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Earthen Dam Failure and Recommendation- A Case Study

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Abstract: *Nishnap dam located in Radhanagri Tahesil, district Kolhapur, and was built in 1999-2001. Water was stored in the dam till 2004 but due to some reasons, dam did not fulfil the function of storing water. Considering dam has failed in 2004 central portion of the dam was removed due to fear of overtopping of dam. The dam is about to restore and reconstruct to serve its purpose..Geological investigations and meterological investigations are done in order to know the feasibility and subsurface details of hard stratum. Remedial measures are suggested to avoid further failure of dam.The remedial measures include providing cut-off trench, filters ,seepage control measures.*

Keywords: *Earthen Dam, Erosion, Breaching.*

I. Introduction

Nishnap dam located in Radhanagri Tahesil, district Kolhapur, was built in 1999-2001. Water was stored in the dam till 2004 but due to some reasons, dam did not fulfill the function of storing water.Considering dam has failed in 2004 central portion of the dam was removed due to fear of overtopping of dam. An embankment dam is a huge structure made up of impervious soil. Dam may be a composition of clay, soil and rock. Embankment dam can be constructed as a rock filled dam and earth filled dam. Embankment dam is prone to failures more than concrete gravity dam because of earth material having low compressive strength and its low resistant to erosion. Embankment failure occurs due to piping which causes the internal erosion of the dam. Also other reasons to failure of the dam are wave action, piping, insufficient capacity of the dam and erosion of dam .This reasons may results into breaching of dam also erosion of upstream slope and downstream slope, seepage failure, hydraulic failure and cracking in the dam body .This project works on recommending remedial measures for avoiding failure of dam or preventing it from failure. The remedial measures include providing cut-off trench, protection on u/s and d/s slope and seepage control measures. Area in the vicinity of the tank belongs to kaladgi series and member of cuddapah system. These rocks are younger than dharwas and older than vindhyan. They occur in the form of inliers completely surrounded by Deccan trap.

Predominant rock types in the area are Quartzite, sandstone and porphyritic basalt. Predominant rock types noticed along the dam line are quartzite, sand stone and compact to porphyritic basalt, sand stone exposed along the alignment is moderately weathered and jointed in condition.

II. Meterological And Geological Investigation

I. Meterological Investigation

In order to have a brief idea about the failure of the dam, data was collected regarding breach characteristics, piping failures of the dam. Data was obtained from government agencies, reports made using resistivity meter apparatus, water intact test were collected and analyzed. It was found that geology of the area has vertical as well as horizontal faults. Two water intake tests were taken:
1) Before the commencement of the project.
2) After the project was demolished.

Data collected from both these reports are compared and conclusion is made. Quartzite and sandstone are present in downstream as well along the hillock. Quartzite is in its initial stage of formation which gives the possibility of fractures and seepage of water. The reasons for the leakages in the dam were from the dam body as well as a subsurface stratum was responsible.

Rainfall data for this particular site was collected from nearby rain gauge station in Gargoti. Rainfall data for over 40 years is collected and arranged in descending order in terms of rainfall. The rainfall corresponding to 50% or 75% dependable yield is calculated and use for calculations.

Based on rainfall yield and runoff is calculated. For Nishnap dam data since 1977 -1997 is collected and arranged in descending order in terms of rainfall. Catchment area of the dam is around 3.31sqkm i.e. 1.27 sq miles. According to the rainfall statement the 50% dependable rainfall is about 1451.55mm (57.14 inches).

Capacity of dam is 1076 Mcft i.e. 36.56 TCM. Using the yield we can design west weir and tail channel. The area receives highest amount of rainfall around 2000 mm. This means during monsoon season there is possibility that dam is full or water may run over the top of dam.

II. Geological and geophysical Investigation

Geophysical as well as geological investigations were made to determine the depth of hard stratum. Geophysical means included resistivity meter test and geological investigation included water intact tests and bore logs.

Resistivity meter equipment is used for archaeological purpose. Earths composition can be found out using resistivity meter. Resistivity meter is used for recognizing earth's strata and layer below the ground level. Depth of operation of it is 100 -200 meters. The strata form is calculated using Wener configuration by ratio= V/I.

