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ORIGINAL PAPER

Geotechnical Problems in Dams: Case Studies

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Abstract Geotechnical engineers and designers, many times come across a problem for which a conventional solution may not work and needs an innovative technique, to solve and complete a project. Three case studies stated below explained the innovative design used, foundation problems encountered with solution, innovative construction technique used and performance after completion.

Keyword Geotechnical problems in dams (GPID)

Case Study I: Geotechnical Problems, Design and Construction of Coffer Dam on Narmada River for Indira Sagar Project in Madhya Pradesh.
Case Study II: Anchored RCC Diaphragm Wall Coffer Dam for Bisalpur Dam Project in Rajasthan.
Case Study III: Restoration and Rehabilitation of Old Pagara Masonry Dam by Grouting Technique in Madhya Pradesh.

Above case studies are dealt, one by one below:

Case Study I: Geotechnical Problems, Design and Construction of Coffer Dam on Narmada River for Indira Sagar Dam Project in M.P.

Abstract Indira Sagar Project in M.P., is a multipurpose project. During post monsoon period, a flow of 300 cumecs of river Narmada was to be diverted so that main dam could be constructed. For this purpose, it was necessary to construct a 24 m high coffer dam to divert the post monsoon flow of river through a diversion tunnel from left abutment. This coffer dam was a very important component, to construct a 92 m high main Indira Sagar Dam and was to be founded on a very complex geological strata. To construct this 24 m high coffer dam, a new concept of innovative design used for first time in any country to construct a coffer dam, using 5 tons pre-cast blocks as shuttering on both faces of a coffer dam and filling the enclosure with boulders and stonecrete them under water. The work of 24 m high coffer dam has been done under water for 12 m height using stonecrete and balance 12 m by conventional stone masonry. This paper narrates the fine intricacies of geotechnical problems faced for this under water structure of 12 m height, along with its design and unique construction under flowing water condition.

Introduction

Indira Sagar Project in M.P., a multipurpose project, comprises of a 92 m high, 653 m long concrete gravity dam on Narmada river, with installed capacity of 1,000 MW Hydel Power (8 units of 125 MW each) provides annual Irrigation to 1.69 Lakh ha and 74 MCM (0.06 MAF) drinking water to rural areas of M.P.

Indira Sagar Project is situated near Punasa about 60 km from Khandwa Town and 120 km from Indore City in Madhya Pradesh. The dam and power house complex of the project was constructed in a period of 13 years from 1992 to 2005. Main Dam is shown in Fig. 1. To construct this dam, it was essential to divert Narmada river during off monsoon period from the original flow route, so that dam can be constructed without any hindrances. For this,

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Fig. 1 Indira Sagar main dam on river Narmada

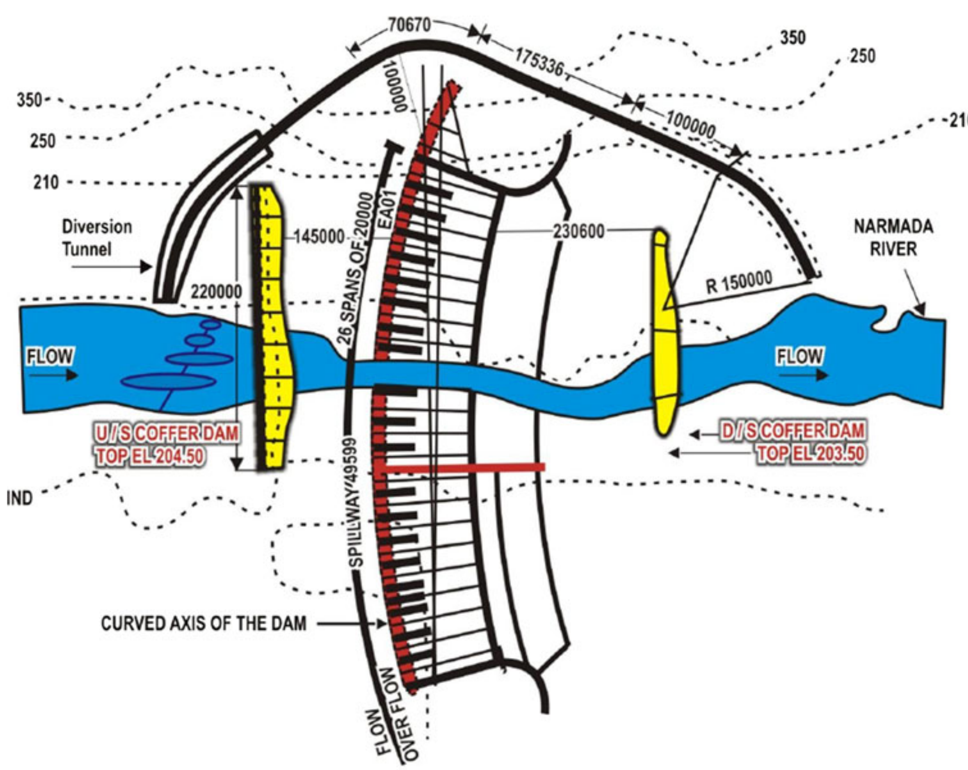
conceptually it was necessary to have two important Component i.e. one upstream and downstream coffer dam to stop and divert Narmada river flowing from the original flow route, where proposed main dam was to be constructed and another a diversion tunnel through which Narmada post monsoon flow of 300 cumecs can be diverted during working period. That is how the construction of upstream and downstream coffer dam was necessitated. A sketch showing the various component like, upstream and downstream coffer dam diversion tunnel, proposed main dam etc. is shown in Fig. 2.

