flood control, recreation etc. Dams are expected to safely withstand the forces created by impoundment of water over a long period. The sudden release of the impounded water in the event of dam failure causes tremendous destruction of life and property in the downstream. Therefore, the proper and safe functioning of a dam is an extremely important matter of economic benefits and public safety and as such proper selection of design flood value is of great importance. While a higher value results in the cost of hydraulic structures, an under-estimated value is likely to place the structure and population involved, at risk.

In India, there are about 4000 completed large dams and 500 under construction, out of which about 68 are more than 100 years old, 1000 are more than 35 years old, 2300 being older than 25 years, and 3500 older than 15 years. All or most of these dams come under the category of public civil works facility and present a degree of risk to loss of life and/or damage to property, should any one of them fail. Inadequate flood handling capability and improper operation of spillways are found to be the most common cause of dam failures as per data of ICOLD.

As most of the dams in India are more than a decade old, the original flood peaks/volumes have been estimated either from only empirical formulae by experienced designers or based on the scarce records of observed flood or extreme rainfall events of that time, which need to be either supported or reviewed based on the in situ additional scientific data collected thereafter as well as the techniques and procedures that got devised from time to time.

Checking and upgrading the hydrologic capability is thus a key technical issue in the national dam safety evaluation programme and computation/review of inflow design flood is the first stage in this sequence. This paper briefly discusses the important aspects that need to be considered while evaluating/studying the hydrological safety of the dams.

1.1 Design Flood

Most important criteria for safety evaluation of the dam and other hydraulic structures is the evaluation of design flood, which is discussed below:

1.1.1 Definition

Since absolute flood protection is unrealistic, rational design of hydraulic structures must take into account the risk of flooding and consequent damages. For design purposes, it is necessary to define a flood corresponding to the maximum tolerable risk. The flood, called the design flood, is defined as the flood hydrograph or the instantaneous peak discharge adopted for the design of a hydraulic structure or river control after accounting for economic and hydrological factors. It is a flood that the project can sustain without any substantial damage, either to the objects that it protects or to its own structures. The risk of damage is equivalent to the probability of occurrence of flood larger than the design flood (WMO, 1994).

The design flood, also known as Inflow Design Flood (IDF) is the largest flood that is selected for design or safety evaluation of the structure. The value of the design flood should increase with increasing consequences of the failure of the structure. Therefore, in the simplest way, design flood may be defined as the "flood adopted for design purpose". It may be the probable maximum flood or the standard project flood or a flood corresponding to some desired frequency of occurrence depending upon the standard of security that should be provided against possible failure of the structure (BIS, 1971).

Probable Maximum Flood (PMF) is the flood resulting from the most severe combination of critical meteorological and hydrological conditions that are reasonably possible in the region, and is computed by using the maximum probable storm which is an estimate of the physical upper limit to storm rainfall over the basin. This is obtained from storm studies of all the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions (BIS, 1971).

Flood of specific return period is estimated by frequency analysis of the annual flood values of adequate length. Sometimes when the flood data is inadequate, frequency analysis of recorded storm is made and storm of a particular frequency applied to the unit hydrograph to derive the design flood. This flood usually has a return period greater than that of the storm (BISA, 1971).

1.1.2 Design Criteria

Even though it is desirable to design all hydraulic structures to withstand the largest possible loads/load combinations possible, such as Probable Maximum Flood (PMF), Maximum Credible Earthquake (MCE), the most severe uplift pressure, etc., normally, attempt is made to economise the design by selecting loads equal to or below these upper levels, due to constraints in availability of resources. Due to high risk involved in failure of water resources storage structures like dams etc., various criteria are being followed world over for selection of design flood for fixing the top level of the dam and the design of the dam structure.

In India, till 1985, the criteria for deciding the design flood for fixing spillway capacity of dams was based on the CWC recommendations given in "Recommended Procedures for Design Flood Estimation", Published by CWC in 1972. In 1985, after prolonged deliberation in the concerned committee, BIS issued a new code, namely IS 112233-1985, "Guidelines for Fixing Spillway Capacity", for deciding the type of design flood to be adopted for spillway design etc. The above code lists four different types of design flood in respect of dams, viz.

1) Inflow design flood for the safety of the dam
2) Inflow design flood for efficient operation of energy dissipation works
3) Inflow design flood for checking acceptability of extent of upstream submergence
4) Inflow design flood for checking acceptability of extent of downstream submergence.

As per the above code, dams are classified as small, intermediate and large with respect to two parameters, viz., the storage behind the dam and hydraulic head (from normal or annual average flood level on the downstream to the maximum water level) of the dam. The provisions of the code are briefly given below in Table 1.

Table 1 - Safety Standard for selection of Design Flood followed in India as per BIS

Classification	Gross Storage	Hydraulic head	Design Flood
Small	>0.5 and <10 Mm^3	>7.5 and <12 m	100 year flood
Intermediate	>10 and <60 Mm^3	>12 and <30 m	SPF
Large	>60 Mm^3	>30 m	PMF

The overall size classification for the dam would be the greater of that indicated by either of the two parameters above.

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

(i) Distance to and location of the human habitations on the downstream after considering likely future developments.
(ii) Maximum hydraulic capacity of the downstream channel at a level at which catastrophic damage is not expected.

Each site is individual in its local conditions and evaluation of cause and effects. While the norms mentioned above may be taken as general guidelines, the criteria could be varied in special cases where the same are justifiable on account of local conditions, and keeping in view the hazard potential, as indicated below:

a) Where studies or judgment for important dams indicate an imminent danger to present or future human settlements, the PMF should be used. Dam break studies may be carried out as an aid to the judgment in deciding whether PMF need to be used. Any departure from general criteria as above on account of larger or smaller hazard should be clearly brought out and recorded.
b) When a dam capable of impounding a huge volume of water is constructed above an area having extensive community, industry and agriculture, a distinct hazard from a possible failure is created. The failure of a dam so located would have disastrous effects. The spillway capacities and free board allowances of such a dam should be adequate to insure against failure of the dam during the most severe flood or sequence of floods considered reasonably possible, irrespective of the apparent frequency of occurrence of controlling conditions.
c) In determining the spillway capacity of relatively low dams, which do not have large storage capacity, where possible failure would not result in serious danger to life and property, a less severe condition may be adopted.

The selection of spillway capacity would be governed by overall economic considerations, such as cost of replacing the dam, maintenance and loss of revenue when the project is impaired. The margin of safety must be made consistent with economic analysis.

1.2 Design Flood Criteria Followed in Other Countries

Brief review of the criteria and practices followed in other countries with regard to review of hydrologic safety of existing large dams is presented below:

1.2.1 United States of America

In United States of America, dams are classified into three categories depending upon the height and storage and the hazard potential due to dam failure. The size classification is more or less the same as that adopted in Indian Standard IS 11223-1985 and the recommended standards for the selection of design floods are given in Table 2.

Table 2 - Safety Standard for selection of Design Flood followed in USA

Hazard	Size	Safety Standard	
Low	Small	50 to 100 year flood	
	Intermediate	100 yr. Flood to SPF	* Replacing PMF by SPF can be considered for concrete dams.
	Large	SPF to PMF*	
Significant	Small	100 yr. Flood to SPF	
	Intermediate	SPF to PMF *	
	Large	PMF	
High	Small	SPF to PMF *	
	Intermediate	PMF	
	Large	PMF	

1.2.2 Australia

In Australia the definition of large dams is as given by ICOLD and selection of design flood is based on incremental flood hazard category as given in the Table 3.

Table 3 - Criteria for Selection of Design Flood in Australia

Incremental Flood	Hazard Category	Design Flood
High	Loss of life, extreme damage	PMF to 1 in 10,000
Significant	Unlikely loss of life, Significant damage	1 in 10,000 to 1 in 1000
Low	No loss of life, minor damage	1 in 1000 to 1 in 100

1.2.3 Switzerland

As per the practice being followed in Switzerland, the safety of the dam against floods is based on 1 in 1000 year flood impinging the reservoir at the normal FRL. However, the design condition of 10% gates inoperative is not considered and even head race tunnels are considered as contributing to evacuation of flood discharge.

1.2.4 Norway

As per the practice followed in Norway, SPF or 1 in 100 year flood is used as the design flood for design of spillway and outlet works and PMF is used as the basis of calculations to check the safety of the dam against failure.

1.3 Approaches for Design Flood Estimation

The design criteria, as already indicated earlier, specifies estimation of either or both of the following mentioned floods for consideration.

- Flood of a selected return period
- Standard project flood (SPF) or Probable Maximum Flood (PMF)

The designer is mainly concerned with the peak value of the flood for many structures like bridges, barrages/weirs, cross drainage structures and small storage structures. On the other hand the volume of flood as well as the shape of the flood hydrograph are also needed while designing major/intermediate storage structures.

Various methods commonly employed for design flood estimation are

(i) Empirical flood formulae,
(ii) Enveloping curves,
(iii) Statistical approach, commonly known as Flood Frequency Approach and
(iv) Rational method involving hydro-meteorological approach, commonly known as the Unit Hydrograph Approach

For approaches (iii) and (iv), adequate data inputs are required for processing and obtaining the design flood outputs. The inputs are generally the long term and short term rainfall and runoff records, annual flood peaks series, catchment or physiographic characteristics etc. The detailed methodology to be adopted in a particular case depends upon the data availability as indicated in Fig.-1.

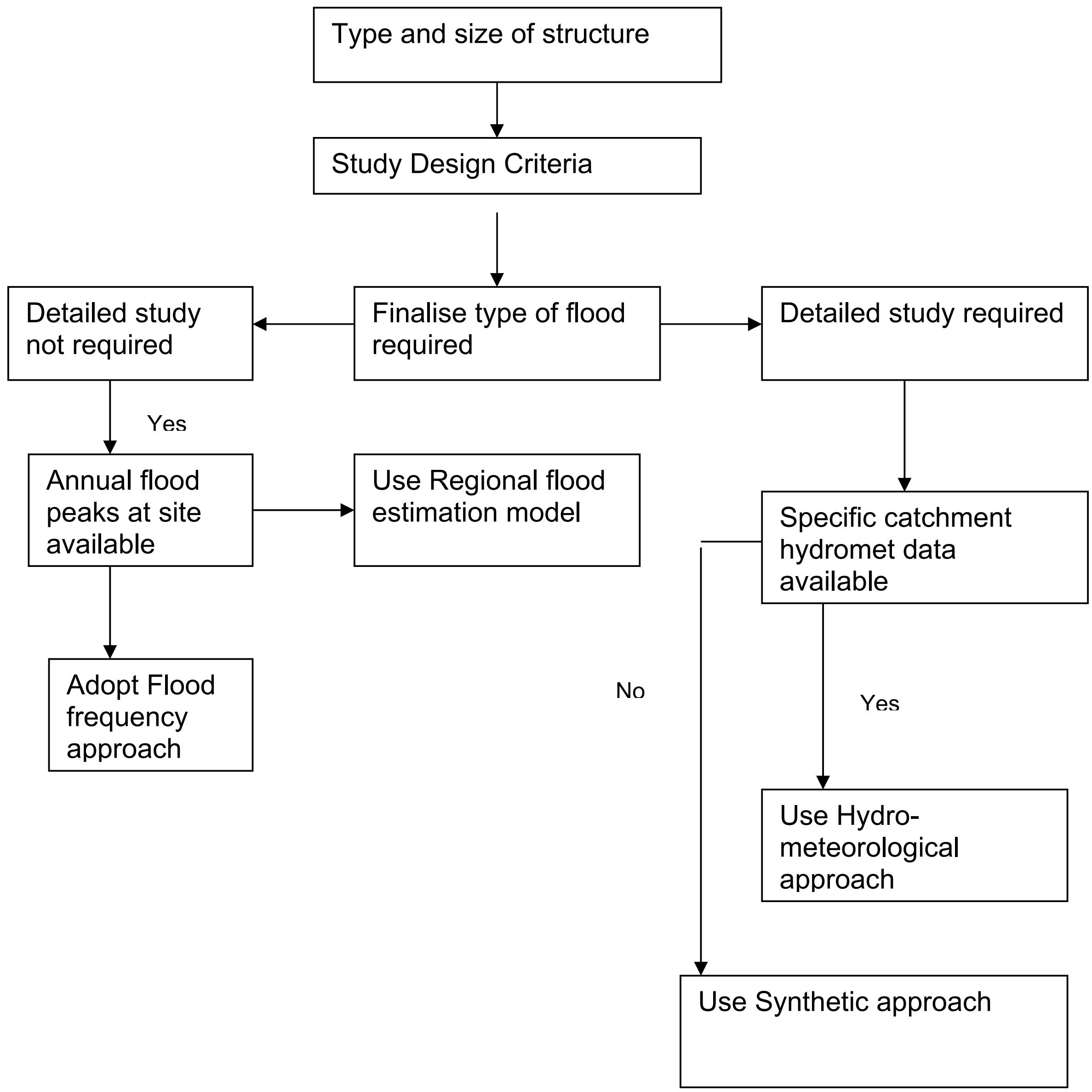

Fig 1- Selection of Method for Flood Estimation

Hydro meteorological approach preferably based on site specific information is suggested for the estimation of design flood of intermediate and large dams. Brief description of above mentioned approaches is given below.

1.3.1 Empirical Flood Formulae

It is a well known fact that hydrology is not an exact science and it continues to be so as it is heavily dependent on over simplified models. The science of hydrology, therefore, was largely empirical as physical basis for most quantitative hydrologic determinants was neither well known nor were many research program of any consequences conducted to produce quantitative information for use by hydrologists and engineers for solving practical problems. During the period 1900-1930, empiricism in hydrology became more evident. During this period hundreds of empirical formulae were developed by deriving regional values arrived at on the basis of statistical correlation of observed flood peaks. Some of the commonly used formulae are summarized in Table – 4.

Table 4 - Some of the Commonly Used Empirical Formulae

Sl. No.	Name	Formula (in metric unit)	Region for which applicable	Value of Co-efficient
1.	Dicke n	$Q = CA^{3/4}$ Q in cumec A in sq.km.	North Indian plains, North Indian hilly regions, Central India, Coastal Andhra, and Orissa	6 11 to 14 14 to 28 22 to 28
2.	Ryves	$Q = CA^{2/3}$ Q in cumec A in sq.km.	Area within 80 km from east coast Area within 80-2400 km from coast Limited area near hills	6.8 8.3 10.0
3.	Inglis	$124A(A+10.4)^{1/2}$ Q in cumec A in sq.km.	For Maharashtra region	-

Most of the formulae have area of the basin as the independent variable. The selection of the formula and the value of the co-efficient depended mainly upon the judgment and experience of the user. Further, these formulae cannot be used with any distinction to estimate flood of various frequencies as may be required by the design criteria to be adopted for different type of structures. Further, these formulae did not consider rainfall characteristics, which, undoubtedly, play a very important role in any flood formation process. The major drawbacks of all these formulae are, (i) they are highly subjective in the selection of certain coefficients (ii) they are applicable only for the region for which it was derived and (iii) it takes into account only some flood producing characteristic of the basin, like area, vegetation etc.

Attempts were made to estimate the design flood (which was earlier considered to be the peak rate of run off) that would occur due to storm rainfall of a given frequency and specified duration on a rational basis and this led to development of Rational Formula. The Rational Formula used by many design engineers is expressed in terms of the following equation:

$$Q = CIA$$

where, Q is the peak discharge, I is the uniform rate of rainfall intensity for a duration equal to or greater than the time of concentration, and A is the drainage area.

1.3.2 Envelop Curves

Studies towards the development of a more rational method by utilizing the data available for a number of basins led to the theory that the maximum floods per unit area experience in one basin is quite likely to be experienced in nearby basin in the same region having same climatological and physiographic characteristics. Based on this theory, observed maximum discharges for a number of catchments in homogeneous hydro-meteorological regions were used to develop envelope curves.