Resistivity meter results showed that hard massive rock was available from depth 10-18 m. The result showed that there were faults present at the depth of 2- 9 m. There were traces of sandstone, quartzite in the area. Along the alignment of the dam there were hard and soft jointed rock .At the toe of the dam there is soft jointed rock and in submergence area there is presence of hard massive rock. Sand, loam, silt, clay is defined by the Values given by the manufacturer

For first geological explorations 5 bore holes were proposed and in second 5 were proposed. The explorations were taken at upstream, downstream, along the dam (centre of the dam), submergence area and on two hillocks. Comparison and analysis of two reports were made. The water intact test was conducted at a depth more than 25 m. In first exploration it was till 10 m of depth. Second geological exploration was done and 5 bore holes were taken. This time depth of exploration was about 30 m. The bore holes location were centre of the dam, One at the upstream toe, one at submergence , two in hillocks for assuring there is no seepage through the adjacent portion .

Results:-

Bore hole no 1:-In first report the rock present at a depth from 0-6 m was quartzite and below 6m was compact basalt.

Bore hole no 2:- In first report the rock present at a depth from 0-6 m was quartzite and below 6m was compact basalt which was considered as hard strata.

Bore hole no 3:-In first report the rock present at a depth from 0-6 m was presence of sandstone and quartzite and below 7 m of depth presence of hard massive basalt was shown.

Average loss of water in First geological exploration

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Bore Hole	Location	Average Lugeons
Bore Hole No 1	Chainage 80 U/S 100m	63.47
Bore Hole No 2	Chainage 80 D/S 60 M	43.33
Bore Hole No 3	Chainage 80 U/S 100m	67.83
Bore Hole No 4	Chainage 102.50 U/S 115	49.50

Average loss of water in Second geological exploration

Bore Hole	Location	Average Lugeons
Bore Hole No 1	Chainage 80 C/C	78.36
Bore Hole No 2	Chainage 80 U/S Toe	88.36
Bore Hole No 3	Chainage 110	73.36
Bore Hole No 4	Random In Submergence	62.25
Bore Hole No 5	Chainage 50	104.00

Findings:-

Bore hole no 1:- In second report rock present at 0-6.25 m jointed quartzite rock. And strata below have hard jointed quartzite rock .Below 11 m there was hard quartzite rock.

Bore hole no 2:-In second report it was stated that presence of hard strata was strata was at a depth below 12 m . The rock type was jointed hard quartzite.

Chainage 50 and 110o are taken in gorge section. Bore hole at chainage 50 couldn't be completed due to presence of loose and soft jointed rock.

Presence of compact basalt was in submergence and very less loss of water.

At chainage 110 the rock was hard and massive at a depth below 20 m.

Water loss observed in both tests had variance as the depth explorations differ in both tests. Water loss in second tests was in variance with respect to depth.

III. Comparison of two reports

Comparison of two reports:-

1) Intact test at centre of dam: - In the first report the water loss about 14-35 liters at about 7 m that means there was presence of hard rock at the depth of 7 m. Second test were contrary to first one as the time duration for test was 5 min and loss was 80 litres up to depth of 10 m and above.
2) Intact test at gorge portion: - At chainage 50 the water intact test couldn't be completed due to stratification of the strata. The bore drills cannot be recovered and the water loss was about 100 liters and above .At chainage 110 gorge portion the water loss reliable say 75 litres up to depth of 12 m from bottom of the gorge) and was no loss through gorge.
3) Intact test at toe: - Test taken at toe had a large variance and didn't match with the existing dam structure. Water loss was high at the depth at depth of 5 m and below. Continuous seepage of water was seen .The inclined faults and fractured subsurface strata may lead to leakage. First report showed the water difference of 16-36 litres.
4) Intact test in submergence: - Tests conducted in submergence showed that there was presence of hard basaltic rock and less loss of water.

Comparison of two reports

	First report	Second report	Analysis
Geology of the area.	Presence of basalt and quartzite rock at and near dam body.	Presence of only quartzite rock at dam body. Jointed and soft fractured segments of rock present.	Sandstone present in submergence area . Jointed and soft fractured hard quartzite present below dam body.
Depth of various rock strata	Hard massive rock was available at the depth of 6m and below	Presence of jointed hard rock at the depth 10 m and below.	Jointed and highly fractured rock present till depth 10 m and jointed hard quartzite below 11.5-12 m.
Rock characteristics	Hard Quartzite and sandstone with low permeability with fissures and faults in the structure.	Presence of jointed hard quartzite at the depth of 10m and below. Also at gorge portion presence of soft fractured rock having high permeability. Compact sandstone in submergence with low permeability.	High permeability was observed at the gorge (chainage 50) and toe area which had a loss of water over 200 liters. Permeability least observed in submergence.