Geology and Geotechnical Problems

The Geological survey of India had carried out elaborate geological studies at the India Sagar Project dam site. It has been indicated that the dam site is located in the upper vindhyan inter bedded sequence of tough quartz arenites (quartzite), sand stones with minor silt stones. The bed in general has an ENE-WSW strike and dip by 15–25 towards NNW with exceptional steep dips of 40–45 due to local warping. Bedding shears of 10–25 cm thickness confined to the silt stone beds are common features. The dam area in the river bed is occupied by a number of ENE-WSW trending vertical fault/shear zone, indicating horst/grabben structure showing relative vertical displacement of blocks. It is WSW continuation of the Sone-Narmada fault. Mapping of the area has identified about five shear zones ranging steep dipping to vertical. These zones are braided with clayey gougey shear seams of 0.10–0.75 m thickness enveloping competent fractured lenses of quartzites and sand stone of 0.50–2.5 m width. Figure 3 shows geological L-section along main dam axis.

The fault zone includes the shearing (Sz-5), by virtue of its disposition and continuity, extends beyond the coffer dam located about 80 m upstream of the exposed section and opens into the water pool, created by the upstream coffer dam inspite of being directly connected to this water pool, created by the upstream coffer dam. Inspite of being directly connected to this water pool with a head difference