In early fifties, Kanwar Sain and Karpov collected data of various Indian rivers and drew two envelope curves one to suit basins of southern India and the other for those of northern and central India.

The PMF figures for a number of projects estimated by Central Water Commission and other organizations during the period 1980-81 have been utilized for developing envelop curves for PMF peaks. The upper envelope, drawn corresponding to the envelope of World Record Floods, and average line and lower envelope, which were drawn are shown in Fig. 2. These curves correspond to following equations:

Upper envelopes	$Q_u = 1585\ A^{0.35}$
Average line	$Q_{av} = 398\ A^{0.425}$
Lower envelope	$Q_l = 100\ A^{0..5}$

where Q is PMF in cumec and A is catchment area in sq km.

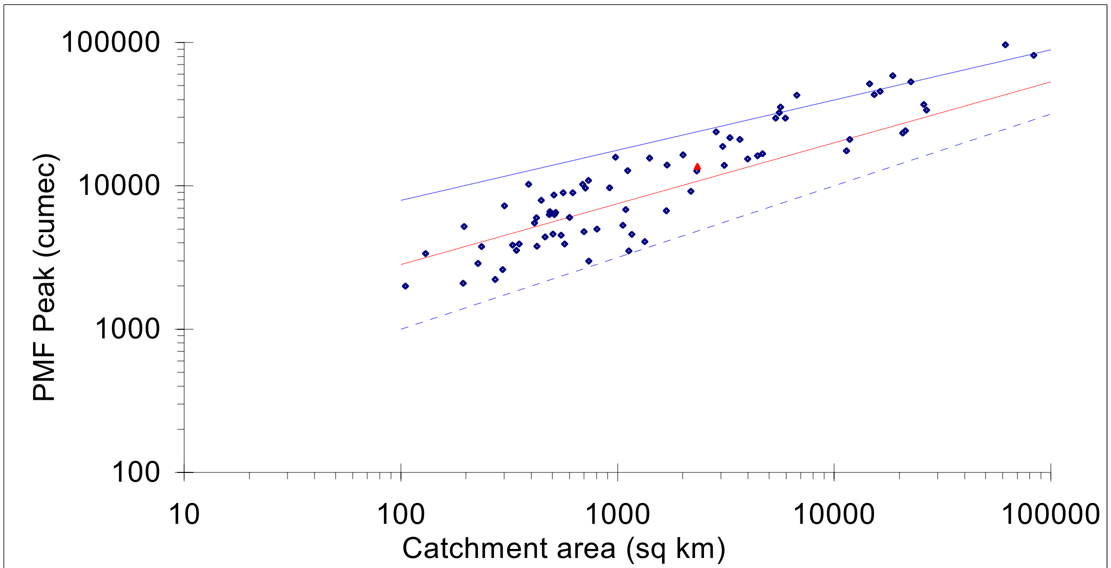

Fig 2 - Envelope Curves for PMF Peaks

These curves have been recommended to be used for prioritizing the existing large dams for further detailed hydrological investigations for dam safety assurance.

1.3.3 Statistical (Flood Frequency) Approach

For the design of spilling arrangements for barrages and small dams, design flood of certain frequencies are adopted. These frequencies are decided taking into account various factors like storage behind such dams, height of dams and importance of project etc. Frequency analysis can be used as a check on the results of the hydro meteorological approach.

Frequency analysis may be performed on the annual flood peak series either directly observed at the site of study or estimated by the suitable method. Alternatively frequency analysis can be performed on the available annual storms record of the project area or for the region. The methodology of Flood Frequency approach is available in any standard text book. The method would be common whether annual flood peaks or annual flood peak of annual storm values are dealt with. The use of this approach is confined to stations where available records are adequate and warrants statistical analysis.

The advantages of statistical approach are:

i) specialized services of trained hydro meteorologists are not required
ii) the method can be computerized to a great extent
iii) associated probability estimates are available

However, the method has certain limitations such as:

a. it yields only the peak, not volume or shape of the hydrograph
b. correct inference about the distribution, which fits the sample data for a site, is crucial as different distributions fitted to same data results in different estimated values especially in the extrapolated range and poses problems for the planners for economic appraisal of the project.
c. difficulties in having homogeneous data due to developments like construction of new storage structures, u/s etc.
d. sufficient long data length to allow reliable estimation of population parameters from the sample data is available only at select locations in the country.
e. elements of risk and uncertainty are inherent in any flood frequency analysis.

Extension of the results of the frequency analysis of a station (or point) data to an area requires regional analysis. For the purpose of regional flood frequency approach statistical homogenous region is defined by carrying out a homogeneity test involving all the stations in the region. Within such a region the result of point data analysis can be averaged to represent the frequency characteristics of the whole region. The regional frequency curve thus obtained can be used in estimating flood for un-gauged basins. Though this approach has been tried here and there, it is not advocated for adoption in general if data for hydro-meteorological approach is available, which is considered more rational and complete (as it provides the peak values as well as the hydrograph shape).

1.4 Hydro-meteorological approach

In the hydro-meteorological method, attempt is made to analyse the causative factors responsible for production of severe floods. Thus in this method, the hydrograph of runoff is assumed to consist of a base flow component and a direct runoff component.

The direct runoff component is obtained by convoluting the excess rainfall hyetograph with basin response function. The excess rainfall is that apart of total storm rainfall, which contributes to the direct runoff. Even though many of these components elude precise physical definition, the method is found to be very convenient and sufficiently accurate for practical purposes.

Thus, in hydro-meteorological method, design flood computation involves estimation of a design storm hyetograph, estimation of antecedent catchment/stream conditions and derivation of catchment response function. The catchment response function can be lumped or distributed lumped system model. In the former, a unit hydrograph caused due to unit effective rainfall distributed uniformly over space and time. In distributed lumped system model, the catchment is divided into smaller sub-regions and the unit hydrographs of the sub- regions along with channel and/or reservoir routing functions will define the catchment response function.

The main advantage of the hydro meteorological approach is that it gives a complete flood hydrograph and this allows making a realistic determination of its moderating effect while passing through a reservoir or a river reach. This approach however is subjected to certain limitations such as:

i. There is requirement of long-term hydro meteorological data for estimation of design storm parameters.
ii. The knowledge of rainfall process as available today has severe limitation and therefore, physical modeling of rainfall to compute PMP is still not attempted.
iii. Maximization of historical storms for possible maximum favorable conditions is at present done on the basis of surface dew point data. Surface dew point data may not strictly represent moisture availability in the upper atmosphere.
iv. Availability of SRRG data for historical storms is too poor.
v. Many of the assumptions in the UG theory are not satisfied in practice
vi. Many times, data of good quality and adequate quantity is not available for derivation of UG.

Nevertheless, the hydro meteorological approach has been found to be a useful tool in design flood studies. Hydro meteorological approach preferably based on site specific information is suggested for the estimation of design flood of intermediate and large dams, especially when the storage has a significant effect on modifying the design flood hydrograph as it flows through the reservoir.

Steps involved for flood estimation using hydro-meteorological approach are shown in Fig. 3.

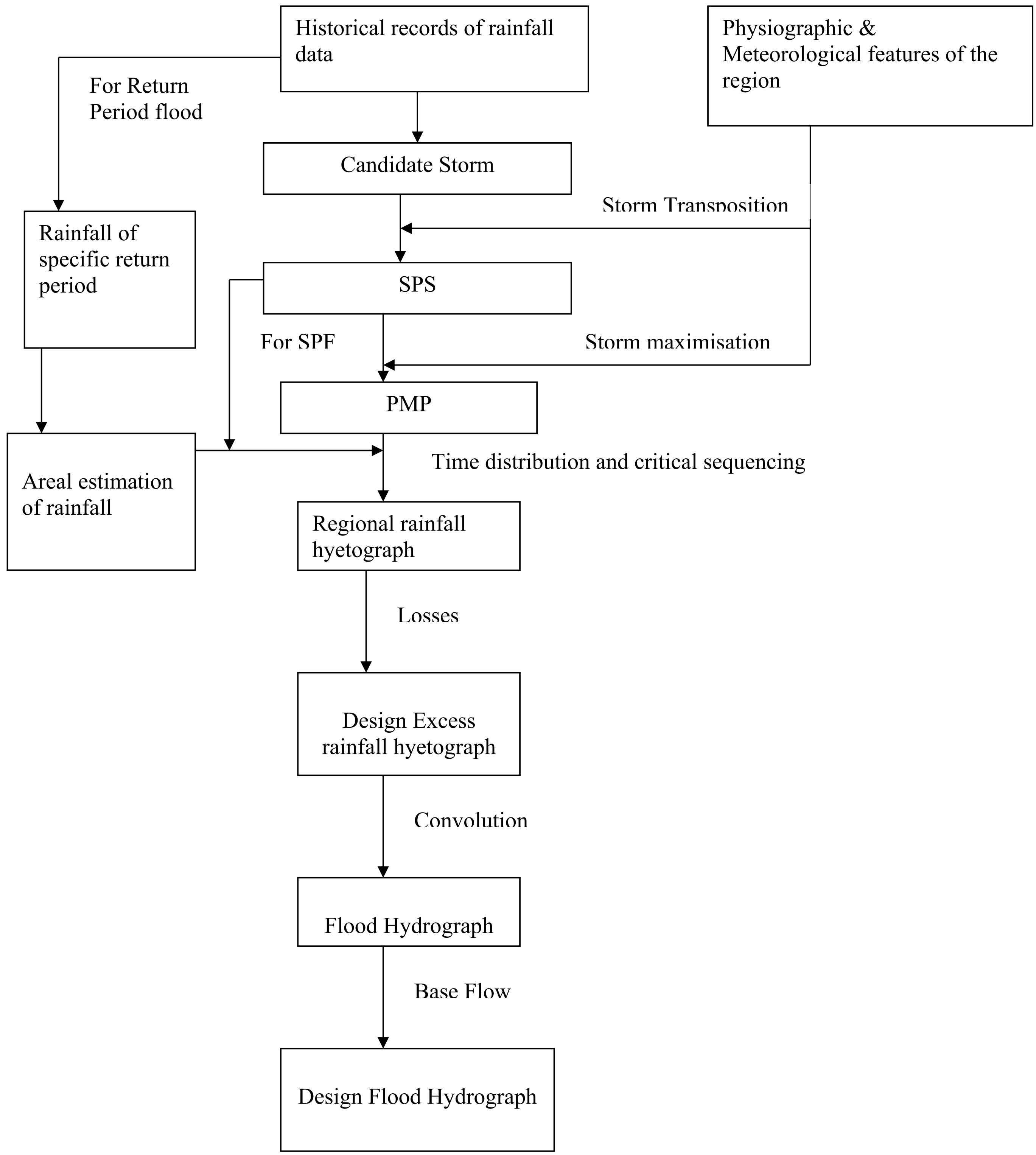

Fig 3- Steps for Flood Estimation by Hydro-meteorological Approach

4.4.1 Use of Generalised PMP Atlases

In the conventional approach for estimation of design storm depths, storm analysis is done for each catchment separately. Since, under the National Dam Safety Project-I, review of a large number of dams were involved, it was considered prudent to develop generalized PMP Atlases, containing ready to use tables/charts for aerial PMP depths, so as to ensure uniformity and consistency as also to speed up the work. These atlases/reports are intended to give a second level screening of existing dams for more detailed analysis for hydrologic capability.

Generalised PMP atlases for the following basins have been completed:

i) Kaveri and Other East Flowing River Basins South of Krishna
ii) Godavari and adjoining river basins
iii) Mahanadi, Brahmani, Subarnarekha and other basins
iv) Sone, Betwa, Chambal and Mahi basins
v) Krishna basin
vi) Indus basin

A "Guideline for use of generalized PMP Atlases" was also prepared in HSO and circulated. PMP Atlas for the remaining basins are proposed to be prepared in the coming years:

4.5 Regional Approach

Generalised studies for determining storm rainfall for specified return period (say 25, 50 & 100 years) and thereupon estimating the design flood of the corresponding frequency render great assistance in quickly arriving at the design flood values of project where detailed studies are not warranted or can not be taken up due to paucity of data. Such regional studies have been carried out jointly by India Meteorological Department (IMD), Research Design and Standards Organisation (RDSO), Ministry of Railways, Ministry of Transport (MOT) and Central Water Commission (CWC). For this purpose, the country has been divided into 7 zones and 26 hydro-meteorologically homogeneous sub-zones. Detailed rainfall studies for each of the regions have been carried out, covering the depth duration frequency analysis, and the design storm components such as conversion factors from point to areal rainfall, short duration ratios and time distribution factors. Flood estimation reports, already published for 23 sub-zones by Central Water Commission, provide the method of the ready to use charts and formulae for computing floods of 25, 50 & 100 years return period of ungauged catchments in the respective regions.

4.6 Hydrologic Safety Evaluation of Existing Dams

As already indicated earlier, there are about 4000 large dams in India out which about 70 are more than 100 years old and 1000 are more than 35 years old. Safety of some the existing large dams was of serious concern, therefore, hydrologic safety evaluation of distressed large dams was taken up as a part of Dam Safety programme. While carrying out review, original hydraulic and hydrologic design assumptions obtained from the project records needs to be assessed to determine their acceptability in evaluating the safety of the dam. All constraints on water control such as blocked entrances, restrictions on operation of spillway and outlet gates, inadequate energy dissipation or restrictive channel conditions, significant reduction in reservoir capacity by sediment deposition and other factors should be considered in evaluating the adequacy of spillway capacity with reference to the inflow design flood.

Studies for review of design flood in respect of about 62 dams in the country were undertaken in CWC in recent past. The results in respect of some of the dams are summarized in Table-5.

Table 5 - Design Flood for Some Dams After Review

Sl. No.	Name of the dam	State	Design Flood (cumec)	
			At planning Stage	After Review
1	Hirakud	Orissa	42474	69632
2	Gandhi sagar	Madhya Pradesh	21200	54390
3	Amravathy	Tamil Nadu	4250	6542
4	Darjang	Orissa	2831	4130
5	Tambrapani	Tamil Nadu	2549	5430
6	Jawai	Rajasthan	1900	6469
7	Kaketo	Madhya Pradesh	1811	5728
8	Vidur	Tamil Nadu	1768	6167
9	Manimuthan	Tamil Nadu	1710	4965
10	Tigra	Madhya Pradesh	1455	4067

From the above table, it appears that the design flood values invariably increase as a result of review studies but it is not so in all the cases. The review studies for design flood for Sapna dam in Madhya Pradesh resulted in reduction of design flood from 500 cumec to about 158 cumec only. Similarly, the review studies in case of Ranganadi project (under construction) in Arunachal Pradesh resulted in reduction of design flood from 13240 cumec to about 9175 cumec.

Yet another interesting case is for Bhakra dam on river Sutlej. The design flood adopted at planning stage was about 11330 cumec. The project was completed in 1963. A peak flood of about 17230 cumec was observed in the year 1971. On the basis of the review studies, the design flood (probable maximum flood) has been tentatively estimated to be of the order of 21850 cumec. During the course of the review studies, it was observed that the changes in the design flood values has resulted due to one or more of the following reasons.

a. Use of empirical methods at the time of planning,
b. Revision in the value of the design storm as a result of availability of additional data/information about severe most storms from hydro-meteorologically homogeneous regions,
c. Adoption of a different temporal distribution pattern for the standard project storm or probable maximum storm etc.,
d. Changes in the response function i.e., unit hydrograph as a result of analysis of more number of flood events or use of improved techniques, and
e. Availability of additional data in respect of observed flood peaks to be used in flood frequency analysis.

Obviously, the revision of design flood is required mainly due to availability of additional data. The changes in the recommended techniques for design flood analysis are mainly with a view to rationalize the process. The upward revision is not necessarily due to adoption of a new technique.