III. Reason For Dam Failure

Analysis of reports was done on the basis of their determination of sub surface strata. For analysis the reports made by Prabhakar Jadhav senior geologist from CSIBER, Kolhapur conducted a resistivity meter test and water intact test did both times. The analysis was the variance of reports which gave the reasons for seepage and failure of dam.

Geology of the area: - The structure consists of main dam and a saddle dam. Explorations done both times had brief idea about the geology and rock characteristics present in the area. Hard quartzite and jointed soft fractured rocks were discovered near the dam structures. These have cleared the ideas for sub surface strata and seepage path of water. Depth and duration were different for both the tests. Presence of various rock strata is given below :-

Quartzite:-The depth at which quartzite was discovered was 12m and below. The quartzite was soft jointed in west weir portion. The main dam had a quartzite in shape of fractured portion and very brittle.

The gorge was also made up of quartzite. The property of quartzite is its crystalline nature and failure to sudden load .Permeability of quartzite is about 10-7- 10-2 cm/sec .But if fractures and joints are available it may have high values up to 10-3 and greater.

Sandstone: - The presence of sandstone was seen in submergence .Low metamorphosed sandstone presence was seen in upstream portion of the dam. The fractures and faults were not at depth greater than 3 m .Sandstone was of compact type that means minimal loss of water is possible.

Faults and fractures: - Soft jointed quartzite is present in the west weir portion, which is fractured rock. Faults are vertical as well as inclined under main dam. These are found due to steep gradient of the area. The water loss at various bore holes and the points connecting the very loss of water shown the faults present at the site.

Design parameters considered while designing a dam are soil weight, rock characteristics and lugeons value obtained for particular strata .Soil weight is used to design and analyze the stability of dam structure. Rock characteristics and lugeons value obtained is used to design impervious blanket, C.O.T depth, drainage filters. Core size is decided on availability of materials, topography of site, foundation conditions, and diversion conditions. Design of dam also depends on the placing technique of various elements such as placing riprap, material used for core and casing, and position of strata at subsurface. Various conditions on site maybe different and cannot be carried according to IS code or design given.

Details and specification

Elements	On site specification	IS code specification
Core	Core has top width of about 3m and slope given at u/s is 1:1 and downstream 2:1.It has a vertical core.	IS: 8826-1978 Core should have minimum width of 3m so it can have wider width.
C.O.T	C.O.T depth was 6.40m and width was 5m and hearting material was compacted at the depth of 1m for better seepage control.	IS: 12169 Bottom width of C.O.T is 0.1 to 0.3H and 0.5m in to the hard strata.
Drainage filter	Drainage filter provided for the rock toe in order are 150mm thick sand layer ,250mm thick graded gravel,250mm thick quarry spalls and a toe drain	IS: 8237 Thickness is governed by wave action .Two layers of filter are usually necessary Coarse and fine. Thickness of filter layer for coarser and finer is 150mm.
Upstream protection	1) Upstream protection was done by 300mm thick stone pitching and 150mm thick spalls.	IS: 8237 Upstream protection of dam is done by RIPRAP. The riprap depends upon wave height and placing of riprap technique.

Soil compaction is defined as the method of mechanically increasing the density of soil. In construction, this is a significant part of the building process. If performed improperly, settlement of the soil could occur and result in unnecessary maintenance costs or structure failure. Almost all types of building sites and construction projects utilize mechanical compaction techniques.
From the field investigation the soil properties so obtained were:-
Compacted density= 1.42 g/cc
Standard proctor test results.
Dry density = 2.57 g /cc water content = 17.25%

Compaction to be achieved on the field is defined on soil characteristics whether it is clay, silt, gravel or cohesive. Soil discovered on the site is red laterite soil which can be easily stripped of when vegetation is removed. The interlocking of soil can be achieved with the help of one of compaction methods.

Conclusion :- The dry density given by CDO(Central design organization) MERI (Maharashtra Engineering research institute) ,Nashik was 2.65g/cc .The dry density obtained from standard proctor test is 2.53 g/cc .This shows variance of soil density to be achieved on site . The soil used is locally available soil which is impervious clay with dry density 2.5 – 2.7. g/cc.