Fig. 2 Layout plan of main dam of Indira Sagar Project



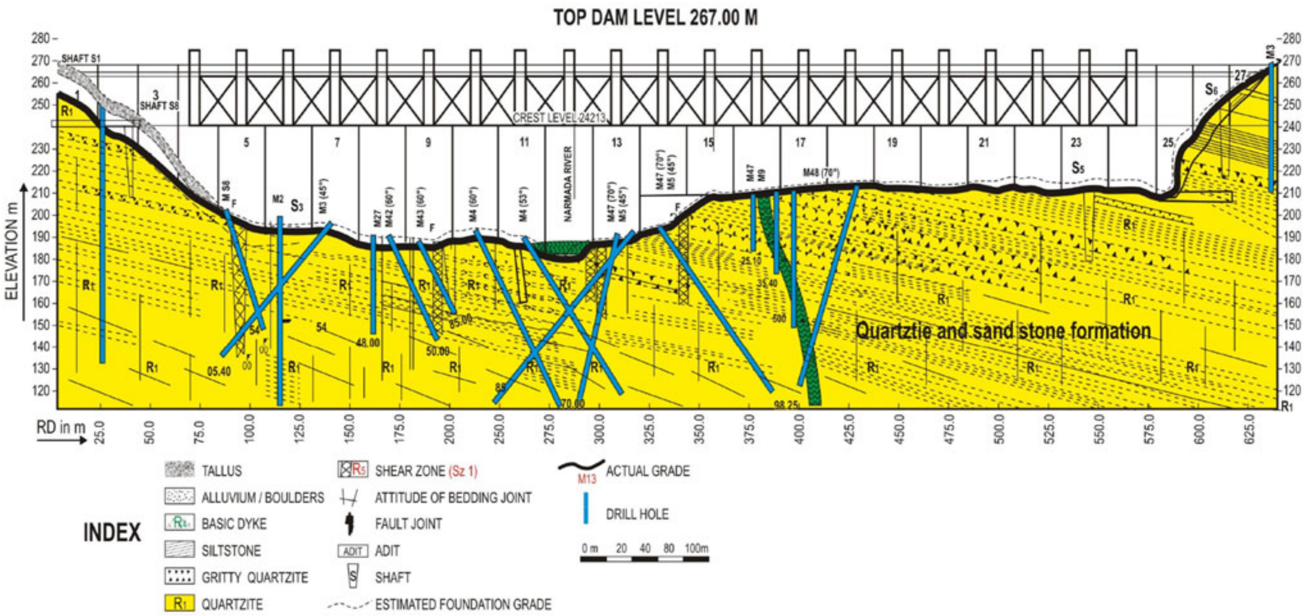


Fig. 3 Geological L-section along main dam axis showing shear zone and other discontinuities

Fig. 4 Treatment of fault zone

of about 18–20 m the exposed section is completely dry and points to the near impervious nature of the fault zone material. Permeability test carried out in the new test holes in the fault zone confirm this observation (<1 Lugeon). Except in the highly crushed zone/intrusion dyke, the fault zone material looks well compacted and was expected to have high in situ density in the range of 2.2–2.3 (Fig. 4). The shear strength parameters were high and grain porosity may not exceed 20 %. The material did not show any significant deterioration notwithstanding the fact that there has been water to a depth of 1–2 m standing on it for a considerable time.

Table 1 Pre-construction stages to determine detailed design parameters

1	Geological mapping.	Over 0.75 Sq. km on 1:1000 scale
2	Core drilling	Double tube barrel over 3,000 m Triple tube barrel over 500 m
3	Trenches	Three parallel trenches of +30 m. Six cross trenches of +15 m
4	Shafts	Six shafts of 9.5–18 m depth, 3.5 m dia.
5	Drifts	Four drifts of +16 to 23 m
6	Bore hole camera studies	In two drill holes to study cavitations in silt stone/bedding shear zones

Even though the strata for foundation of coffer dam looked positive, for further dam design and stability of coffer dam foundation, detailed geotechnical investigation were necessary to eliminate geological surprises. Due to lukewarm report about highly Crushed zone/Intrusion dyke the foundations of coffer dam needed additional treatment to make it water tight. Further site investigation were carried out as given in Table 1.

It may be observed that during pre-construction stages, to determine the detailed design parameters, following method of site investigations were performed.

This Data indicated that the sheared/crusted rock mass shall get consolidated if consolidation grouting is carried out and after completion of coffer dam, curtain grouting is also recommended, 1 m from the upstream face of coffer dam. Both these treatments were absolutely necessary in view of typical geotechnical problem faced.

Consolidation grouting in each block of the coffer dam for full area of foundation was carried out and also the curtain grouting as recommended 1 m from upstream face of coffer dam. This treatment made the coffer dam fully watertight.

Coffer Dam: Upstream and Downstream

The upstream coffer dam was necessitated to stop and divert the post monsoon river flow, to facilitate construction of main dam. This post monsoon flow ranging from 300 to 100 cumecs was required to be diverted from a diversion tunnel which was under construction through the left abutment, to discharge the flow back in the river on downstream side. A downstream coffer dam was necessary, to stop this diverted water through the diversion tunnel so that it should not come back to the seat of proposed main dam.

As the construction of diversion tunnel was getting delayed considerably, provision of six sluices in the body of upstream coffer dam was envisaged to pass the post monsoon flow of 300 cumecs during the construction of coffer dam which was scheduled to be completed in three working seasons. After completion of coffer dam, these sluices were to be plugged, so that post monsoon flow could be diverted through the diversion tunnel.

Concept

The work consists of design and construction of upstream and downstream coffer dam of Indira Sagar Project. This was the new concept of design used for first time in any country to construct a coffer dam using 5 tons pre-cast blocks as shuttering on both faces upstream and downstream side of a dam, and filling the enclosure with boulders and stonecrete them under water. All work of coffer dam has been done underground and under water up to RL 193.5 m.