4.7 Limitations of Available Hydrologic Data

The accuracy of estimated hydrological parameters depends on the quality of hydrological and hydro-meteorological data, network density of the stations and the length of record used in the study. The quality of data is all the more important in case of design flood studies because the data observed during the highest flood are considered to be most useful. On the other hand, it is the time when there is a tendency either to skip the observations or to make observations without adhering to the standard procedures. The poor quality of data leads to inconsistency, which many a times forces the hydrologist to abandon the data. In some cases, observed data for entire period are found to be inconsistent. Such a situation arises mainly due to:

a. observational error because of poor knowledge of the observer about the standard procedures,
b. error due to faults in instruments,
c. error in the rating charts or use of current meters etc. without timely ratings,
d. error during computation of discharges from observed variables,
e. filling up the gaps during the period of non-observation without proper analysis and non-reporting of such fillings.

This results in either no data situation or a deficient data condition and the hydrologist has to make a number of assumptions. Yet another major problem is about the length of the data. For most of the projects, a discharge site is established as a part of the investigation and abandoned as soon as the surveys etc. are completed. Obviously, the hydrological analyses are based on limited data condition and the recommendations about hydrological inputs are invariably conditional (to be updated or reviewed at pre-construction stage). It is a well known fact that the preparation of detailed project report (DPR) and the clearance of the project takes considerable time. Many a times, the scope of the project is revised necessitating the review of hydrological analysis. At this stage, no additional data is available and there is no scope for any meaningful review or updating. It is very much desirable to properly plan the network of hydrological and hydro-meteorological stations, establish them as per standards and continue observations at these stations to get specified length of data. As a matter of fact, some representative hydrological stations are required even after the completion of the project for efficient operation and subsequent reviews.

4.8 Hydrological Safety of Existing Dams

4.8.1 Guidelines for Safety Evaluation

The guidelines for determining hydrological safety of existing dams have been included in the "Report on Dam Safety procedures" 1987 published by Central Water Commission. A brief extract of the guidelines is given below:

Every artificial storage can be potential hazard to downstream life and property. All storage dams are provided with spillways, the primary purpose of which is to reduce this artificial hazard to negligible or acceptable level. The decision on spillway capacity of a dam including the decision on its surcharge storage, free board etc. constitutes an important hydrologic and engineering decision affecting the safety of the dams.

The dam safety service in the country has highlighted the need to review the design floods to ensure the safety of the existing dams in general in the light of the existing criteria for the new dams.

In its part 1.1 of scope, it is stated that the existing BIS guidelines/standard is for constructing new spillways and are not for deciding the adequacy of old structures. Thus, there seems to be a need for separate guidelines for deciding adequacy of old structures.

4.8.2 Need for Separate Guidelines

The question of whether guidelines for fixing spillway capacity consistent with the safety of dam should be of a general nature covering both new and existing structures or whether these should be separate for the two types of structures is debatable. An important view is that social equity would demand the same standards of safety for the population in the country irrespective of whether they are likely to be affected by new or old structures in case of hydrological failure. However, new guidelines, which generally are more stringent, can easily be accommodated while planning new projects, which are tagged on with a package of costs and benefits. Another view therefore is that it would be more pragmatic and practicable to have separate guidelines, for review of the old dams.

The fact that a particular dam is not having adequate safety from hydrologic angle may or may not necessitate immediate remodeling to ensure the safety. This again is debatable and different views as indicated above could be taken. In this situation presently the following procedure is being followed:

There shall be two separate guidelines one for existing dams and another for new dams. However, the two guidelines would not differ in the choice of inflow design floods. They may differ marginally in regard to the free board, clearance and safety factors. Thus, although two guidelines would exist these would not differ radically.

A separate guideline for fixing priority amongst the existing dams in regard to their modification for greater safety under floods would be formulated. While it would be desirable to improve the status of each dam which does not conform to the guidelines for existing dams, the guidelines would allow differing action on some dams, in preference to some others in view of constraints on funds and organizations capabilities as they may exist.

The prioritization of the dams not coming up to the norms for interse priority of action could be done on two considerations as follows.

a) On the basis of the largest flood that can be safety negotiated as compared to the appropriate inflow design flood; or,

b) On the basis of the likely structural status (say, factor of safety), which the dam would have in case the inflow design is encountered (with relaxed ambient conditions) as appropriate to the case, as compared to the desirable structural status.

4.8.3 Review of Adequacy of Spillway and Allied Provisions in Existing Dams

It is proposed that these guidelines would be the same as the guidelines for new dams which have already been discussed above and which have also been reflected in the latest Indian Standards. However, while reviewing adequacy of spillway capacity, relaxation of the following ambient conditions may be considered on the merits of each case.

a) For existing dams, impingement level to be considered can be lower than that for new dams, after taking into account a practicable schedule of filling.
b) For existing dams, where a flood forecasting possibilities exist and have been proved in the field, a reasonable pre-depletion may also be allowed, although this is not allowable for new dams.
c) For existing dams, where gate maintenance is very satisfactory, and after making sure of standby arrangements, the design condition of one gate inoperative may not be considered.
d) Relaxation of free board and clearance

For new dams, free board and clearance above MWL are necessary as per the relevant BIS. For existing dams, these could be relaxed on case to case basis. A properly designed solid parapet wall may be considered for the free board.

4.8.4 Prioritization for Modification of Existing Dams for Greater Safety

Since at any time funds for this purpose would be limited and also there could be constraints on organizational capabilities etc., it would be necessary to have a guideline for deciding the priorities in tackling existing dams for greater safety under floods. These could be arrived as follows:

a) Consider an existing project and its spillway and other allied provisions.
b) Work out the inflow design flood as per the BIS Standard for the new dams.
c) Route the flood with ambient conditions as per new dam and check if it is successful. If it is successful no action is required. If not consider if ambient conditions and free board and clearance can be relaxed as method above. If after such relaxation the dam is successful in negotiating the inflow design flow no action is required. If the dam is unsuccessful, the behaviour of the dam under the design flood, such as reduction of free board and clearances, physical overtopping, likelihood of development of tension, likelihood of water hitting the deck bridge etc. may be brought out.

d) By trial studies through proportionate reduction of inflow design flood the percentage of the inflow design flood which can be successfully negotiated by the dam under the relaxed conditions may be worked out.
e) The inflow design flood type depends upon the hazard potential in the eventuality of a failure. In some projects it may be possible to re-allocate the down stream habitation at a safe level in such a way that after such re-allocation the standard project flood may become the relevant inflow design flood for this dam. Such possibilities may be examined, if found feasible. In such cases the safety of the dam

against reduced inflow design flood may be worked out, repeating the steps one onward.

f) After completing such studies for all dams within a State, the extent of inadequacy of the spillway capacity may be classified according to the following:
   - The maximum inflow flood that can be passed through the dam with the relaxed ambient conditions, expressed as a proportion of the inflow design flood.
   - The likely effect of the inflow design flood on safety of the dam, i.e. whether the dam has reasonable chance of not failing under such a flood.

g) Where an existing dam is found to have inadequate hydrologic safety in the review, immediate action would also be initiated to frame an emergency action plan as a disaster prevention measure. Such an action plan would be kept active until the permanent corrective measures are evolved and implemented.

4.8.5 Measures for Handling PMF for Dams with Inadequate Spillway Capacity

After review, if it is found that spillway capacity is insufficient to pass the design flood without encroaching into the freeboard, as per the norms, then various alternatives such as lowering the initial reservoir level, changing the spillway configurations for free board, relaxation of clearance criteria for safely passing the flood etc. can be considered. If these options are not successful then, structural solutions like strengthening the dam etc. and/or non-structural solutions like pre-depletion of the reservoir based on early flood warning system will have to be considered. The solution to be adopted will vary from case to case and following alternatives for example, which are not exhaustive can be considered:

a. Augmenting the existing spillway capacity through addition of more spillway bays of the same type as existing.
b. Provision of breaching sections or fuse plugs. If suitable sites are available it is preferable to locate such breaching sections on a saddle rather than on the main dam section. However, it is required to investigate the alignment of the surplus channel till it meets the main channel to assess the likely damages in the surrounding valley in the event of design flood at dam causing a breaching of the section.
c. Increasing the freeboard above FRL of the dam by provision of parapets including strengthening of sections where necessary so that the flood cushion available will be increased.
d. Early warning system: An early warning system involving flood forecasting by utilizing real time data of rainfall, stage and discharge at upstream stations will greatly help in evacuating the reservoir in anticipation of severe flood inflows into the reservoir so as to obtain higher moderation in peak.

   In cases 2 and 3 above, the routed flow will have a lower peak and from the consideration of d/s flooding this may be a better solution.
e. Increasing the flood storage by lowering the conservation storage level, so that flood moderation will be enhanced. However, this may also result in some reduction in benefit and on the positive side it will involve little investment required for modifications.

A suitable and most favourable alternative is to be chosen by considering the various options that are feasible and working out the relative benefit cost scenario.

4.8.6 Preparation of Emergency Action Plans and Inundation Maps for Downstream Area

The international practices, especially in the United States aim at a programme of assessing the risk to public safety & corrective action to reduce or eliminate such risks from time to time under the Dam Safety Programme. Thus, the effort is to eliminate unacceptable risks but not to provide a risk free environment. Probable Maximum Flood is a theoretical flood value/magnitude that is possible at the dam location as per the data/methodologies available and can occur as a natural flood event if the structure does not exist. There are other concepts that are relevant here, such as the Threshold Flood (experienced as a portion of the PMF) that may cause the dam to fail. Similarly there is also a Base Safety Condition, at which no significant incremental loss of life can be measured between total losses and natural flood losses. This term refers to a hydrologic loading condition beyond which no significant incremental risk to life due to increased flood flow occurs. (By incremental risk is meant the increase in risk to life and property due to the dam compared to the situation when there is no dam existing). These landmark flood values have to be assessed by the state/dam owners before corrective actions are processed for avoiding incremental increase in loss of life. This, aspect specially assumes importance in valleys spanning more than one State where a failure may occur in one State and difficulty is in the downstream State.

However, it needs to be noted that practices developed in USA where the average density is of the order of 2-3 persons per sq km, cannot be directly applied in Indian context, since the population density in our country is very high with an average value of the order of 240 persons per sq km and high values of the order of 700 in Kerala and West Bengal. Solutions specific to our needs are yet to be evolved in the country.

Even though floods of the order of PMF have only very low exceedence probabilities, their possibility cannot be ruled out and in order to counter such emergency situations there is need to adopt both well thought out structural remedies as well as an emergency preparedness plan, which can be set in motion as soon as an event is perceived.

4.9 Conclusions

In the case of dams with inadequate spillways, certain relaxations in assumptions/ambient conditions, to be selected on a case-to-case basis, have been proposed in the relevant Indian Guidelines. In case, the dam is still unable to handle the revised inflow design flood hydrograph, various structural/non-structural alternative solutions may be available for arriving at a remedial action. These have to be examined in each case, giving due considerations to local conditions as also the associated risk and cost considerations. In the case of all large dams, with catchment area large enough to justify the introduction of a flood forecasting system, pre-depletion of the reservoir based on such forecasts, leading to lower flood impingement levels, greatly improves the capacity of the system to handle larger floods. This needs to be examined before committing huge investments that may be required for spillway modification.

Further, since floods of the order of PMF are rare events, it may suffice to provide emergency type spillways to handle higher order flood events.

In any case, it is essential that an emergency preparedness plan is planned, well documented and put in place, for all large dams, to meet the eventuality of the unexpected rare event actually materializing.

In the above paragraphs, only some of the important points regarding essential hydrological aspects in relation to Dam Safety requirements have only been explained. The concepts involved need to be understood properly before adopting these to actual practice, for which a reference to detailed discussions in various relevant references, is essential.

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Chapter 13

Specialized Materials and Methods from point of view of Dam Safety

"Dam, like all structures, will be broken in the end — just as all people will die in the future. It is the purpose of the medicine and engineering to postpone these occurrences for a decent interval".

- J. F. Gordon

1. INTRODUCTION

In India, there are 4857 large dams (National Register for Large Dams (2014), Central Water Commission). These dams have served the country well for the economic stability even in the worst years of drought, floods, cyclones, etc. At the same time, dams are capital intensive projects costing thousands of crores. Since most of optimum locations for construction of dams have already been utilized, conservation of existing dams is very essential. This becomes more important since in the present socio-politico-legal set up making land acquisition a difficult task, construction of new dams is very taxing, if not next to impossible. Further, most of these dams, i.e., 3859 (79.5%) dams are more than 25 years old. Due to ageing, wear and tear, these dams have developed various structural deficiencies and shortcomings in operation and monitoring facilities. Few of them do not meet the present design standards – both structurally and hydrologically. Therefore, it is very essential to ensure safety and continued operation of dams for benefit cost optimization; else their viability and sustainability will become questionable. The safety and continued operation imply that the dams continue to perform their intended functions.

Engineering process involves continuous improvement of technology, both in terms of materials and methods. This applies more to dam safety, since each dam is unique in itself due to uniqueness of its topography, geology, hydrology, etc., and accordingly, dam safety and rehabilitation technique will also required to be innovative with newer materials and methods. In this lecture, an attempt has been made to provide a generalized picture of new materials and methods as applicable to dam safety and rehabilitation. However, practicising engineers need to identify the cause(s) of failure(s) and to suggest and adopt remedial measures, as appropriate on case to case basis.

2. GENERAL REASONS OF DAM FAILURES

The failure of dams can be grouped into following categories

- Planning failure
- Design failure

- Quality control failure
- Ageing / wear & tear Failure

The general reasons of these categories of failure are;

2.1 Planning Failure

The planning failure could be due to any of the following reasons;

- **Inadequate or wrong investigations** – Inadequate or wrong investigations at planning stage yields faulty ground data, which will obviously cause planning failure. For example, topographic survey error in river bed levels at tail race outfall location will lead to inadequate or excessive discharge and lower or higher heads affecting power generation.

- **Improper planning decisions** – Often at the planning stage, all options, i.e., various alternatives regarding location & type of components, equipments, etc., are not properly evaluated resulting into improper planning decisions. These improper planning decisions manifest itself during operation stage of the project resulting into perpetual loss due to poor generation and higher operation & maintenance costs.

- **Improper layout planning** – Layout planning is equally important during planning stage of the project. Improper layout planning due to lack of experience etc., adversely affects the performance of the projects. For example, improper placement of intake structure may not ensure proper drawal of water into the water conductor system.

2.2 Design Failure

The design failure could be due to any of the following reasons;

- **Inadequate or wrong investigations** – The efficacy of design of any structure depends on the adequacy and accuracy of topographical, geotechnical, hydrological, seismological investigations, etc. If the input data is wrong, the design is bound to be faulty and may result into structural failure.

- **Improper design assumptions & calculations** - The efficacy of design of any structure also depends on the appropriateness of assumptions and accuracy in calculations. Inappropriate design assumption result into use of improper theories / equations. In addition or otherwise, if there is a mistake in calculations, only God can save the structure.

- **Ignorance of critical loading conditions** – Often designers, due to inadequate knowledge of actual construction sequence, ignore critical loading conditions, which may result into improper design, and subsequently, a failure.

2.3 Quality Control Failure

Most of the failure is generally attributed to the improper quality control during construction or operation & maintenance. There can be some practical limitations in achieving perfect quality of works. But what is most disturbing is the fact that often site engineers ignore fundamental aspects of engineering, as a result project is doomed to under perform throughout its life. The most common distress situation in hydropower is cracks and seepage (through joints or cracks), which are primarily due to improper quality control. Even the problem of misalignment, due to which hydro-electro-mechanical equipments like gates, cranes or even turbines do not work properly, is due to improper quality control.

2.4 Ageing / Wear & Tear Failure

Ageing / wear & tear failure is due to due to deterioration of quality of any material due to its use with the passage of time and it is a fundamental truth. This is the reason that we have concept of serviceable life of the structure or project. However, proper surveillance & maintenance can reduce the ageing / wear & tear substantially and increase the serviceable life. This aspect is becoming more important as more number of dams is getting older.