According to tender documents and specifications provided by Government of Maharashtra ,criteria for control of compacted dam embankment are for hearting :-

1. Minimum acceptable density 98% of proctor
2. Desirable average density 100% of proctor
3. Desired moisture content 2% of O.M.C
4. Compaction should be done on +2% of wet of optimum.

From above norms it can be concluded that on site the required compaction is

1. Minimum acceptable density is 98% of 2.65g/cc
2. Desirable average density is 100% of 2.65g/cc
3. Desired moisture content is 2% of 17.57%
4. Compaction should be done on +2% of 17.57% of omc.

Permeability of rock strata is the count on which embankment dam is built. According to IS 6066 the permeability of rock so needed is 10-4cm/sec . The obtained permeability of the rock is 10-3 and above in the site region. This clears the seepage problem of the dam that resulted into erosion. The compact basalt encountered in submergence has permeability of about 10-7cm/sec. According, to permeability obtained from above tests is concluded that site has fractured structure and with that the seepage of groundwater has incurred.

IV. Remedial Measures

I) Protection on downstream side

The site lies in heavy rainfall area and so pore pressure and water seepage problem is at high risk .This may lead to piping problem in the dam. Extra provisions may be given for protection of downstream side as the water seeping through abutments and u/s face can be collected through drainage provided in the dam.

Reason:-

The previous provision for a toe drain and cross drain was not enough to drain away excess amount of water from adjacent structures . A system of "open paved drains (chutes)" along the sloping surface terminating in the longitudinal collecting drains at the junction of berm and slope shall be provided at 90 m centre-to-centre to drain the rain water (IS-9429).Providing drains may reduce the seepage path of water which will travel from u/s face of the dam to the toe drain instead the water can be thoroughly connected from the path it travelled and avoid saturation of embankment material. The spacing may be reduced based on regional climatic conditions and type of slope protection. The site has closely spaced hillocks closely spaced and steep slopes. The water flowing over the surface of hillock may run over the downstream face of the dam .Pitching provided may not be enough for protecting d/s face of the dam from action of water.

Specifications:-

1) The longitudinal drains shall be provided with 150 mm diameter pipes laid at a slope of 1: 50 for discharging the collected water to the downstream. If cross drains are provided, the lowermost sloping drains shall drain into the cross drains. Otherwise the sloping drains shall be connected to the toe drain.
2) Where internal drainage arrangements have not been provided in the body of the dam or in cases where rock is readily available from compulsory excavation, rock-toe of 0.2H (where H is the hydraulic head) should be considered.
3) The maximum and minimum height of rock toe shall generally be 6 m and 1.5 m. In case the maximum tail water is expected to rise above the crest of rock-toe 300 mm thick riprap with proper filter shall be provided from crest of rock-toe to an elevation 1 m above maximum tail water level.
4) **Toe drain:-** Toe drain is provided at the downstream toe of the earth/rock fill dam to collect seepage from the horizontal filter or inner cross drains, through foundation as well as the rain water falling on the face of the dam, by suitable means according to site conditions. Additional longitudinal drain and cross drains connected with the toe drain are sometimes provided where outfall conditions are poor. It is preferable to provide the toe drain outside the toe of rock toe, to facilitate visual inspection. The section of the toe drain should be adequate for carrying total seepage from the dam, the foundation and the expected rain water.
5) **Specifications:-**Depth of toe drain is 0.60 m and maximum depth is 1.20 m. It is constructed away from the downstream at a depression or river fall.

II) Compaction:-

Leaks in earth-filled dams that lead to dam failures are often the result of inadequate compaction levels. Therefore it is important that effective compaction is achieved. This can be undertaken by applying the required compaction effort to high clay content materials.

Reasons: - Backfilled material and material used for hearting should be compacted to nearest omc having dry density greater +2. This is necessary because the soil obtained from the nearest quarry has density of about 2.5-2.7 g/cc and to achieve close watertight contact in the regions of irregularities. There are irregularities observed on site due to blasting and cutting of abutments. Compaction near the edges of the section should be done thorough and cutting and benching should be adopted to achieve good bond between old and new surface.

Provisions undertaken to achieve proper compaction are

1) All fill material for the embankment should be placed in layers (or lifts) no greater than 150mm thick.
2) Each layer should be thoroughly compacted before the next layer is placed. A minimum of 6 spasses to achieve the required compaction effort is generally required by a suitable machine.
3) Care should be taken material should not swell and crack during initial compaction.