The maximum height of upstream coffer dam was 24, 12 m under water and 12 m above water. The lowest foundation level in river bed was ±180.60 m. For under water portion 5 tons hollow pre-cast c.c. blocks were casted and placed on the upstream and downstream face of dam, as a shuttering in a cell of 15 m length with the help of divers. Before placing the blocks, river bed was leveled by using blasting under water and with special technique, PC blocks were placed. Boulders were filled in the enclosure of a 15 m long cell and grout pipe with safety reinforcement were placed. The cell of 15 m was thoroughly caulked from outside, so that river flow does not have any effect in the cell. A colloidal grout, made out of sand, cement water and super plasticiser was pumped through the grout pipe at bottom and level of grout slowly built up from down upward. Thus colloidal grout (colcrete) replacing the water in the voids of the boulder and converting the boulder mass into concrete. This under water work was carried out up to 193.50 m level. About six pre-fabricated construction sluices of $2 \times 3\ m^2$ were placed to take care of post monsoon discharges up to 300 cumecs as the diversion tunnel was under construction and was not ready.

Stone masonry was constructed above water level over the under water works from RL 193.5 m and raised up to RL 204.50 m.

Design

Height of upstream coffer dam was 24 m with the lowest foundation level kept at 180.6 and top of coffer dam as RL 204.5 m. It has been designed on the principle of gravity dam, duly checked, for stability in various condition like Reservoir empty and Reservoir full etc. with following parameters:
Length of Coffer Dam.

Upstream	220 m
Downstream	110 m
Unit weight of masonry/stonecrete	23.54 kN/m^3
Unit weight of water	9.81 kN/m^3
Angle of Internal friction for foundation bed	50°
Cohesion for foundation bed	147.09 kN/m^3
Tensile strength of steel water face 147,123.90 kPa	

Zoning of Material

Coffer Dam Above water level, Random rubble masonry with a compressive strength of 10.5 N/mm^2 (cement mortar 1:4 approximate) with selected stone placed in upstream and downstream.

Under water stonecrete masonry in 1:2 and 1:3 colcrete with P.C.C. Blocks on upstream and downstream.

PCC Blocks in M-15 Grade Concrete

Coping Concrete of M20 grade (c.c. 1:1.5:3) with 20 mm graded metal.

It may be observed that for underwater work, PCC blocks were kept on upstream and downstream section of the coffer dam and they were considered as a homogenous part of the full section. Dam L-section and cross section can be seen vide Figs. 5 and 6.