3. ANALYSIS OF FAILURES

International Commission on Large Dams (ICOLD) Bulletin 99 (1995) has statistically analyzed causes of Dam Failures on the basis of details of failed dams. The major conclusions are;

(a) The percentage of failures of large dams has been falling over the last four decades; 2.20 percent of dams built before 1950 failed, failure of dams built since 1951 are less than 0.5 percent.

(b) Most failures involve newly built dams. The greater proportion (70 percent) of failures occur chiefly in the first ten years, and more especially in the first year after commissioning.

(c) Foundation problems are the most common cause of failure in concrete dams, with internal erosion and insufficient shear strength of the foundation, each accounting for 21 percent.

(d) With earth and rockfill dams, the most common cause of failure is overtopping (31 percent as primary cause and 18 percent as secondary cause), followed by internal erosion in the body of the dam (15 percent as primary cause and 13 percent as secondary cause) and in the foundation (12 percent as primary cause and 5 percent as secondary cause).

(e) With masonry dams, the most common cause is overtopping (43 percent) followed by internal erosion in the foundation (29 percent).

(f) Where the appurtenant works were the seat of the failure, the most common cause was inadequate spillway capacity (22 percent as primary cause and 39 percent as secondary cause).

4. REHABILITATION TECHNIQUES

Rehabilitation involves carrying out necessary repairs to the structures so as to bring them back to their normal functioning. As already pointed out that each hydropower project is unique in itself due to uniqueness of its topography, geology, hydrology etc., and accordingly, rehabilitation techniques will also require to be unique. The generalization brought out in this lecture is only intended to generalize approaches for rehabilitation of hydropower structures for sake of easy comprehension. Practicising engineers need to identify the cause(s) of failure(s) and to suggest and adopt remedial measures, as appropriate on case to case basis.

4.1 Rehabilitation of structures in distress due to planning failures

Rehabilitation of structures in distress due to planning failures is possible only by *identifying and correcting the mistakes*. This will involve a thorough review of planning and then devising suitable ways of correcting the mistakes, which may involve *addition of any appropriate structure* or *modification of existing layout / structures*. A case study of *Eastern Gandak Canal H.E. Project (Bihar)* is described below to illustrate this point.

Eastern Gandak Canal H.E. Project was constructed during early nineties on the bye-pass channel of Tirhut Main Canal (TMC) on the eastern bank of the Gandak river at Valmikinagar, Bihar. The installed capacity is 3 x 5 MW envisaging annual generation of 64.17 MU at 75% load factor, utilizing a maximum discharge of 320 cumecs. But from the very beginning, the power generation has been very low; annual average generation being 23.54 MU only. This generation loss is alarming, and therefore, a detailed study was done to identify the reasons of low generation and remedial measures thereof. The study revealed that low generation is mainly due to non-availability of adequate discharge due to lower irrigation water requirement in TMC and canal closure, even though there may be adequate discharge in the river. Thus, the project was not properly planned.

An escape channel from tail race channel to river Gandak was, accordingly, planned to remove dependency of discharge in the power channel on TMC discharge improving average annual generation to 58.77 MW.

4.2 Rehabilitation of structures in distress due to design failures

As in the case of planning failures, rehabilitation of structures in distress due to design failures is also possible only by *identifying and correcting the mistakes*. This will involve a *thorough review of geotechnical evaluation, design assumptions & calculations* and then devising suitable ways of correcting the mistakes, which may involve *strengthening* or even *replacing the structure*. A case study of *Lower Jhelum H.E. Project (Jammu & Kashmir)* is described below to illustrate this point.

Lower Jhelum H.E. Project (Jammu & Kashmir) on river Jhelum is located in Uri Tehsil of Baramulla Distt. of J&K. The Jhelum river is draining the entire Srinagar valley. As a result, it carries large amounts of wastes, debris and floating trash. The trash racks need to be cleaned on regular basis. But often cleaning of trash racks is not done, which causes choking of screens developing large differential head. The designer has to take into account this eventuality and design the structure accordingly. Unfortunately, design was not done accordingly and more reliance was paid on the requirement of cleaning of trash racks on regular basis. Result, the entire trash rack structure failed, and the rehabilitation program consisted of rebuilding trash rack structure a fresh.

4.3 Rehabilitation of structures in distress due to quality control failures

As already pointed out, quality control failure during construction and maintenance is the most common reason for distress. Site engineers, often under pressure to achieve fast progress or for other reasons, overlook or compromise with work quality causing permanent damage to the integrity of the structure. Most common distress reasons due to quality control failures observed in dams are cracks, seepage, settlement, deformation & misalignment.

4.3.1 Cracks

Cracks may appear in a hydraulic structure due to several reasons. They may affect appearance only, or they may indicate significant structural distress or a lack of durability. First & foremost requirement is to identify the location and extent of cracking. It should be determined whether the observed cracks are indicative of current or future structural problems, taking into consideration the present and anticipated future loading conditions. This may involve direct / indirect observations and destructive / non-destructive testing. Based on the careful evaluation of the extent and cause of cracking, procedures can be selected to accomplish one or more of the following objectives;

- Restore and increase strength;

- Restore and increase stiffness;
- Improve functional performance;
- Provide water tightness;
- Improve appearance of the concrete surface;
- Improve durability; and/or
- Prevent development of corrosive environment at reinforcement.

Following the evaluation of the cracked structure, one or combination of following repair procedures can be selected;

(i) Epoxy injection
(ii) Routing & sealing
(iii) Stitching
(iv) Additional reinforcement
(v) Drilling & plugging
(vi) Gravity filling
(vii) Grouting (cement / chemical grouting)
(viii) Crack arrest
(ix) Polymer impregnation
(x) Overlay & surface treatment
(xi) Autogenous healing

4.3.2 Leakage / Seepage

Leakage / seepage through hydraulic structure may be due to cracks, erosion / piping or waterstop failures. They may be within acceptable limits indicating normal situation or they may indicate significant structural distress or a lack of durability. As in the case of cracks, first & foremost requirement is to identify source and extent of leakage / seepage. Rate of seepage should be closely monitored as variation in the rate of seepage vis-à-vis water level, unit discharge, etc., provides vital input to identify source and determination of remedial measures. Tracer technique is normally used to confirm the source.

Once the source & extent of seepage are identified, proper dam safety and rehabilitation techniques can be evolved.

(1) **Seepage / leakage through cracks** – In case, seepage / leakage is through cracks, various crack sealing techniques, such as, Routing & Sealing, Grouting, Epoxy injection, Drilling & plugging, Polymer impregnation, and / or Overlay & surface treatment can be used.

(2) **Seepage / leakage due to erosion / piping** – Dissolution of soluble materials or transportation of fine grained / loose materials under high hydraulic gradient lead to erosion / piping. It is mostly observed in case of foundation in soils / jointed rockmass and has a tendency to worsen the situation with time.

Appropriate rehabilitation measures include plugging, grouting, grout curtain, cut-off wall etc.

(3) **Seepage / leakage due to waterstop failures** – Joints are necessary in any civil structure to facilitate construction in stages, to prevent destructive or unsightly cracks, and / or to reduce or eliminate the transmission of stresses. These joints are most vulnerable locations for seepage / leakage in any hydraulic structure. Embedded waterstops are generally provided to stop seepage / leakage through these joints, especially to ensure structural integrity and serviceability as water seeping through these joints deteriorate the structure permanently. But often these waterstops fail due to one or combination of following reasons;

(a) Excessive movement of the joint, which rupture the waterstop;
(b) Honeycomb areas adjacent to the waterstop resulting from poorly consolidated concrete;
(c) Contamination of the waterstop surface, which prevents bond to the concrete;
(d) Puncture of the waterstop;
(e) Breaks in waterstop due to poor or no splice; and/or
(f) Complete omission of waterstop during construction.

All the above reasons are generally due to poor construction practices. Since it is usually impossible to replace an embedded waterstop, grouting or installation of secondary waterstop is the remedial measure most often used. Remedial measures must ensure that waterstops remain capable of accommodating movements parallel to the axis of the waterstop as a result of joint opening or closure and perpendicular to the plane of the waterstop due to differential movement as a result of differing foundation conditions. These are achieved through surface plates, caulked joints, drill holes filled with elastic material and chemically grouted joints.

A detailed description of repair of waterstop failures along with their case studies are given in the US Army Corps of Engineers Technical Report REMR-CS-4 dated November 1986, which can serve as a useful reference.

4.3.3 Erosion

Erosion in a hydraulic structure may be due to various reasons, such as, dissolution of soluble materials, transportation of fine grained / loose materials under high hydraulic gradient, abrasion due to silt laden water etc. If it is due to dissolution of soluble materials, transportation of fine grained / loose materials under high hydraulic gradient, it may lead to piping or excessive seepage / leakage endangering the stability of the structure. The rehabilitation measures required will be similar to those to check seepage / leakage due to erosion / piping.

If it is as a result of abrasion due to silt laden water, the rehabilitation measures may include epoxy painting, relining with richer cement / epoxy mortar or even steel lining. Micro silica cement has indicated higher abrasive resistance, and may, therefore, be adopted in sluices, spillways etc., where flow velocity is very high and / or silt laden water is expected.

4.3.4 Excessive Settlement / Deformation

Structure may exhibit excessive settlement / deformation due to various reasons, such as poor foundation, inadequate foundation treatment, improper design / detailing, poor construction practice etc. This has a very damaging effect on electro-mechanical equipments, as their alignment goes into disarray. Though jacking or lifting up the foundation is possible as a rehabilitation measure, it is very difficult to apply in case of civil structures of hydropower plants due to their massiveness. Local modification in civil structures should, therefore, be attempted, if required.

Excessive lateral deformation may be checked by way of anchoring or, providing stiffeners. Thorough design review should be carried out first to establish the cause of excessive deformation and the extent of stiffening required.

4.3.5 Structural failure

Structural failure is the ultimate distress situation. It may be due to poor planning, design, construction and / or maintenance practices. Once a civil structure has failed, not much can be done by way of rehabilitation except its life can be somewhat increased by strengthening, such as propping, jacketing, stiffening etc. The appropriate measure will depend upon the cause and extent of structural failure. This may require a thorough design review. Rebuilding the civil structure may be warranted in certain cases.

4.4 Rehabilitation of structures in distress due to Ageing / Wear & Tear Failure

Following basic strategies are generally adopted while considering Rehabilitation, Modernization and Life Extension of dams;

4.4.1 Dam Safety Inspections – Assessment of present condition of dam structures based on technical and operational performance and condition monitoring. This is like safety review and provides very important input for identification of distress and residual life assessment. All dam components should be inspected by an independent team of experts having specialization in hydropower, hydraulics, geology, concrete, gates, etc., at least twice in a year; one before and another after monsoon. Existing condition / behaviour monitoring and recording instruments (strain gauges, stress gauges, piezometers, seepage / leakage flow meters etc.) should be made use of during the inspection.

"Standardised data book format, sample checklist and proforma for periodical inspection report", CWC Publication, October 1988 may be used to record the findings of the visual inspection. Photographic records should also be maintained for future reference.

4.4.2 Upkeep – Maintaining the dam without any technical improvements, such as painting, local repair of broken parts etc. Such strategy will ultimately lead to high maintenance cost because of increase in repair costs over the years, and also, if this happens too often.

4.4.3 Modernisation - Modernisation means refurbishing, upgrading and uprating ageing components rendering the dam to operate for many more years. Modernisation cost is always higher, but it is justified in the sense that it is much less in comparison to constructing a new dam. The modernisation scheme for a particular component has to match with the modernisation strategy of the dam as a whole.

4.4.4 Augmentation / Replacement – Augmentation of the existing facilities like addition of new component or replacement of existing component. This is capital intensive but will utilize full potential.

4.4.5 Upgrading – Upgrading means increasing storage capacity with modification of existing facilities by increasing dam height. This might result into additional submergence calling for additional environment & rehabilitation measures.

4.4.6 Decommissioning – Removing the dam from operation to avoid future operation & maintenance costs. This strategy is generally not adopted, as it will not be wise to forego the huge investments made in the dam since its conception.

5. MODIFICATION / RE-DEVELOPMENT

At the time of original planning, design & construction, the state of art technology may not have been so advanced, as it is today. Dam safety and rehabilitation of dams presents an opportunity to review the original plan to ensure most optimum utilization of natural resources in the form of water quantum & head available to obtain maximum benefits. Emphasis should be given on following aspects during modification / re-development;

(a) Hydrological Review

Hydrology plays a major role in dams, as the basic input, availability of water depends on hydrology. Due to upstream developments, i.e., construction of barrages, dams (including check dams) affecting water withdrawals, regeneration etc., or even due to climate change, hydrology of the region undergoes changes. Thus, hydrology review with regard to reassessing dependable flows & Probable Maximum Flood should be carried out as a first step towards modification / redevelopment. If the dependable flow has increased due to regulated discharges from the upstream reservoirs, the up-rating may be undertaken. But if the dependable flow has decreased due to more withdrawals in the upstream, up-rating may not come out to be techno-economical.

(b) Water conductor system

Majority of head loss occurs in water conductor system and any improvement in its conveyance efficiency will go a long way in improving the performance of the hydropower plant. There are a number of hydro power plants constructed in the past, where long water conductor systems had to be provided on account of limited construction and geotechnical investigation facilities available at that time. While taking up of RM&LE of such plants, possibility of realigning / modifying the layout of water conductor system to reduce the overall head loss should invariably be studied and techno-economic studies should be carried out.

(c) Silt problem

High silt content in river water poses a very severe siltation problem in dams. These not only reduce the effective storage behind dam, but also cause operation problems of gates etc., besides damaging turbine blades severely. Proper modification in plan is needed to ensure less silt intake into head race canal / tunnel.

Silt problem in a dam can be mitigated to some extent by carrying out catchment area treatment/ watershed development in upstream areas and by adopting a reservoir operation policy to allow first flood wave containing maximum silt content to pass through and to build up storage only in the receding months of monsoon.

6. ADVANCEMENTS IN MATERIALS

Material science is one area where significant developments have taken place and are in progress. Two most important construction materials used in dam construction/rehabilitation are concrete and steel and advancements in these two materials are briefly described below;

6.1 **Concrete**: Dam construction / rehabilitation has following special requirements for concrete;

- low heat (mass concrete)
- high abrasion resistance (spillway)
- high strength (spillway piers, power house)
- high ductility (machine floors, shotcrete)

The newer materials are;

(i) **High strength concrete** - Normal Strength of the concrete is 20-50 MPa. High Strength concrete have compressive strength of 50-100 MPa with concretes having strength 100-150 MPa and > 150 MPa are defined as Ultra High Strength and Specially High Strength concrete. High strengths are made possible by reducing porosity, in-homogeneity, and micro-cracks in the hydrated cement paste and the transition zone.

(ii) **High performance concrete** – It is different from the High Strength Concrete and normally defined as a concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practice. Use of silica fume, fly ash, super-plasticizer, fibre reinforced, admixtures, self compacting, etc., improves the strength and performance of the concrete.

(iii) **Polymer concrete –** also known as resin concrete, is a constructional composite, a variation of concrete, in which traditional binder - cement, has been completely replaced with synthetic resins, such as, Epoxy resins styrene-butadiene and polyacrylate copolymers, etc., with a hardening agent and filler: mixture of sand-and-gravel and quartz powder. **Latex-modified concrete** (LMC) by replacing part of mixing water with a latex and **Polymer-Impregnated Concrete** (PIC) by impregnating or infiltrating a hardened concrete with a monomer and subsequently polymerizing the monomer in situ are most common variants of polymer concrete.

(iv) **Reactive Powder Concrete** - A composite material in which microstructure is optimized by precise gradation of all particles to yield maximum density using highly refined silica fume.