Concrete structures

Reason:-

Concrete structures involved in an embankment dam are conduit and gated well which controls the flow of water into the canal .The faulty construction of the conduit was inspected by field officers when the closed the gates they heard water falling inside the conduit. Later it was seen that crack were developed at the floor of conduit and the concrete was washed away due to heavy discharge of water through it. Conduit was designed to carry maximum discharge of 0.09 cumecs. It was designed for maximum operative head of 16 m and the water raised during the failure was observed about the crest level of the dam. As we know the advantage of gravity dam is it can sustain pore pressure and allow water to stand at u/s face but in embankment dam it creates seepage forces and form internal erosion in dam. The concrete work in conduit section and approach channel should be such that it can carry large amount of water for short duration without causing any harm to dam body.

Provisions for concrete structures:-

According the IS-CODE 457-1957 " Code of practice for general construction of plain and reinforced concrete for dams and other massive structures" used in dams and other structures may vary in character from mass concrete having a maximum size of aggregate from 10 cm (preferably 15 cm) to 23 cm [or 4 in (preferably 6 in) to 9 in] and a cement content ranging from about 150 to 235 kg per cu m (or 250to 400 lb per cu yd) to heavily reinforced concrete having a maximum size of aggregate of 2 cm (or 3/4 in) and a cement content of approximately 355 kg per cu m (or 600 lb per cu yd). To affect the greatest economy, the concrete should preferably contain the maximum size aggregate for the place of use and giving the specified strengths.

1. All flat surfaces shall then be coated with mortar about 1.5 cm (or 4 in) thick in the case of concrete surfaces and 2 cm (or 2 in) thick on rock surfaces.
2. No concrete shall be deposited until the foundation has been inspected and approved. Where the rock is dry enough to absorb water from the mortar layer, it shall be soaked for at least 24 hours prior to placing the concrete. Detailed instructions shall be issued for preparing scaly or cracked foundations requiring special treatment or grouting.
3. In the internal portions of conduit guiniting should be done. On the UCR massonary foundation sand mortar based grouting should be done near the west weir portion wing wall which is exposed the jilts should be filled with the help of guiniting.
4. If repairs are considered for the conduit structure then the mortar shall be placed in layers of 2.5 cm (or 1 in) each layer thoroughly tamped, and the finishing layer shall be smoothened to form the surface continuous with the surface of the holes and shall be sound and free from shrinkage cracks and surrounding concrete. All filling shall be bonded rightly to the hollow areas after the fillings have been cured and have dried.

III) Protection on upstream side and downstream side against pore pressure
Reasons

Pore pressure can be reduced in dam by providing filters at upstream and downstream side. The water can be collected from the upstream side of the dam and can be discharged through filters on downstream side. The horizontal filters and inclined filters may help in reducing the internal erosion and piping in the dam body. In the design of the filter previously there were provisions been made for longitudinal drain and impervious core. Horizontal and intermediate filters are provided so as to reduce pore pressure during prolonged rainfall. The reason behind providing these filters is this site lie in heavy rainfall zone and needs a proper drainage surface for the increasing pore pressure.

The thickness and vertical interval between the filter layers are to be decided depending upon the height of the dam and the range of permeability values of shell material and filter material. This avoids excessive flattening of upstream slope from considerations of stability.

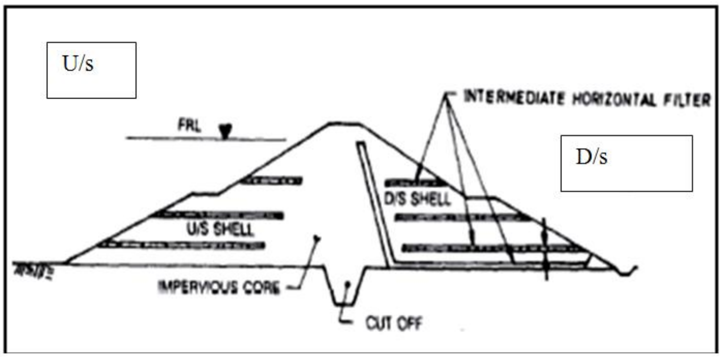

Fig No. 6.3 Protection on Upstream and Downstream (Horizontal Filter)

The thickness of intermediate filter provided should be 50 cm. On the abutments, seepage through the dam travels towards the foot of the embankment mainly through the inclined filter. Seepage water may not travel through horizontal filter and toe drain. However, horizontal filter should be provided to take care of rainwater and some part of the seepage through dam and abutment.