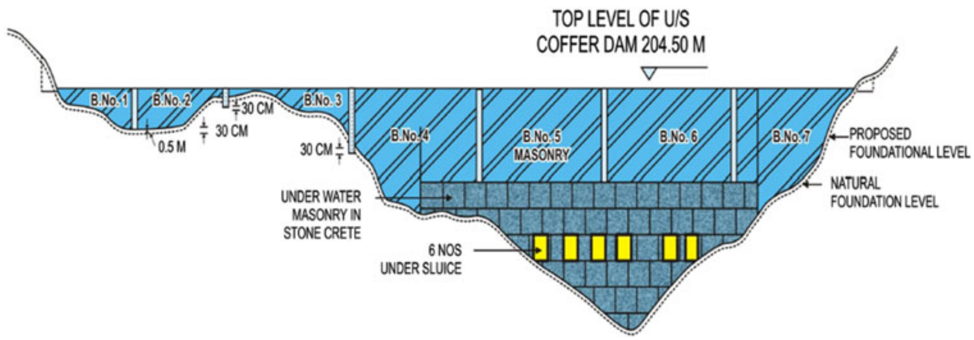


Fig. 5 L-section of upstream coffer dam

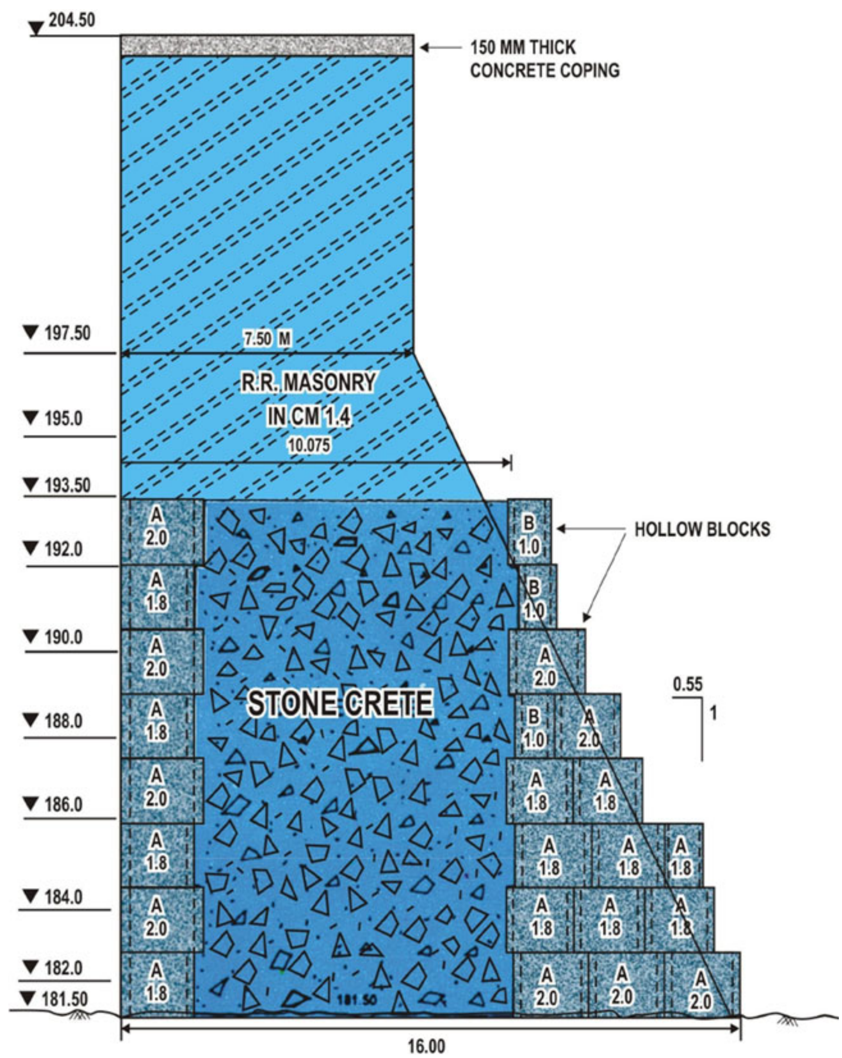


Fig. 6 Typical X-section of upstream coffer dam

Construction Sequence of Coffer Dam Upstream

Following is the Construction sequence.

Casting Yard for Pre-Cast Hollow Blocks

A casting yard having all facilities to cast hollow blocks under controlled conditions was made on left bank. The pre-cast/hollow blocks of size of 1.5 × 1.5 × 1.5 m^3 were casted in this yard. Suitable storage for form work and construction materials like stone grit, sand, water curing tank etc. were arranged on this platform to keep adequate stocks at site. The yard was equipped with form vibrators etc. and the travelling gentry with 10 T capacity. One electric hoist was fitted at this platform. The blocks were handled and loaded in flat bottom trucks by said gantry to carry it up to working platform on left flank from where, finally they were taken for construction using crawler mounted cranes at site.

Pre-Cast Hollow Blocks

It was proposed to use hollow-precast blocks in the upstream and downstream of the dam profile under water.

This enclosure were termed as stonecrete cell. At a time, 15 m length of coffer dam was undertaken in hand. Selected rubbles were filled within this enclosure along with colgrout pipes to carry out under water work. These colgrout pipe of 80 mm ϕ were kept in a grid of 3 m and individual pipes were surrounded with a circular coil made of 6 mm, Tor to protect them during boulder filling. These blocks in addition to forming the enclosure have helped to stop flow of water within the enclosure as well as in voids of the rubbles.

The shape of the blocks on upstream and down streams faces of the coffer dam were nearly confirming to the designed profile.

To provide necessary interlocking amongst the blocks, male and female grooves were provided in each block. The blocks of special dimensions for maintaining uniform level of courses were casted as per requirement. The necessary shear keys and lifting hooks were provided in each block. The blocks were casted in the rigid steel forms so as to ensure uniform dimensions and minimum tolerances. The blocks were cast in advance and stacked in the casting yard.

Preparation of Foundation

The left and right flanks which were much above water level of river were excavated to reach sound rock level to accommodate the length of coffer dam. Right flank in particular, was braided with clayey gaougey shear seam of varying thickness from 2 to 6 m. On the left flank excavation bedding shears of 16–25 cm thickness confined to the silt stones were commonly seen. Few photographs of the shear seam and excavation are shown in Figs. 7 and 8.

Foundation preparation in the river portion comprised of removal of silt, debris, loose rock and leveling of bed rock by underwater blasting wherever necessary. This was done using expert divers.