6.2 <u>**Steel**</u>: Next important material, i.e., steel has following special requirements in dam construction / rehabilitation;

- corrosion resistive (RCC, gates)
- high yield strength (spiral casing, penstocks)
- high ductility
- good weldability

The newer materials are;

(i) Abrasion / Wear Resistant Steel (Hardened boron steel)
(ii) High Performance Steel (HPS 50W (345 Mpa) / HPS 70W(485 Mpa) - Quenching and Tempering (Q&T); Thermal-Mechanical Controlled Processing (TMCP)
(iii) Cold formed steel (members, panels, and prefabricated assemblies)

(iv) Carbon steel for high pressure vessels, e.g., penstocks

7. ADVANCEMENTS IN METHODOLOGIES

The most important advancements in dam safety methodologies have been in the field of dam safety monitoring and hydrological review, as briefly described below;

7.1 **Dam Safety Monitoring** – Under the Dam Rehabilitation and Improvement Programme (DRIP), a dam safety monitoring application called **Dam Health and Rehabilitation Monitoring Application (DHARMA)** is being developed. The application has following modules;

(1) Basic Feature' - fundamental data as in NRLD;
(2) 'Project Portfolio' - all lateral/layered project components with photo;
(3) 'Engineering Feature' - salient features, design, hydrology, geology, construction history, operation plan, instrumentation, maintenance schedule, earlier studies, safety related events, known deficiencies, etc.;
(4) 'Stakeholders' - owners, operations, beneficiaries, contractors, emergency action etc.;
(5) 'Dam Health' – periodic inspection report;
(6) 'Dam Rehabilitation' – repairs works, technique involved, cost of work, agencies involved, extent of mitigation, etc.;
(7) 'Analysis and Report' - time-series data of pertinent parameters for the purpose of analysis and report preparation.

7.2 Hydrological Review – Hydrology is both science and arts, and therefore, large variation is observed in design floods when done by different agencies. Therefore, in order to standardize Hydrological Design Practices in the form of design aids for uniform use, all over the country, using state of the Art technology to the extent possible, **Hydrological Design Aids** are being developed under the Hydrology Project. It has got following three components;

- HDA-Y: Assessment of Water Resource Potential – Availability/Yield Assessment
- HDA-F: Estimation of Design Flood
- HDA- S: Sedimentation rate estimation

It is important to note here that one of the major recommendations of the National Water Policy, 2012 is

"Planning and management of water resources structures, such as, dams, flood embankments, tidal embankments, etc., should incorporate coping strategies for possible climate changes. The acceptability criteria in regard to new water resources projects need to be re-worked in view of the likely climate changes."

Therefore, water resources projects should be design for future hydrological scenario, may be by generating flow series considering land-use and climate change, rather than by considering past 30 years or so hydrological data.

8. CONCLUSION

Dams have played a key role in fostering rapid and sustained agricultural and rural growth and development, which have been key priorities for the Government of India since independence. Irrigated agriculture and hydropower development have been major pillars of the government's strategy to achieve these priority goals and to ensure food security. Rainfall, which occurs mainly in intense and unpredictable downpours within a four-month monsoon season, is of high temporal and spatial variability and does not meet year-round irrigation and other water demands. Climate change has aggravated the variability. India ranks third in the world after China and the United States in terms of number of dams. Over the last fifty years, India has invested substantially in infrastructure necessary to store surface runoff in reservoirs formed by large, medium, and small dams with associated appurtenances. It is hoped that with proper rehabilitation using newer materials and methodologies, existing dams are conserved and benefits continue to be obtained for a longer period.

Chapter 14

Disaster Management Planning in case of Dam Break Events

"In today's society while hazards, both natural or otherwise, are inevitable, the disasters that follow need not be so and the society can be prepared to cope with them effectively whenever they occur. The need of the hour is to chalk out a multi-pronged strategy for total risk management, comprising prevention, preparedness, response and recovery on the one hand, and initiate development efforts aimed towards risk reduction and mitigation, on the other. Only then can we look forward to "sustainable development."

- *Disaster Management - The Development Perspective,*
Ministry of Home Affairs, Government of India

1. INTRODUCTION

The eventuality of a dam break event and consequent grave disaster brings us to a state of despair. Luckily, Incident Command System (ICS) has proved to be an effective management tool towards disaster risk reduction giving us a ray of hope. But this is often based and focused on the proven fireground tactics applied by most, if not all, fire fighting agencies and industrial fire brigades. ICS needs to be supplemented with strategic planning for required levels of preparedness for entire range of disaster scenarios that may save lives and reduce property damage in areas affected by dam operation or failure. This is provided to some extent by the Emergency Action Planning. Therefore, there is a need to evolve a Disaster Management Plan (DMP) with reference to dam break scenario around the concept of Incident Command System and Emergency Action Planning integrating therewith the disaster management practices being followed in India. Hereinafter, the suggested approach towards disaster management planning in the event of a dam break is being presented.

2. SCOPE OF DISASTER MANAGEMENT PLAN

As we all know, DMP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. The DMP specifies actions the dam owner should take to moderate or alleviate the problems at the dam site as well as in the areas downstream of the dam. The DMP will guide the dam operation supervisory personnel in identifying, monitoring, responding to and mitigating emergency situations. It contains procedures and information to assist the dam owner in issuing early warning and notification messages to responsible emergency management authorities, viz., District Magistrate / Collector, Armed forces, Paramilitary forces, Project Authorities and other Central/ State Agencies. It also contains inundation maps to show the emergency management authorities of the critical areas for necessary relief and rescue actions in case of a dam break disaster. It outlines *"who does what, where, when and how"* in an emergency situation or unusual occurrence affecting the dams.

A DMP is needed for following two main reasons:

- ❖ To pre-plan the coordination of necessary actions by the dam owner and the responsible Local & State officials to provide for timely detection, warning, and evacuation in the event of an emergency.
- ❖ To reduce the risk of loss of life and property damage, particularly in downstream areas, resulting from an emergency situation.

3. STEPS IN DEVELOPING A DMP

Careful and coordinated planning with all involved parties will lay the foundation for a thorough and practical disaster management plan. The process of developing an DMP generally follows the nine steps listed below:

3.1 HAZARD ANALYSIS

Hazard analysis is identification, studies and monitoring of any hazard to determine its potential, origin, characteristics and behaviour. It is necessary to determine and identify potential hazards, i.e., the situation(s) or triggering event(s) that initiate or require an emergency action and/or disaster management. Major elements of hazard analysis are:

(i) **A listing of the conditions or events which could lead to or indicate an existing or potential emergency** - The prominent causes are Extreme storm, Landslide, Earthquake, Overtopping, Structural damage, Piping, Equipment malfunction, Foundation failure, Sabotage, etc. The above unusual situations can be divided into three main categories:

(a) **Hydrologic**: These are related to flooding due large releases, seepage, slumping, piping, embankment cracking, embankment deformation, embankment overtopping, movement of concrete section (sliding or over turning) settlement, failure of spillway gates or supporting structures, spillway & outlet works releases, equipment malfunction, etc.

(b) **Earthquake**: These are related to impact of earthquake at dam which could lead to embankment piping, embankment cracking, embankment deformation, liquefaction and movement of concrete section, etc.

(c) **All other events**: These are related to hazardous material spills / releases, equipment failures, security / criminal actions, fish / wildlife impacts, wildfires, structural fires, landslides, extreme storm, sabotage, etc. One or more of the above unusual situations will initiate declaration of an emergency for internal alert or external alert.

(ii) **Determination of threshold value for event creating hazard situation –**

Next step would be to determine the threshold value above which the occurrence of event may lead to hazardous situation. For example, seismic events (earthquakes) lead to hazards but events with earthquake intensity less than 4.0 Richter scale are of no consequence. Hence, there is no need to press the panic button and initiate disaster management activities, if earthquake events with intensity less than 4.0 Richter scale occur.

(iii) A brief description of the means by which hazard could be identified –

The means could include the data & information collection system, monitoring arrangements, surveillance, inspection procedures and other provisions for early detection of conditions indicating an existing or potential emergency. This would make aware the people about impending hazards.

3.2 VULNERABILITY ANALYSIS

Vulnerabilities can be manifested as physical (human, agricultural, structural, etc.) and/or socio-economic vulnerability. In case of dam breaks, the vulnerability analysis would include assessment of

(a) condition of the dam and the degree, if any, of dam safety deficiency,
(b) population at risk and community vulnerability,
(c) scale of flood risk costs,
(d) range of other consequences (eg. on property, the environment, or community value of the dam),
(e) stakeholder perceptions and expectations, and
(f) state of knowledge and planning commitments for different scenarios.

These call for dam safety inspection, dam break flood forecasting and survey of downstream area (along with demographic and socio-economic details) likely to be affected in the event of a dam break.

3.3 RISK ANALYSIS

Estimates of probability are required in dam safety risk analysis for estimating both the probability of failure and the consequences of failure (e.g. life and/or property loss). Emergency events occur with unusual situations varying degrees of severity and predictability. An emergency may develop gradually and be steadily monitored providing ample response time. Conversely, an emergency may develop suddenly requiring immediate emergency response to prevent devastating loss of life or impacts to structures or the environment. Emergency events may be classified as the Blue, Yellow, Orange and Red level alert according to an ascending and progressive order of severity to which the dam downstream or off-site population, structures or environment are threatened. Condition may dictate that situation is classified as imminent without passing through the less severe situations. A smooth transition should occur, if the situation is classified as “developing” prior to “imminent.”

Following need to be addressed as part of risk analysis:

<u>Definition of alert levels</u>: If an incident happen with the dam or problem arises related with the dam over its foundations or with landslides into the reservoir, earthquakes, adverse meteorological situations or any other, alert must be given to civil protection system. The gravity of the problem may obviously vary with the situation, but in order to assure that all the entities involved in emergency actions are behaving in accordance with the situation, the alert levels need to be previously defined, and must be the same for the internal and external emergency plan of each dam.

These alert levels are established in colours, from Blue, the lowest level that corresponds to a routine or normal situation, to Red, corresponding to a serious or catastrophic situation.

Alert and warning systems: When facing problematic situation, the entity exploring the dam must judge the corresponding alert level and notify the civil protection system, in order to allow the development of the necessary actions, according to the situation.
Procedures, aids, instructions and provisions for interpreting information and data to assess the severity and magnitude of any existing or potential emergency should be clearly defined.

The alert levels, warning system and response system may be defined as presented in Table 1.

Table 1. ALERT LEVELS FOR DAM BREAK PLANNING

Type of alert	Alert level	Situation	Response system	Engineer in charge Actions
INTERNAL ALERT	Blue	No immediate off-site impact anticipated or detection of anomalies in the dam or other events that do not compromise the structural dam safety nor its operational elements, and do not make unviable the dam observation system. Situation is stable or developing very slowly. The gravity of existing problems must let belief that no consequences are expected in the valley downstream of the dam.	Direct Command System	1. Measures to solve problem. 2. Give internal alert signal of blue level. 3. Make notification: Inform to: - Dam Owner (CE)
	Yellow	Existence of anomalies or events that might comprise up to some degree; The structure and / or operational dam safety or the dam observation system, assuming that eventual small consequences downstream the dam can happen: Existence of meteorological adverse conditions; Detection of anomalies in dam structural / operational elements and/or observation system; Existence of foundation problems; Situation is developing slowly.	Direct Command System	1. Measures to solve problem 2. Give internal alert signal of yellow level 3. Make notifications. Inform to - Dam owner (CE) - District Collector (if necessary)
EXTERNAL ALERT	Orange	Situation with high probability of dam failure, belief that it might not be possible to control the situation, and might cause serious consequences downstream of the dam: Detection of anomalies in dam structural / operational elements and/or observation system. 1. Existence of severe foundation problems 2. Occurrence of floods with high recurrence interval. 3. Dam owner / operator need assistance from outside agencies or jurisdiction. 4. Situation is progressing rapidly. 5. "Some amount of time" is available for analysis, decisions and mitigation to be made before off-site impact may occur.	Incident Command System	1. Measure to solve problem. 2. Give external alert signal of orange level. 3. Implement Incident Command System. 4. Make notification Inform to - District Collector - CE/E-in-C - State Flood Control Cell Warning – Population downstream of the dam to be ready for evacuation.
	Red	Situation of inevitable catastrophe Imminence of dam failure Dam failure "Little or no time is available" for analysis, decisions & mitigation to be made before downstream of dam impacts occur. Situation is worsened and a breach is apprehended. "Little or no time is available" for analysis, decisions and mitigation to be made before off-site impact occur.	Incident Command System	1. Give external alert signal of red level. 2. Make notification. Inform to: - Dam owner (CE/E-in-C) - Civil protection (District Collector) Commissioner - State Flood Control Cell Warning: Population downstream of the dam to evacuate quickly.
ACTION		Dam is failing or failed	Incident Command System	1. Call and coordinate with 2. Civil Protection (Collector & SP) & CE 3. Inform Commissioner and E-in-C 4. Ensure official notifications are made.

3.4 DEFINING RESPONSIBILITIES

The next step, after carrying out hazard, vulnerability and risk analysis, is to define the responsibilities for carrying emergency actions and disaster management to avoid any confusion in a crisis situation. For example;

Responsibility for notification

Generally, the dam owner is responsible for notifying the appropriate officials when flooding is anticipated, or a failure is imminent or has occurred. The India Meteorological Department (IMD), Central Water Commission and / or other State and Central agencies have the general responsibility for issuing flood warnings. It will, therefore, be desirable to notify the IMD, CWC or other appropriate agency of any pending or actual dam break flooding, so that its facilities can enhance warnings being issued.

Responsibility for evacuation, rescue & relief

In the federal set up of India, the basic responsibility for undertaking rescue, relief and rehabilitation measures in the event of a disaster is that of the State Government concerned. The district level is the key level for disaster management and relief activities. The Collector / District Magistrate is the chief administrator in the district. He is the focal point in the preparation of district plans and in directing, supervising and monitoring calamities for relief.

DMP Coordinator's responsibility

If appropriate, designate a DMP coordinator, who will be responsible for DMP–related activities, including (but not limited to) preparing revisions to the DMP, establishing training seminars, coordinating DMP drills, etc.

3.5 ESTABLISHING NOTIFICATION PROCEDURES

Notification procedures should be established to ensure timely notification of persons responsible for taking emergency actions. The procedures should be brief, simple, and easy to implement considering the following information for each of the emergency conditions considered:

a) Who is responsible for notifying each owner representative(s) and / or public official (s) ?
b) Who is to be notified?
c) Prioritized order in which individuals are to be notified?

For each type of emergency situation, the DMP should clearly indicate who is to make a call, to whom it is to be made and in what priority. This should include individual names and position titles, office & home telephone numbers and alternative contacts & means of communication (e.g. mobile phones).

It would be better to evolve a Notification Flowchart, which is a schematic representation of the hierarchy for notification in an emergency situation, including who is to be notified, by whom and in what priority. The plan should designate a spokesperson to disseminate information. The news media, including radio, television and newspapers, should be utilized to the extent available and appropriate.

3.6 DEFINING PREVENTIVE / EMERGENCY ACTIONS

The essence of Disaster Management lies in prevention of hazard and prompt action at site. The DMP should describe preventive actions to be both prior to and following the development of emergency conditions, to prepare for any emergency. Preventive actions involve the installation of equipment or the establishment of procedures for one or more of the following purposes:

a) Preventing emergency conditions from developing, if possible, or warning of the development of emergency situations.
b) Facilitating the operation of the dam in an emergency situation.
c) Minimizing the extent of damage resulting from any emergency situations that do develop.

There are several types of preventive actions that should be considered when developing an DMP. These actions include:

<u>Surveillance</u>:

Surveillance is key to prevention and early warning. At important dams, the dam owner should also consider installing a remote surveillance system that includes instrumentation and telemetering facilities at the dam site, to provide a continuous reading of headwater and tail water levels at a central operations control centre that is manned 24 hours a day.

The DMP should also describe procedures for providing round-the-clock surveillance for periods of actual or forecasted high flows. It may be necessary to post a special observer to the dam during these periods and not rely on the instrumentation alone. In addition, it is recommended that an expert observer be at the dam when flood conditions or signs of serious structural distress have been identified.