Provisions:-

1. Horizontal filter should be provided at minimum slope of 1 in 100 towards the toe drain is desirable for quick disposal of seepage water.
2. Provision of 0.6 m thick layers at about 6-m intervals, according to permeability of the core.
3. Horizontal filter should not be provided where FRL touches the ground level.
4. For avoiding the erosion from tail water, which was seen in the above site, riprap of 300 mm thickness over proper filter layers shall be provided up to 1 m above the maximum tail water level.
5. During monsoon season there is possibility of heavy rain through the hillock which may damage the dam body. To avoid that catch water drain should be provided and water be should transferred elsewhere.
6. If already constructed drains are in good shape they shall be connected with L drain and shall be removed totally and reconstructed to ensure free drainage.
7. A toe drain at a distance of 8.0m from the downstream too shall be removed and totally is reconstructed..

Control of seepage through abutments:-

In Spite of all precautions like benching, stepping and proper compaction the possibility of weak spots remaining at the junction between new and old embankment cannot be ruled out. In order to prevent seepage through the weak spots following provisions should be adopted:-

1. 1.2m thick layer of hearting should be laid, later 1.2 m thick of casing shall be laid.
2. After reconstruction of the gorge the rock toe should be reconstructed as below:- Height of rock toe should be maximum of the following :-

i) Such as to be 1.0 m above T.W.L or 15% of hydraulic head
ii) Such as to be 0.50m above horizontal filter may be provided.
iii) The height of rock toe worked out in 1 and 2 above will be subjected to a minimum of 1.0m and limited to 4.00 m

IV) Under-seepage control measures:-

Positive –Cut off trench:-

The positive cut-off trench consists of an impervious fill placed in a trench formed by open excavation into an impervious stratum. Grouting of the contact zone of the fill and the underlying strata constitutes an integral part of the positive cut-off. The pockets of particular size that compaction equipment cannot be operated and pot holes with overhangs should be filled with concrete.

Reasons

The geological strata present on Nishnap site has fractured and fissured rocks .The water loss obtained water intact test were about greater than 50 lugeons. White Quartzite rock present in the subsurface strata .The hard impervious stratum is present below 10m of depth. Flatter slopes are generally provided for stability of c.o.t. Impervious blanket is generally proposed for the purpose of avoiding piping and seepage through dam. Partial cut-off is a trench where it does not penetrate through impervious stratum and provided to a certain depth. This partial cut-off can be provided at u/s toe of hearting to avoid seepage of water through dam. On site it was seen that there was a passage of flowing water which appears at upstream 270m u/s in a well and later appears at downstream 80m d/s which means there is no trace of flowing of water on surface in between these two points. The pore pressure because of this water may be high due to fractured and fissured rocks.

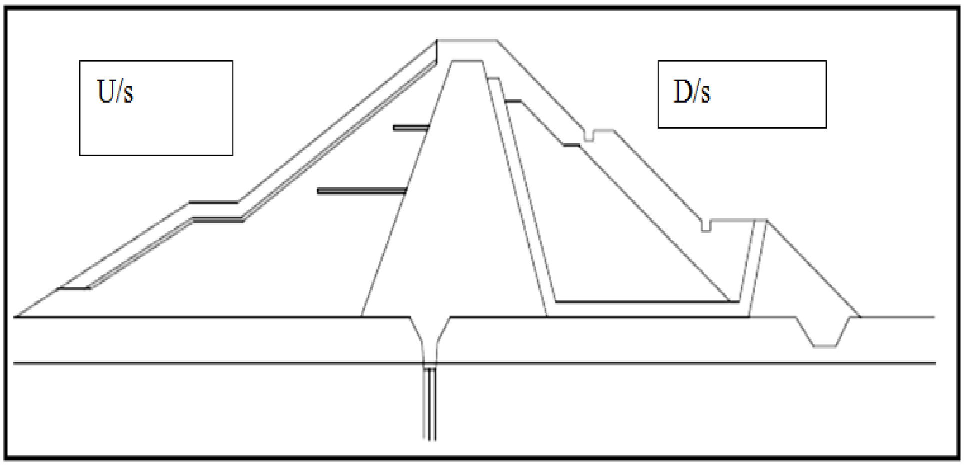

Fig No. 6.4 Positive Cut-Off with horizontal filters

Specifications:-

The depth of the positive cut-off trench is governed by the geological features influencing the configuration of the impervious substratum and the profile of unweathered mass of bed rock. Depth of positive c.o.t is 0.4m into impervious strata. For positive cut off slope should be ½:1 and ¼: 1 or flatter is provided.