In order that the precast blocks from the pattern masonry walls required to be raised in uniform courses, the precise soundings were taken and loose materials were removed from its underneath. The area was leveled using special sizes of the blocks, or executing under water concreting for leveling course. It was observed that foundation rock was undulated at places. Hence levels were taken at a grid of 2 m and drawn on graph sheet. The gap between the leveled foundation and underneath of the blocks were caulked to achieve reasonable water tight joint.

Fig. 7 Shear seams in foundation

Fig. 8 Shear seams in foundation

Launching of Blocks

After the river bed is cleared of loose materials and leveled to receive the first course of the blocks as described under preparation of foundation para above, the pre-cast hollow blocks were lifted from the working platform and carried by crane and lowered in position in the cell. Before lowering the PCC blocks, a steel frame made of 100 mm M.S. angle, is first lowered in place on 40 mm bed of stone chips and this frame is leveled horizontally on this bed. Expert divers had positioned the blocks at proper places either on the upstream or downstream of the enclosure as required, but within these steel frames which were leveled horizontally on the bed of 40 mm stone chips. Figures 9 and 10 shows Launching of Blocks.

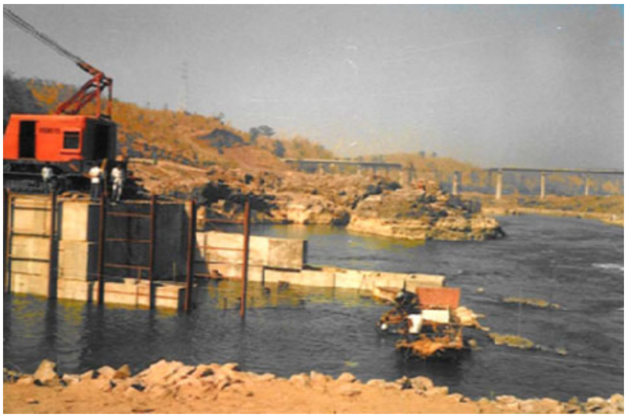


Fig. 9 Launching of blocks

Fig. 10 Launching of blocks

Each operation of block launching consisted of placement of blocks in the bottom course, to be followed by blocks in upper course. Till they were placed up to the level of 193.50 m. The launching of blocks are shown in photograph herewith.

Normally the blocks will be placed in required courses on up-stream and downstream sides of the upstream and downstream coffer dams. As the blocks are required to be in course it will be imperative to break the joints between the courses. Proper care was taken to break these joints in subsequent courses. As a matter of abundant precaution, the space in between the rows of blocks, will be filled with selected rubble near the blocks and around the pipes placed for colgrouting, so that the same does not get disturbed while filling up of rubble/stones in the cell.

Packing of Rubbles

After the blocks are carefully launched and erected on either side in courses and the space in between intersped with colcrete pipes, as stated above, rubble will be placed to fill-up the entire space between the rows of blocks in a 15 m cell. This rubble filling shall be done layer by layer in a systematic way using the large buckets with drop bottoms, handled by cranes.

Stonecreting Operations

The stonecrete process consists of making a grout of cement, sand and water in which cement has been so completely hydrated by high speed mechanical mixing, that the grout attains a colloidal form. This grout is stable and particularly fluent. It contains no chemical admixtures which might ultimately be harmful. When colloidal grout is poured in rubble aggregate the voids in the rubble filling are completely filled by penetration and the whole mass sets as a dense, solid concrete which is termed as "stonecrete".

Preparation of Colloidal Grout

The Colloidal grout was prepared in double drum colcrete mixer consisting of sand, cement and water in desired proportions to obtain colloidal grout. In colloidal mixer, the wetting of solid ingredients results from the shearing action which takes place in the specially designed impellers and matching casings of the colloidal mixers. The colloidal grout of specific gravity up to 1.8–2 is obtained using these high velocity mixers. Colloidal grout has enough fluidity to flow like grout and does not get separated when it comes in contract with water. It displaces water from the voids of stone/rubbles due to high specific gravity. Figures 11 and 12 shows preparation for colgrouting.

The double drum stonecrete mixer produces colloidal grout at the rate of 5–10 M^3/h of 1:2 mix or 6–12 M^3/h of 1:3 mix. When colloidal grout is stored without agitation in tanks after mixing, a little settlement is to be expected because sand invariably contains some oversize particles.