<u>Access to the site:</u>

The description of access should focus on primary and secondary routes and means for reaching the site under various conditions (e.g., foot, boat, motor vehicle, helicopter, etc.). Also detail the expected response (travel) time.

<u>Response during periods of power failure:</u>

The DMP should also include the response to potential or actual emergency conditions during periods of power failure, such as actions to illuminate access routes, power control rooms, etc.

Response during periods of adverse weather

The DMP should also include discussion of emergency response under adverse weather conditions including the expected response time.

Alternative systems of communication

The DMP should also include description of the availability and use of alternative communication systems at the site.

Emergency supplies and resources

There are certain planning and organizational measures that can help dam personnel and local officials manage emergency situations more safely and effectively. These measures include stockpiling materials and equipments. Where applicable, document should describe:

a) Materials needed for emergency repair, their location, source and intended use. Materials should be as close as possible to the dam site.
b) Equipment to be used, its location and who will operate it.
c) How the contractor is to be contacted.
d) Any other people who may be needed (e.g., labourers, engineers), and how they are to be contacted.

Coordinating information on flows

Where applicable, the DMP should describe coordination of information on flows based on weather and runoff forecasts and action required in case of any failure or other emergency condition. The IMD or other appropriate agency like CWC, etc., may also be able to supplement the warnings being issued by using their own communication system.

Providing alternative sources of power

Where applicable, the DMP should describe the alternative sources of power for spillway gate operation and other emergency uses.

Finally, DMP should also include any other site-specific actions devised to mitigate the extent of possible emergencies.

3.7 DEFINING EXTENT OF DISASTER ZONE

Extent of disaster zone or the area of impact in the event of disaster scenario is very important input for disaster management. The dam break events cause inundation, defined through inundation maps, which are outline of the area covered by the dam break or excessive release flood in enough detail to identify dwellings and other significant features that are likely to be directly affected. Following are the types of inundation maps important for disaster management planning;

Index maps

A list of town & villages, important public buildings & installations, railway lines, railway stations, Post & Telegraph offices and roads which may come under the flood line, prohibitive zone, restrictive zone and caution zone are to be marked on index map, generally prepared to the scale of 1:50000.

Detailed maps

In respect of cities or towns and the villages falling in the likely inundation area of dam break floods, detailed contour maps of the entire area showing contours at 0.5 m intervals should be prepared.

Maximum Inundation Maps

The Maximum Inundation Maps show the area which would be flooded in the event of a failure of the dam with the reservoir at pre-specified breach elevation and depths with different coloured area.

Leading Edge Times Maps

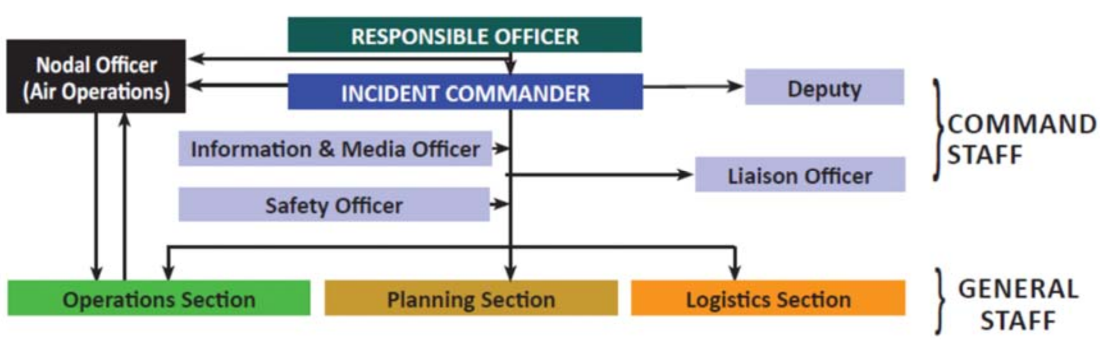
Leading edge times maps show the travel time of flood water in the event of failure of dam and maximum spillway release in hours with different colours.

Other Maps

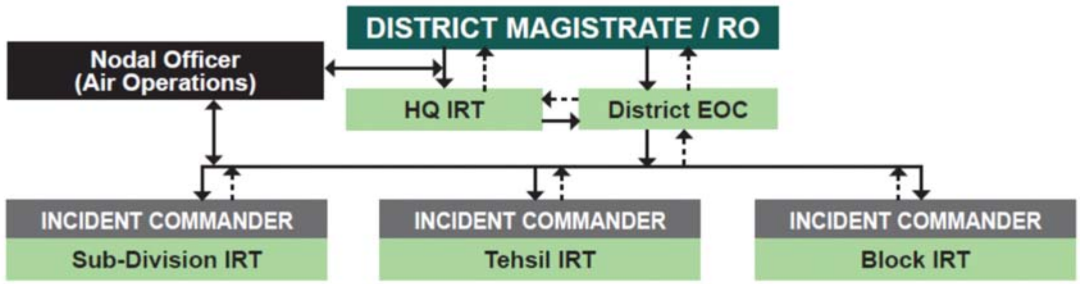
Layout plan showing head works, plan showing position of rain gauge stations in catchment area and plans showing position of wireless or other communication means and other relevant maps should also be given.

3.8 DEFINING INCIDENT RESPONSE SYSTEM

This is the most important aspect of Disaster Management Plan because success of any disaster management lies in effective command structure. This should be planned in the lines of Guidelines for Incident Response System of the National Disaster Management Authority, which proposes following structure;



The Guidelines for Incident Response System (NDMA, 2010) suggests following structure of Incident Response Organisation at District level;



3.9 RECONSTRUCTION AND RECOVERY

The Disaster Management Plan should also include plan for initiating reconstruction and recovery in post disaster phase. Such a plan may include

a) establishment of a Recovery Committee,
b) survey and assessment of damage to life and properties due to disaster,
c) establishment of welfare centres;
d) provision for welfare support including personal needs, advice and counselling.
e) provision for temporary emergency housing and assistance to return home;
f) emergency financial assistance;
g) integration of normal developmental plans with recovery and reconstruction activities,
h) restoration of public facilities and services, etc.

If resources at this level are found deficient, the State/National Recovery Emergency Management Plan may be activated for additional support.

3.10 ADDITIONAL INFORMATION

Following the main body of the plan, additional information in the form of several appendices, for a more clear division of information should be included, which contains basic information about the dam, data used in the development of the DMP and instructions for the maintenance of the plan.

Listed below are some of the specific topics to be covered in the appendices accompanying the DMP:

<u>Description and location of the dam</u>

First appendix should summarize the principal features of the dam and give a listing of the Salient Features, besides providing key drawings and/or photographs of the dam and appurtenances. Such an appendix could also describe the upstream and downstream areas and topography and establish the location of the dam, using maps and narrative description.

Investigation and analysis of dam break floods

This appendix can identify and briefly describe the method and assumptions selected to identify the inundated areas and a brief description of the methodology and outputs of the dam break modelling performed.

Training Plan

Training of people involved in the DMP should be conducted to ensure that they are thoroughly familiar with all elements of the plan, availability of equipment and their responsibilities & duties under the plan. Exercises simulating dam failures are excellent training mechanisms for ensuring readiness. Cross-training in more than one responsible position for each individual is advisable in order to provide alternates. A careful record by roaster should be kept of training completed and refresher training conducted.

3.11 TESTING OF DMP

The DMP should prepare scenarios for the various emergency conditions and a plan to test the state of training and readiness of key personnel responsible for actions during an emergency, to make sure that they know and understand the procedures to be followed and actions required. Any special procedures required for nighttime, weekends and holidays should also be included. The tests should involve a drill simulating emergency conditions, preferably up to but not including actual evacuation.

At least once every five years, a drill should be conducted that is coordinated with all State officials having responsibilities under the plan. The involvement of State emergency services can be encouraging and can foster enthusiasm for the maintenance & timely implementation of the DMP. Immediately following a test or actual emergency, a review should be conducted with all parties involved. After the review has been completed, the plan should be revised, if necessary, and the revisions disseminated to all parties involved.

3.12 REGULAR UPDATING OF DMP

The DMP should be updated promptly after each change of personnel involved or their telephone numbers. The exercise should be conducted together with local government officials, for a comprehensive review of the adequacy of the DMP at intervals not to exceed one year. Copies of any revision that do result from updation of the plan or from periodic testing of the plan, should be furnished to all individuals to whom the original plan was distributed. A procedure should be established to ensure that all copies of the plan are revised and updated regularly.

4. CONCLUSION

The National Water Policy (2012) inter-alia proposes that to increase preparedness for sudden and unexpected flood related disasters, dam/embankment break studies, as also preparation and periodic updating of emergency action plans / disaster management plans should be evolved after involving affected communities.

Disaster management as an administrative measure has been developed to a satisfactory level in India, but the technical back up is still lagging. For example, dam break modeling is a must to provide necessary technical input in evolving effective disaster management in the downstream areas. But, dam break modeling exercise is seldom carried out, and even in cases, where it carried out, it is seldom integrated with local disaster management practices. There is a need to carry out dam break analysis and integrate that in Disaster Management Plan of the District/State so that people be assured of timely warning and assistance in the unlikely event of dam breaks. However, iIt must be recognized that each dam break scenario is unique in itself due to uniqueness of its topography, geology, hydrology, etc. and most important breaking mechanism. And accordingly, each Disaster Management Plan will also require to be unique.

DAM SAFETY BILL, 2019

WELCOME

1

BACKGROUND

- In 1982, GOI constituted a Standing Committee (under Chairman, CWC) to review existing practices & evolve unified procedures of dam safety. Committee submitted its report in July 1986.
- One of the recommendations of Committee was the 'enactment of Dam Safety Legislation'.
- A comprehensive Draft Dam Safety Bill prepared in 2002, and circulated to State for comments.
- States responded well to the Draft Bill. Govt. of Bihar passed the Act in May 2006.
- Govt. of AP adopted a resolution (Mar.2007) for regulation of Act by Parliament.
- Govt. of WB passed similar resolution (July 2007) empowering Parliament to pass Act.
- Parliament empowered to legislate on any subject provided two or more states consent (Art. 252).

2

BACKGROUND – CONTD.

- Dam Safety Bill 2010 finalized with MoLJ, and placed before the Parliament on 30th August 2010.
- Bill subsequently referred to the Parliamentary Standing Committee on Water Resources. Committee's recommendations vide its Seventh Report given in the Parliament in August 2011.
- Owing to significant changes entailed while complying with recommendations of Standing Committee, MoWR decided to withdraw the Bill and introduce a new Bill in the Parliament.
- By the time, the term of the 15th Lok Sabha came to an end, and therefore the Dam Safety Bill 2010 lapsed with the dissolution of 15th Lok Sabha.
- Fresh process for enactment of Dam Safety Bill under Article 252 of the Constitution of India could not be taken up in absence of a response from any of the bifurcated states of Andhra Pradesh and Telangana.

3

BACKGROUND – CONTD.

- MoWR,RD&GR approached the MoLJ for exploring the possibility of enactment of the Dam Safety Bill through Entry-56 or Entry-97 of the List-I of the Constitution of India for pan India coverage.
- MoLJ sought the opinion of the Attorney General, GoI on the proposal for enactment of Dam Safety Bill.
- Matter was further referred by Attorney General to Solicitor General for opinion.
- The learned Solicitor General of India in his considered opinion given on 15.02.2016 stated:

" it will be constitutionally and legally valid to pass the Dam Safety Bill by exercising power conferred under Article 246 read with Entry 56 and Entry 97 of List I of Seventh Schedule of the Constitution".

4

BACKGROUND – CONTD.

- Accordingly, (Draft) Dam Safety Bill, 2016 was prepared in consultation with MoLJ for coverage across whole of India incorporating the recommendations of the Parliamentary Standing Committee on the Dam Safety Bill 2010.
- Also Cabinet Note was updated based the suggestions of the Standing Committee and circulated to concerned 10 Ministries/Department.
- Observations/views of Six Ministries on the draft Cabinet Note on Dam Safety Bill, 2016 (circulated on 18.05.2016) were forwarded to CWC for comments. CWC's observations on the same were submitted.
- The Union Cabinet in June 2018, approved the Draft Dam Safety Bill 2018.
- The State of Tamil Nadu and Kerala have certain reservations on the draft Bill. The provision of the Bill addresses the objections raised by these States.
- The Draft Dam Safety Bill was introduced in Lok Sabha by Minister of Water Resources, River Development and Ganga Rejuvenation on December 12, 2018.
- The term of the 16th Lok Sabha came to an end, and therefore the Dam Safety Bill 2018 lapsed with the dissolution of 16th Lok Sabha.

5

THE DAM SAFETY BILL, 2018

Arrangement of Clauses

- Chapter I: Preliminary
- Chapter II: National Committee on Dam Safety
- Chapter III: National Dam Safety Authority
- Chapter IV: State Committee on Dam Safety
- Chapter V: State Dam Safety Organisation
- Chapter VI: Duties & Functions in relation to DS
- Chapter VII: Safety Inspection & Data Collection
- Chapter VIII: Emerg. Action Plan & Disaster Mgmt.
- Chapter IX: Comprehensive DS Evaluation
- Chapter X: Offences and Penalties
- Chapter XI: Miscellaneous

- 3 Schedules on functions of: NCDS,NDSA & SCDS

6

PRELIMINARY

Preamble

to provide for proper surveillance, inspection, operation and maintenance of specified dams and to provide for institutional mechanism to ensure their safe functioning and for matters connected therewith or incidental thereto.

- Applicability
  - The Act will apply to the whole of India.
- Definitions

7

NATIONAL COMMITTEE ON DAM SAFETY

- Composition of NCDS:
  - Chairman, CWC – Chairperson, ex officio;
  - Central Govt. representatives (not exceeding 10);
  - Engineer-in-Chief or equivalent representatives of States (not exceeding 7, by rotation);
  - Specialists in the field of dam safety and allied fields (not exceeding 3);
- Reconstitution after every three years.
- To perform functions necessary to evolve policy, guidelines and standards, for proper surveillance, inspection, operation and maintenance of dams.
- Function set out in First Schedule of the Bill.
- Provision for sub-committees as necessary.
- Secretarial assistance by NDSA.

8

NATIONAL DAM SAFETY AUTHORITY

- NDSA headed by an officer not below the rank of Additional Secretary to the GoI(by appointment).
- To perform functions necessary to implement the policy, guidelines and standards evolved by NCDS.
- Function set out in Second Schedule of the Bill.
- NDSA to also look into unresolved points of issue between SDSOs of two or more states, or between SDSO of a State and dam owner. Decisions of NDSA to be Final and Binding.
- Headquarters of the Authority at Delhi.
- Functions, powers and terms and conditions of the service prescribed by the Central Govt.

9

STATE COMMITTEE ON DAM SAFETY

- Compositions of State Committee:
  - Engineer-in-Chief/ equiv. officer – Chairperson;
  - Technical/ scientific officials of CE rank (Max 6);
  - CE level representative of u/s States in case reservoir area of any dam extends to that State;
  - CE level representative of d/s State in case flood release of any dam flows to that State;
  - Director Level representative of CWC;
  - Experts from engineering institutes (Max 2);
  - Specialists in dam safety and allied fields (Max 2, not belonging to the State or bordering States).
- Reconstitution after every three years.
- Function set out in Third Schedule of the Bill.
- Secretarial assistance by SDSO.

10

STATE DAM SAFETY ORGANIZATION

- SDSO to be established in WRD or ID or PWD.
- In case of States having more than 20 specified dams, SDSO to be headed by officer not below CE rank, in other cases by officer not below SE rank.
- Organisational structure and work procedures of SDSO to be prescribed by the State Government.
- Functions, powers and terms and conditions of the service prescribed by the Central Govt
- Suitable number of officers to be provided by the State Government. Such officers shall have sufficient experience in the field of dam safety preferably from areas of dam-design, hydo-mechanical engineering, hydrology, geo-technical investigation, instrumentation, or dam-rehabilitation.