The width end side slopes are generally selected according to convenience of construction and to ensure stability of excavated slopes. Detailing of the positive cut-off near abutments and junctions with structures needs careful consideration, since the efficacy and continuity of the impervious filling (both in the cut-off trench and in the core) are compromised by differential settlement cracks developed near junctions, abutments and foundation irregularities .Minimum width is 4 m and bottom width is 10 to 30% of hydraulic head .

In addition to that an impervious blanket of 1.20m thick laid over the top width of this C.O.T is also proposed .The surface shall be stripped off by minimum 0.50m then this surface shall be covered by impervious material of 1.20m thick which is then shall be covered by 1.50m thick casing material. Before doing new earthwork proper stepping and deep benching should be done to have good bond between old and new earthwork.

Also partial cut-off can be proposed, as partial cut-off is suited for horizontally stratified foundations with nearly more pervious layer on the top. A cut-off which does not go down to impervious stratum.

Diaphragm-wall :-

Where the foundation strata consists of boundary deposits, open gravel pockets, zones of talus, slide areas, contacts of formation of different geological age, fractured zones or similar features, it is obvious that voids/openings may be larger and contact with diaphragm wall face may not be continuous. Under such circumstances it is important

To see that bending stresses between contact free lengths are taken care of. Any flexible type of diaphragm wall which undergoes large deformations at low stress levels is not suitable. Rigid type diaphragm wall is preferable under such conditions.

Reason

The material used for hearting is very fine and the meterological conditions present on the site suggest that a internal barrier in the form of diaphragm wall should be used for the purpose. This diaphragm wall helps in transferring the percolated water to the impervious blanket and through that to the toe drain. Also the diaphragm wall can be used at u/s toe as the geological strata present is fractured. The diaphragm wall at u/s toe will avoid the under seepage and transfer it to longitudinal drain and with that to d/s face of the dam.

V. Types Of Diaphragm Wall

Depending on the use of construction materials there are the following types of diaphragm walls:
Rigid type - Enforced cement concrete.
Flexible type - Plastic concrete, Cement bentonite slurry trench, and Earth backfilled slurry trench.

Flexible diaphragm wall is selected as it is suitable for embankment as it behaves well under changing loads and pressure. Cement bentonite diaphragm wall should be constructed because of its high compressive strength over plastic concrete and earth backfilled slurry trench. In this type of diaphragm wall the backfill is generally made by mixing the in-situ materials obtained with slurry during excavation of the trench and earth material from additional sources, if required. As cement is not used, it will not be possible to obtain compressive strength. Hence imperviousness is the governing criteria. Plastic concrete has permeability more or equal to conventional concrete but when cement bentonite is added to conventional concrete the permeability is much reduced.

Provision and specifications:-

Provide cement bentonite diaphragm wall type as under seepage control measures. In this proposal diaphragm wall is 15.0m in impervious strata and 0.60 m thick. This method uses water bentonite slurry to support the hard strata. A slurry trench is excavated in the overburden which employs the method for side stabilization and is displayed by cement bentonite slurry usually made by mixing bentonite clay having a high content of mineral montmorollite and water.Cement bentonite diaphragm wall is flexible and eliminates joints between the panels and thus brings overall imperviousness.This wall shall be also covered by 1.20 m thick impervious blanket and 1.50 m casing cover as described above.

D. Optimize solution

From above remedial measures suggested the optimized measures to be adopted on site depends on its feasibility. The measures suggested will help in seepage control through and under the dam .

The foll..owing selective measures from above are suggested for seepage control and to avoid allied failures :-

L-drain should be needed on the hillocks as the site lies in heavy rainfall area. With that an impervious blanket to be provided at the base of embankment to avoid percolation of water. The filters (Refer 6.1) in the dam should be connected by intermediate filters to allow seepage water to be transferred on d/s side of the dam.

Compaction of earth material is very important as the embankment is made up of very fine silt clay. The density to be achieved is 98% of 2.65 g/cc that the thickness of the layer shall be 20 to 23 cm loose and 15 cm compacted for sheep foot rollers, 15 to 11 cm loose and 11 cm compacted for smooth wheeled rollers and 8 cm Loose and 5 cm compacted by hand or pneumatic or mechanical tampers.