Fig. 11 Preparation for colgrouting

Fig. 12 Preparation for colgrouting

When it is pumped direct by the mixer to the work in normal practice, the oversize particles do not have time to settle out.

Placement of Colloidal Grout

Colloidal grout does not mix with water unless agitated with it.

The colloidal grout so prepared is pumped, through 80 mm ϕ pipes placed in the rubbles using special roto or colmono pumps. The grout will be pumped at the bottom of pre-packed stones under pressure and will be allowed to rise uniformly in the cell displacing all the water from the voids due to its gravity. Once the grout travels up to the top of the course, the colgrouting is stopped when it emerges out of boulders at a level of 193.50 m of working platform level. One such 'stonecrete cell' of 15 m length is shown in Fig. 13.

After completion of 15 m cell, crawler mounted cranes will be advanced to tackle the next cell of the coffer dam

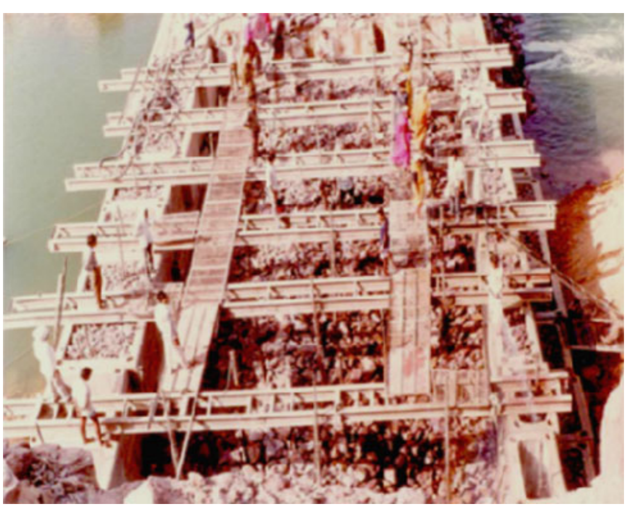


Fig. 13 Stonecrete cell 15 m (colloidal grouting)

till entire length 220 m length of upstream coffer dam is completed from one end. Construction sluices were left in the upstream coffer dam for diverting the water in final stages of closure of the coffer dam.

Construction Sluices

Pre fabricated M.S. sluice barrel were lowered in the central portion of upstream coffer dam keeping invert level at RL 186. In all, six sluices of 2×3 m^2 were installed. Rigid steel boxes were provided in the blocks for forming the approach tunnel for sluices. The construction sluices were installed under water with the help of the expert divers and is shown in Figs. 14 and 15.

Masonry Works Above Water Level

Construction of masonry in the flank blocks and above water level on stonecrete platform, up to top of coffer dam was done using conventional method of construction. The coffer dam above RL 193.5 was constructed in masonry up

Fig. 14 Fixing of construction sluice

Fig. 15 Lowering of sluice gate

Fig. 16 Masonry work over RCC blocks

to RL 204.5 m and is shown in photograph (Fig. 16). This was taken up immediately after the construction up to RL 193.5 under water using stonecrete. A coping 150 mm thick was laid at RL 204.5 m on the masonry using M-20 grade concrete.

Drilling and Grouting

Consolidation Grouting

Looking to the geology, it was recommended to adopt a grid of drill holes at 3 m c/c on both side, besides about 82 nos. of special grout holes were identified keeping in mind the location of various fault zones.

Depth of consolidation grout holes was 6 m in foundation rock.

Curtain Grouting

It was recommended to provide a single row of grout curtain, 1 m from the upstream face of the coffer dam, spacing of holes were kept as 3 m c/c. Depth of grout curtain holes in foundation rock was kept as 15 m.

This grout curtain was provided in stages of 5 m of drilling and grouting in descending order method.

It was observed that in consolidation grouting, intake of cement was 45 kg/m and in curtain grouting it was 26 kg/m.

Downstream Coffer Dam

The construction of downstream coffer dam was done using the same methodology as explained under. The top of downstream coffer dam was kept as 191.00 m i.e. 13.50 m below the upstream coffer dam top. The main purpose of this coffer dam was, not to allow the river water which was diverted through the diversion tunnel (back water) in the downstream of the river.

Figures 17 and 18 for completed upstream and downstream coffer dams.

Conclusion

Coffer Dam upstream and downstream, for the Indira Sagar Project had been successfully constructed and performed well, as a result the work of 92 m high I.S.P. main dam could be expeditiously carried out on mighty Narmada River.