11

DUTIES & FUNCTIONS IN RELATION TO DAM SAFETY

- For all dams under jurisdiction, and with a view of achieving satisfactory level of dam safety assurance as per guidelines, standards and other directions issued by the NCDS, every SDSO shall:
  - Keep perpetual surveillance;
  - Carry out inspections; and
  - Monitor the operation and maintenance.
- SDSO shall make such investigations & gather such data as may be required.
- SDSO to classify each dam as per vulnerability and hazard classification criteria laid down by NCDS.
- Maintain a Log Book/ Data-base recording activities of surveillance/ inspection and important events.

12

DUTIES & FUNCTIONS IN RELATION TO DAM SAFETY – CONTD.

- SDSO to maintain records of major dam incidents.
- SDSO to render its advice to the concerned dam owner on the safety or remedial measures.
- Dam owner to earmark sufficient funds for maintenance & repairs, and to implement the recommendations of SDSO.
- Dam owner to compile all technical documentations related to dam safety, along with information on resources/ facilities to be affected by dam failure.
- Dam owner to have state-of-the-art data mgmt. tools.
- Individuals responsible for dam safety to possess qualifications & experience specified by NCDS.
- Individuals to undergo adequate trainings.

13

DUTIES & FUNCTIONS IN RELATION TO DAM SAFETY – CONTD.

- All specified dams to fall under jurisdiction of the SDSO of the State in which dam is situated; and full co-operation to be extended by owner of dam.
- Authorised representative of NDSA/SDSO may enter any part of dam/ site, and apply such investigation methods as considered necessary.
- Representative of NDSA/ SDSO to report any remedial measures to officer-in-charge of dam.
- In case of dams found to be endangered, NDSA/ SDSO may also suggest such remedial measures on such operational parameters as necessary.
- Costs on investigations to be paid by dam owner.

14

DUTIES & FUNCTIONS IN RELATION TO DAM SAFETY – CONTD.

- Construction or alterations of dams:
  - Investigation, design & construction by recognised organizations.
  - Use of the relevant standard codes and guidelines of the BIS.
  - Qualified, experienced and competent engineers, as specified by NCDS, for the purpose of investigation, design & construction.
  - Demonstration of safety of design, operational parameters and policies to NDSA/ SDSO for approval.
  - Undertaking of quality control measures, as specified by NCDS.
  - Construction of new dam or alteration/ enlargement of existing dam only with the approval of the Competent Authority.
- Before initial filling of any reservoir, the filling criteria & initial filling plan, to be prepared.
- Safety inspection by SDSO Before initial filling.
- Establishment of O&M setup with adequate staff.
- Ensuring a well documented O&M Manual.

15

SAFETY INSPECTION & DATA COLLECTION

- For each dam, owner to establish within his O&M setup a 'dam safety unit'.
- Dam owner to undertake, through dam safety unit, pre-monsoon & post-monsoon inspections of dam.
- Special inspections during & after floods, after earthquake, on sign of distress/ unusual behaviour.
- Engineers, as agreed by SDSO, to be stationed at dam site throughout monsoon period, and during period of emergency following earth-quake/ hazard.
- For each dam, owner to have a minimum number of dam instrumentations installed; maintain record of readings; and forward analysis to SDSO.
- Hydro-meteorological station at each dam site.
- Seismological station for dams higher than 30m or falling in zone III or above.

16

EMERGENCY ACTION PLAN & DISASTER MGMT.

- Every owner in respect of each dam to —
  - Establish hydro-meteorological network/inflow forecasting system;
  - Establish an emergency flood warning system;
  - Test periodically for the aptness of above systems;
  - Make available information on anticipated inflows, outflows, flood warnings & adverse impacts to authorities and public domain;
  - Render cooperation to NDSA in running of early warning system.
- Every owner in respect of each dam to —
  - Carry out risk assessment study at interval as specified by NCDS. First such study within five years;
  - Prepare emergency action plan within 5 years; and for new dams, before the initial filling.
- Emergency Action Plan to include —
  - Type of emergencies likely to occur;
  - Likely flood in the event of dam failure, along with probable areas, population, structures and installations likely to be affected;
  - Warning procedures, inundation maps and advance preparations for handling adverse situations to avoid loss of human life.
- Consultation/ cooperation with DM agencies.

17

COMPREHENSIVE DAM SAFETY EVALUATION

- Comprehensive safety evaluation by independent panel of expert. First CSE within 5 years, and thereafter at regular intervals specified by NCDS.
- CSE shall consists of, but not be limited to,—
  - Review and analysis of data;
  - General assessment of hydrologic and hydraulic conditions with mandatory review of design floods in certain cases;
  - General assessment of seismic safety of dam with mandatory site specific seismic parameters study in certain cases;
  - Evaluation of operation, maintenance and inspection procedures;
  - Evaluation of any other hazardous condition .
- CSE compulsory in case of,—
  - Major modification to structure or design criteria;
  - Discovery of unusual condition at dam or reservoir rim;
  - An extreme hydrological or seismic event.

18

COMPREHENSIVE SAFETY EVALUATION – CONTD.

- CSE report shall include, —
  - Assessment of the condition of structure;
  - Recommendations for any emergency measures or actions, to assure immediate safety;
  - Recommendations for remedial measures;
  - Recommendations for additional detailed studies;
  - Recommendations for improvements in routine maintenance and inspection of dam, if required.
- SDSO to pursue with concerned dam owners to ensure carrying out of the remedial measures.
- Points of dispute, if any, between owner and independent panel to be referred to SDSO; and in case of further disagreement to be referred to NDSA for guidance and recommendations to the concerned State Govt.

19

OFFENCES AND PENALTIES

- Punishable with imprisonment for 1 year, or/ and fine (2 years for loss of lives) if a person obstructs any officer/employee or refuses to comply with any direction of the Central/ State Govt or NCDS/ NDSA/ SCDS/ SDSO.
- If offence by any department of the Govt, the head of the department deemed to be guilty.
- If offence by a company/body corporate, every person in charge of/responsible for conduct of business of company, deemed to be guilty.
- No cognizance of offence except on a complaint by Central/ State Govt or NCDS/NDSA/SCDS/SDSO.
- Jurisdiction: Courts not inferior to Metropolitan Magistrate or Judicial Magistrate of the first class.

20

MISCELLANEOUS

- SDSO to prepare annual report within 3 months of preceding FY for placement in State Legislature.
- NDSA to prepare a consolidated annual report within 6 months for placement before parliament.
- NDSA / SDSO to forward annual reports to NDMA/ SDMA and also make them available in public domain.
- Safety measures by owners in respect of dams other than specified dams as per State Govt. guidelines.
- Measures required by NDSA in respect of dam located outside the territory of India. Central govt. to endeavour to enter into treaty, agreement or convention, and lay report in the Parliament.
- Power to Central Govt. to amend Schedules/ give direction.
- Power of Central/ State Govts. to make rules.
- Power to NCDS/NDSA for making/ enforcing regulations.
- Rules/regulations to be placed in Parliament/Legislature.
- Power to Central Govt to bring amendments(within 2 years).

Thanks

21

SUPPORTING SLIDES

22

Schedule-I: Functions of NCDS

1	Review/ update norms of satisfactory level of dam safety assurance.
2	Act as a forum for exchange of views.
3	Analyse causes of major incidents/ failures suggest changes.
4	Evolve comprehensive dam safety management approach.
5	Render advice on matter referred by Central/ State Government.
6	Make recommendations on safety of dams in other countries.
7	Make recommendations on ageing dams of the country.
8	Provide strategic supervision for dam rehabilitation programs executed in states through central or externally aided funding.
9	Identify areas of R&D and recommend provision of funds.
10	Make recommendations on coordinated reservoir operation.
11	Make regulations on recommendations of NDSA, or otherwise.

23

Schedule-II: Functions of NDSA

1	Monitor functioning of State Dam Safety Organisations.
2	Provide state-of-art technical/ managerial assistance to SDSOs.
3	Maintain data-base of all specified dams in the country.
4	Maintain liaison with all concerned.
5	Lay down guidelines/ check-lists for inspection/ investigation.
6	Maintain records of major dam failures in the country.
7	Examine cause of major dam failure, and report to NCDS.
8	Examine cause of any major public safety concern.
9	Lay down uniform criteria for vulnerability/ hazard classification.
10	Give directions regarding maintenance of Log Books.
11	Give directions on qualifications/ experience requirements of individuals responsible for safety of dams.
12	Accord recognition/ accreditations to organisations involved in investigation, design or construction of new dams.
13	Give directions on qualification/ experience requirements of individuals for investigation, design and construction of dams.

24

Schedule-II: Functions of NDSA – Contd.

14	Give directions regarding quality control measures.
15	Give directions on competent levels of engineers in DS units.
16	Give directions on instrumentation requirements & installation.
17	Directions on data requirements of hyd-meteorological stations.
18	Directions on data requirements of seismological stations.
19	Directions on time interval for the risk assessment studies.
20	Directions on time interval for updating emergency action plans.
21	Directions on constitution of independent panel of experts for CSE
22	Directions on time interval for comprehensive safety evaluation.
23	Lay down guidelines for review of design floods.
24	Guidelines for review of site specific seismic parameter studies.
25	Establish an Early Warning System.
26	Promote general education & awareness on dam safety.
27	Provide secretarial assistance to the National Committee and its sub-committees.
28	Provide coordination and overall supervision of dam rehabilitation programs executed through Central or externally aided funding.
29	Any other specific matter relating to safety of dams referred by th

25

Schedule-III: Functions of SCDS

1	Review work done by State Dam Safety Organisation.
2	Establish priorities for investigations.
3	Order further investigations & use of non-dep. Resources.
4	Recommend appropriate measures for distress condition.
5	Establish priorities for projects of remedial works.
6	Review progress on measures recommended.
7	Assess implication/ coordinate mitigation with u/s State.
8	Assess implication/ coordinate mitigation with d/s State.
9	Assess probability of cascading dam failures, and coordinate mitigation measures with all concerned.
10	Recommend provision of funds.
11	Provide strategic supervision for such dam rehabilitation programs that are executed through State funding.
12	Any other specific matter referred by the State Government.

26

International Comparison

Particulars	USA	Australia	Canada	Norway	India
Legal Arrangement	State-wise	State-wise	State-wise	Central	Centr./ State
Institutional arrangement	Yes	Yes	Yes	Yes	Yes
Approval for Constr. / Alteration	Yes	Yes	Yes	Yes	Yes
Initial filling Plan	Yes	Yes	Yes	Yes	Yes
Periodical Inspection	As per haz.class.		Quarter/ annual	Annual	Pre & post monsoon
Hydro meteorological/Seism. St.	Yes	Yes	Yes	Yes	Yes
Instrumentations	Yes	Yes	Yes	Yes	Yes
Comprehensive DS Evaluation	5 yrs		5 yrs	5 yrs	by NCDS
Emergency Action Plan	Yes	Yes	Yes	Yes	Yes
Risk Assessment	In few states	Only in NS. Wales	Only in Quebec	Yes	Yes
Hazard Classification	Yes	Yes	Yes	Yes	Yes
Penal Provision	Yes	Yes	Yes	No	Yes

27

DHARMA : An Integrated Approach for Effective Management of Large Dam Assets

Central Project Management Unit

Central Water Commission

December 2019

1

Objective of DHARMA Implementation

- DHARMA, a web-based asset management software, will support the effective collection and management of asset data for all Dams in India.
- DHARMA will enable various stakeholders to collect, store and compile basic as well as engineering data, health & improvement data
- DHARMA will allow the systematic presentation and interpretation of data and information for effective monitoring of the health of Dams.
- DHARMA will enable gathering and updating of Dam asset information in a centralised and structured manner so as to overcome limitations.
- DHARMA will not only act as Data Repository tool but also act as Decision Support tool

2

2

DHARMA Initiative under DRIP

Component - I
Rehabilitation & Improvements Works

- DSRP & DFR
- Design Review
- PST Approval
- Procurement
- 3rd Party Supervision
- Rehabilitation Reports

Component - II
Institutional Strengthening

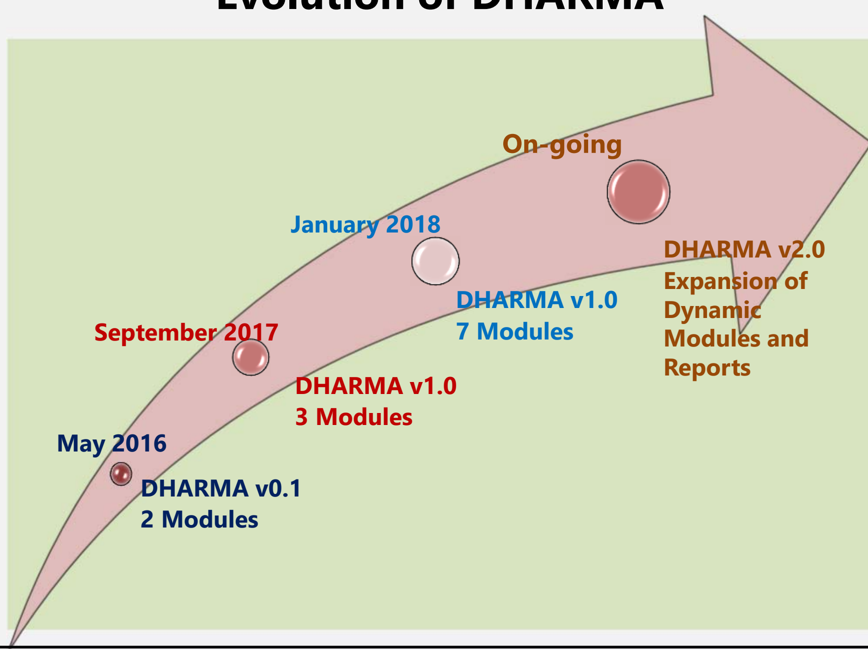
- Capacity Building
- Academic & Research
- Guidelines & Manuals
- ISO Certification
- DRIP Website & MIS
- **DHARMA**
- DHARMA Help Centre
- Dam Safety Website

Component - III
Project Management

- Planning & Monitoring
- Document Management
- Procurement Management
- Financial Management