1. Under seepage was considered as a major problem for the failure of dam . The under seepage control measures such as positive cut-off up to impervious stratum that is till depth 12m should be provided for seepage problem. Diaphragm wall can be provided till depth of 7m with bentonite slurry filling.Upstream toe RL is 73.5-7=66.5m. depth diaphragm wall should be provided.
2. Abutments adjacent to dam have quite irregularities due to blasting of dam. For this purpose proper stepping and benching should be provided with flatter slope such as ½:1 to achieve stability and good bonding between old and new joints. Flatter slopes provide water off away from the fill.Horizotal filters provided in dam will allow percolation of water from abutments to transfer on downstream side of the dam(Refer 6.6.2)
3. Conduit reconstruction is to be done, as it failed to full fill its purpose.The previous conduit was made to operate under operative head of 16m which was insufficient for the dam. For the proper functioning of conduit, it should operate under operative head of 18 m. If not possible bypass pipes should be constructed to carry away the surplus water to downstream.
4. Concrete structures (approach channel) to be constructed with proper supervision as the concrete were washed away during first operation. Concrete should be compacted of excess water and should be protected from heating and drying for at least 72 hours initially after placing and cured for 14 days . Water should be allowed to flow through conduit for testing the quality of concrete work.

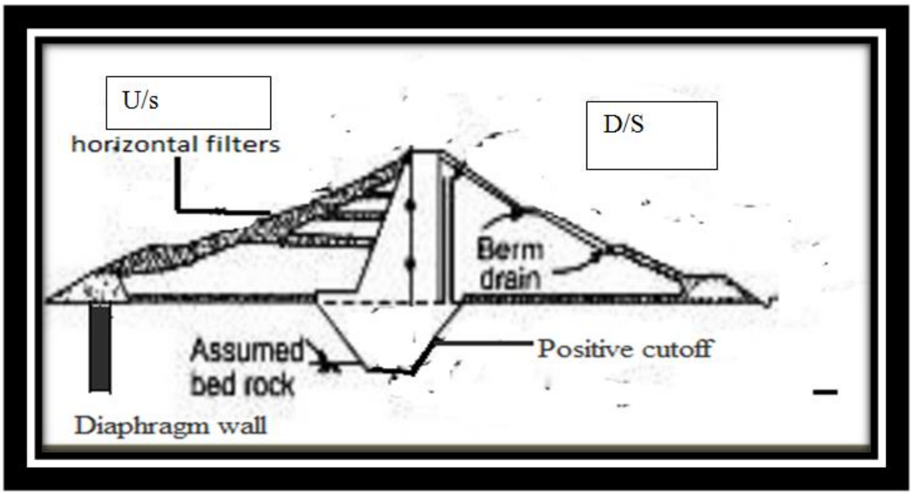

Fig no 6.6 Optimized solution

References

[1]. Amit Srivastava. (2011),"Geotechnical Instrumentation, monitoring and Surveillance in earth dam safety program". Indian Geotechnical Conference,Kochi.

[2]. Huy Thanh Pham, Htet Zaw Oo, Cheng Jing.(2013) ,"Stability of Slope and Seepage Analysis in Earth Dam Using Numerical Finite Element Model".Study of Civil Engineering and Architecture (SCEA) Volume 2 Issue 4.

[3]. Jay Greska, Bryce Mochrie ,Chii-Ell Tsai .(2013),"DAM BREACH ANALYSIS SIMULATION ON THE LOWER SUSQUEHANNA RIVER" . Journal of the Hydraulic Engineering Division, ASCE.

[4]. Thomas C. MacDonald1 and Jennifer Langridge-Monopolis2. (1984), "BREACHING CHARACTERISTICS OF DAM FAILURES"**.** J. Hydraul. Eng. 1984.110:567-586**.**

[5]. Vijay P. Singh,1 Member, ASCE, and Panagiotis D. Scarlatos ,(1988)"ANALYSIS OF GRADUAL EARTH-DAM FAILURE" .J. Hydraul. Eng.114:21-42.

[6]. Xu1 and L. M. Zhang, (2009), "Breaching Parameters for Earth and Rock fill Dams". J. Geotech. Geoenviron. Eng 135, No. 12.