3

Evolution of DHARMA



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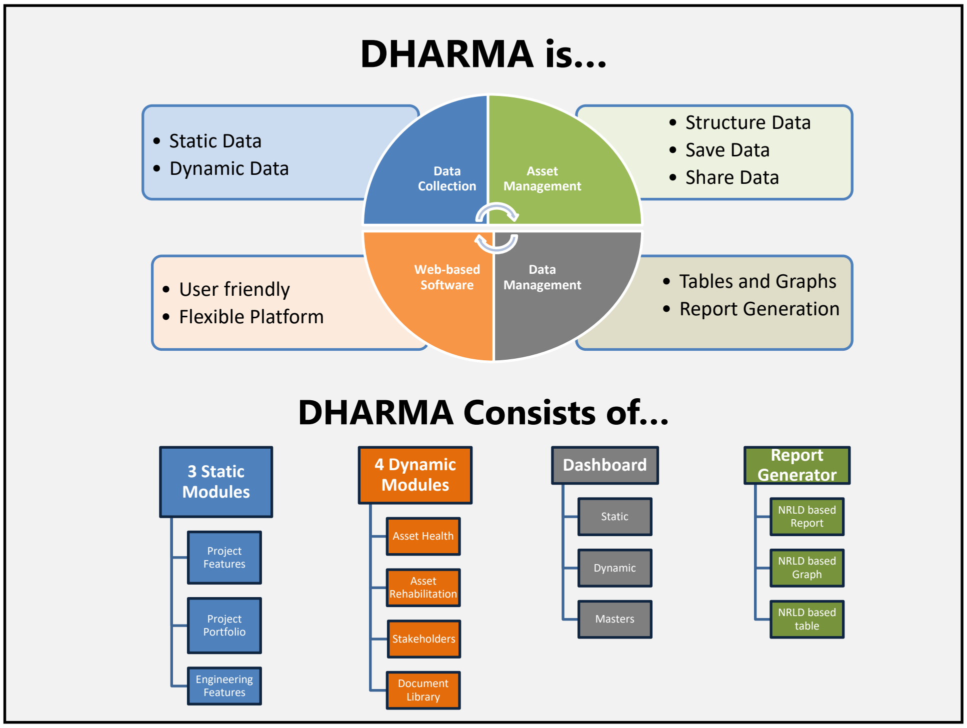
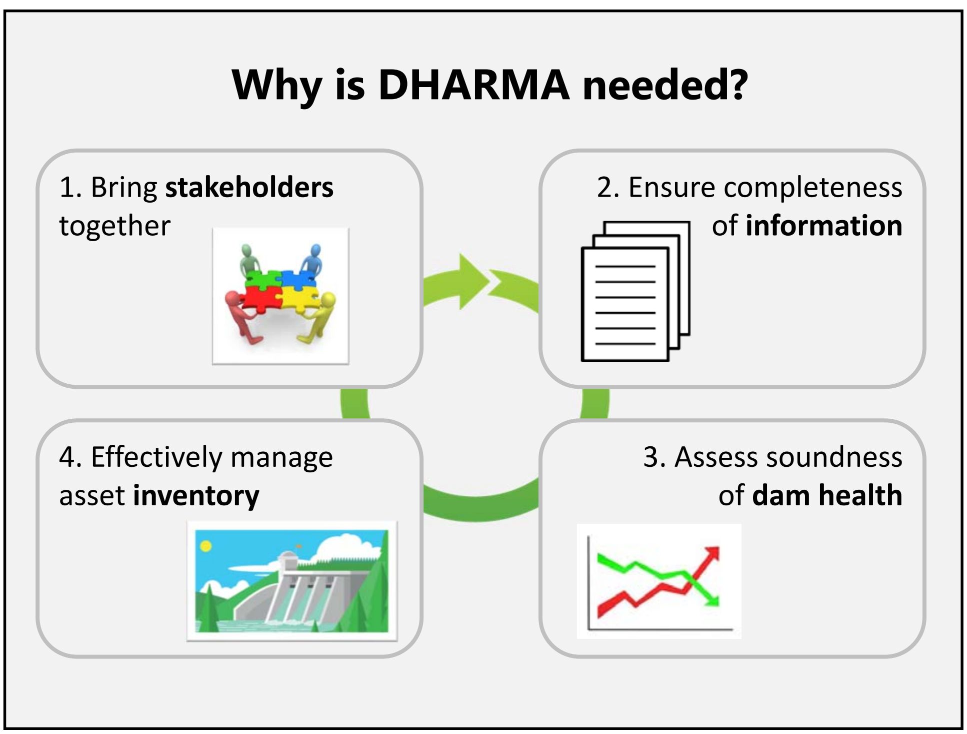

DHARMA is...
Static Data
Dynamic Data
Data Collection
Asset Management
Structure Data
Save Data
Share Data
User friendly
Flexible Platform
Web-based Software
Data Management
Tables and Graphs
Report Generation
DHARMA Consists of...
3 Static Modules
Project Features
Project Portfolio
Engineering Features
4 Dynamic Modules
Asset Health
Asset Rehabilitation
Stakeholders
Document Library
Dashboard
Static
Dynamic
Masters
Report Generator
NRLD based Report
NRLD based Graph
NRLD based table

5


Why is DHARMA needed?
1. Bring stakeholders together
2. Ensure completeness of information
4. Effectively manage asset inventory
3. Assess soundness of dam health

6

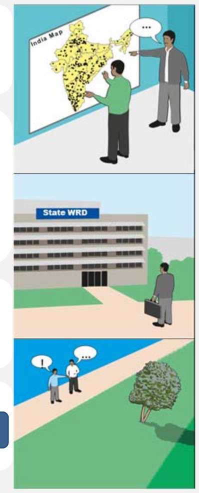
Users of DHARMA

Webmaster
- Web Master

Licensors
- Central Admin
- Central Super Admin
- State Admin

Licensees
- Licensee Admin
- Licensee Super Admin

Application Users
- Dam Data Manager
- Dam Health Engineer
- General User



7

DHARMA Modules

4. Asset Health
Inspection, Instrumentation Investigations, O&M and EAP details

3. Engineering Features
Technical Details of all components (gallery, drain...)

5. Asset Rehabilitation
Rehabilitation works implemented at dam site

2. Project Portfolio
Virtual Model of the dam using 17 components

6. Stakeholders
Contact details of the dam's staff and suppliers

1. Project Features
High level features of the Dam (location, access...)

DHARMA

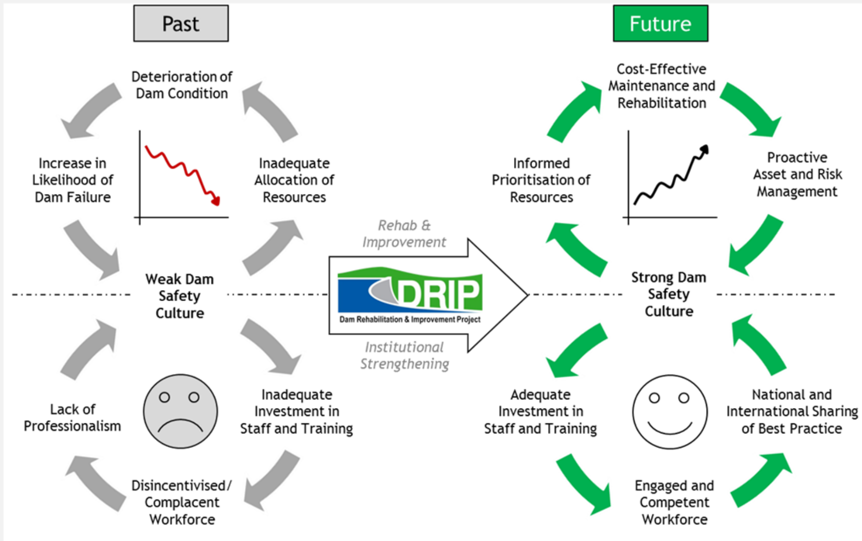
7. Document Library
Database of important dam documents

Static Modules

Dynamic Modules

8

DHARMA as a Catalyst

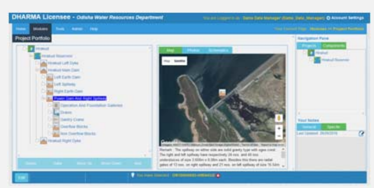


Dam Safety Institutional Context (adapted from Blockley and Godfrey, 2000)

9

DHARMA Progress To Date

- ❑ DHARMA v1.0 released in September 2017
- ❑ 28 trainings held for 1023 participants from 9 States
- ❑ 900 users and over 1000 dams assigned
- ❑ 600 active users have entered data for 1500 dams
- ❑ Successful migration of National Register of Large Dams (NRLD)



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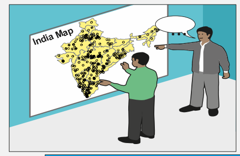
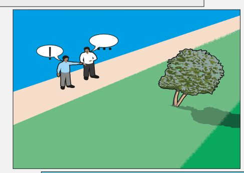
DHARMA Progress and Prospects

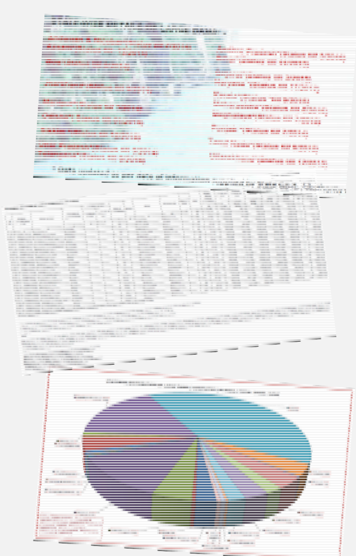
- Digitisation of the National Register of Large Dams
- Modernisation of Dam Safety Inspection Procedure
- Optimisation of planning for Rehabilitation Works
- Complete Asset Management and Decision Support System





11

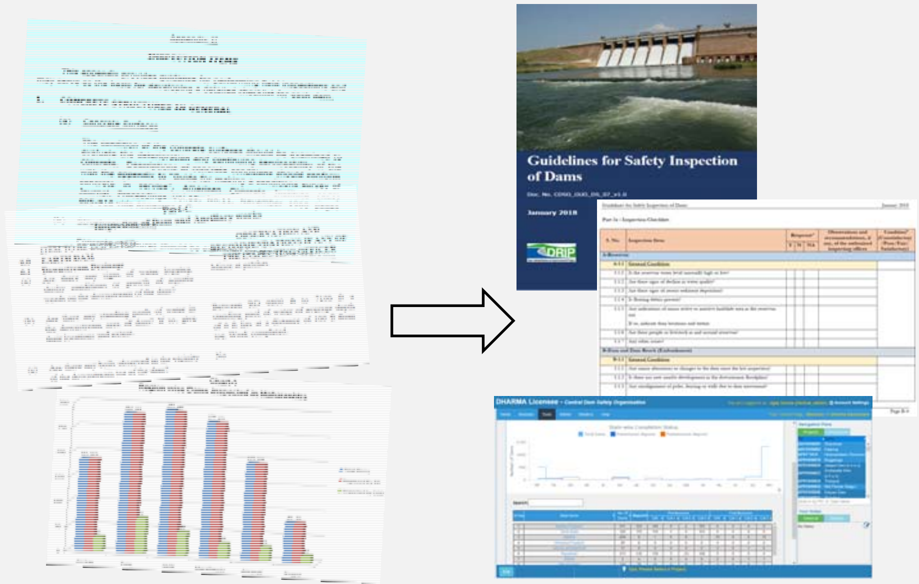
National Register of Large Dams



Manual → **Automatic**

12

Dam Safety Inspections

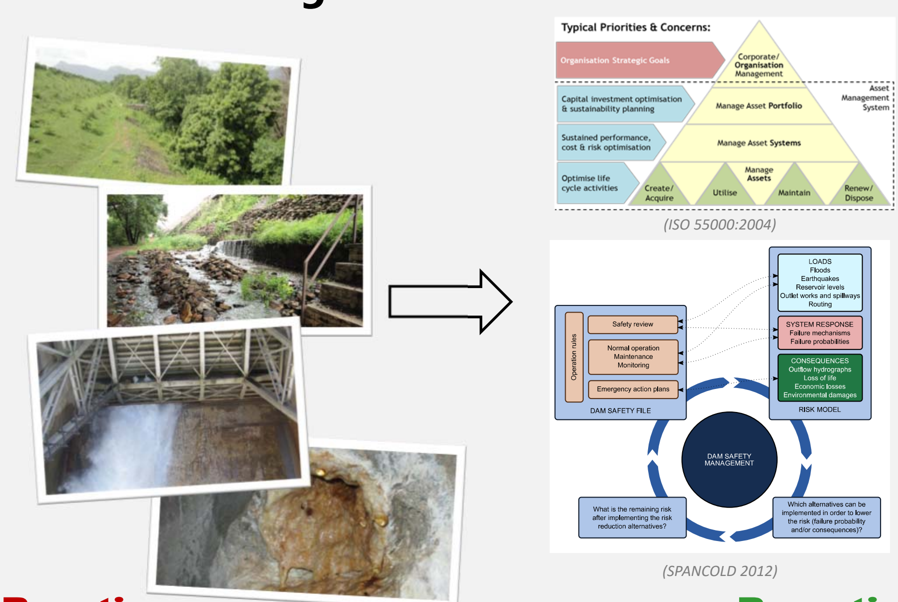


Static Data

Analytic Data

13

Planning Rehabilitation Works

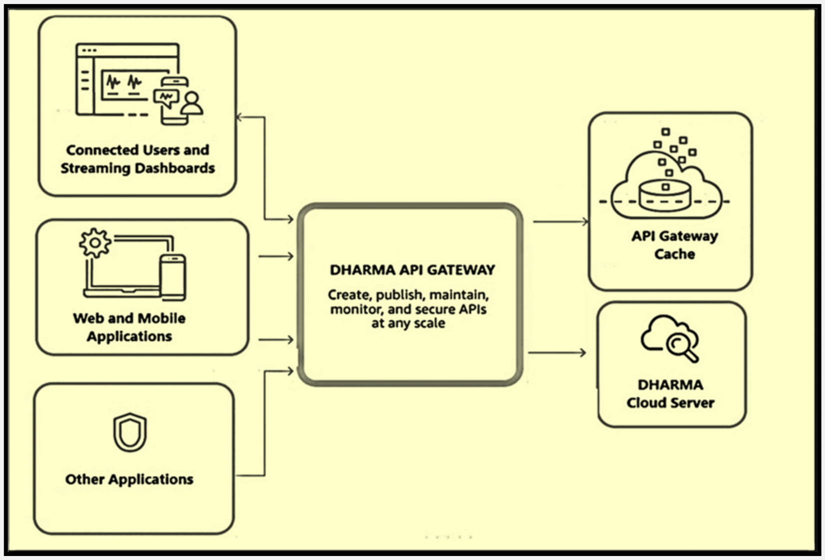


Reactive

Proactive

14

DHARMA Interaction Model with External World



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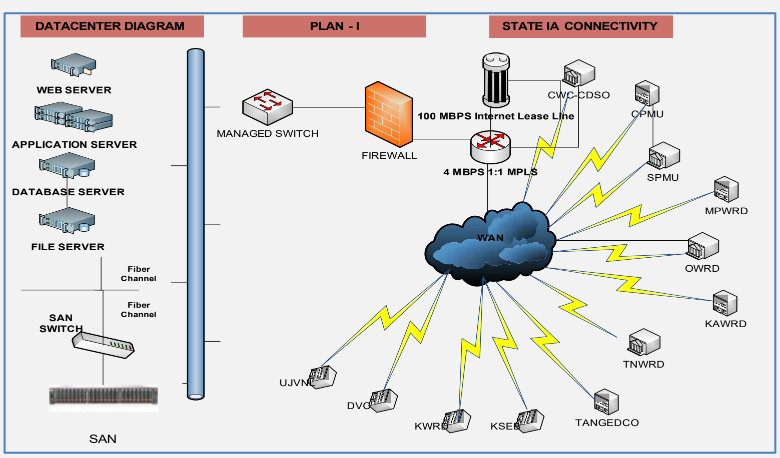
Cloud-Based Web-Enabled DHARMA Network Diagram



16

Conclusions

- DHARMA has made **good progress** but **prospects** remain for its continuous improvement…
  - NRLD successfully digitised
  - safety inspections being modernised
  - planning of rehabilitation works to be optimised
- The **next steps** for DHARMA include…
  - continue training programme
  - complete Asset Management and Decision Support system
  - support implementation of Guidelines and Manuals

17

राष्ट्रीय जल अकादमी

पुणे स्थित राष्ट्रीय जल अकादमी, केन्द्रीय जल आयोग की एक विशिष्ट संस्था है। जल संसाधन क्षेत्र से जुडे राज्य तथा केन्द्र सरकार में विविध स्तर पर कार्यरत अभियंताओं के प्रशिक्षण के क्षेत्र में राष्ट्रीय जल अकादमी एक "उत्कृष्ट केन्द्र" के रूप में कार्य कर रही है। राष्ट्रीय जल अकादमी जल संसाधन के विकास एवं प्रबन्धन के क्षेत्र में अल्प एवं मध्यम अवधि के पाठ्यक्रमों के नियमित आयोजन के साथ-साथ केन्द्रीय जल अभियंत्रण (वर्ग 'क') सेवा के अंतर्गत चयनित अधिकारियों के लिए लम्बी अवधि का प्रवेशन कार्यक्रम भी आयोजित करता है।

राष्ट्रीय जल अकादमी की वेबसाइट http://nwa.mah.nic.in से इस संबंध में अधिक जानकारी प्राप्त की जा सकती है ।