The unique and innovative design, using 5 Tons Pre-cast hollow blocks with underwater stonecrete technique for the first time in the country, has successfully been used in Indira Sagar Project.

Case Study II: Anchored RCC Diaphragm Wall Coffer Dam for Bisalpur Dam Project in Rajasthan

Abstract Construction of dam foundation of Bisalpur masonry dam on river Banas was posing a problem due to

Fig. 17 Completed upstream coffer dam

Fig. 18 Completed downstream coffer dam

presence of medium and coarse sand in the river bed. In post monsoon period, though there remains very little surface flow in the river, but ipso-facto entire river flows through 10–12 m thick sand bed. Hence RCC diaphragm wall was conceived to play dual role i.e. to cut off the flow through the sand bed and also to act as coffer dam to divert the surface flow in the diversion channel. 60 cm thick RCC diaphragm wall has been constructed and used as cut off as well as coffer dams on upstream and downstream side of main dam.

Introduction

Bisalpur project across Banas river near Bisalpur village, 23 km from Deoli town in district Tonk, Rajasthan, is an irrigation cum drinking water supply project and consists of main masonry dam, water conductor and canal system. It is a masonry cum concrete dam having length of 574 m (Fig. 19). The maximum height of dam is 27.50 m. The dam abuts hillocks available on both flanks. The project was completed in 1992.

The river bed of Banas is filled up with medium coarse sand which is highly pervious and no work on the main structure of the dam was possible unless working area is rendered dry. Geological details are given under Geology head. Entire post monsoon flow in river passes through 10–12 m thick sand deposits and dam foundation on rock below sand bed was not possible due to heavy flow of water existing in the river bed through sand bed. Hence this concept of providing RCC diaphragm wall as a water barrier cropped in so that working area does not have flow of water during construction. The river bed made up of sand was mobile and gets eroded in high floods partially depending upon the intensity of the flood. The sand is deposited again, in receding flood. Designed peak flood at the dam site is estimated to be of the order of 12,000 cumecs.

Fig. 19 Bisalpur main dam on Banas river

Main Objective

To construct the main masonry dam, two problems were to be taken care of, which were going to be encountered during construction.

The first one was to provide two cut offs on upstream and downstream of the proposed main dam, through the entire depth of coarse sand, suitably anchored in the bed rock so that main working area becomes practically dry during foundation excavation and construction of main dam structure could commence.

The second objective was to provide a suitable temporary structure during the construction of the main dam which can divert the winter and summer discharges of river surface water flow away from the working area i.e. construction of upstream and downstream coffer dam which can divert the flow through the diversion channel on the right flank of the proposed dam site. Height of this coffer dam was envisaged to be 2 m above the average river bed.

The most economical and best solution to achieve both the objective i.e. cut off through the sand and coffer dam to divert the river flow away, was conceived as R.C.C. diaphragm wall suitably anchored in the base rock which can handle the hydrology of this river and dam site, during the flood as well as during the construction period from October to May of coming years.

Geology

Main Dam

Proposed Bisalpur dam site is located at about 250 m, upstream of the steep gorge, forming on the river Banas cutting through quartzite hill on both sides (Fig. 20). The abutments expose the Delhi quartzite and quartz mica-schist, underlying Aravali gneisses and schist. In the river section, the bed rock is marked by thick alluvial deposition consisting of medium to coarse sand in thickness of 10–12 m. The type of rock below this sand deposit is fairly impervious (water tight) having permeability of the order of 5–25 Lugeons. Few pre-construction bore-holes were done to know the permeability of sand and Rock strata. Details are indicated in Table 2.

Coffer Dam

Though the alignment of upstream and downstream coffer dam was based on the geological investigation done earlier in 1985 and 1986 but it was essential to carry out further geological investigation during the construction. Hence additional few bore holes on the proposed alignment of the coffer dams were under taken. On the upstream coffer dam the same results of previous investigations were confirmed i.e. the maximum depth of rock was found at 12.00 m depth from the river bed. But for downstream coffer dam, some geological surprises were awaiting the construction engineers. The river bed rock had suddenly dipped to a depth of 50 m, in the central portion of the downstream alignment and hence again a new alignment of downstream coffer dam was identified, based on the geological bore holes and it can be seen that for about 140 m length the alignment was shifted to 30 m towards the dam axis, to circumvent the sudden dip of rock. However in new alignment of downstream coffer dam maximum depth of rock was found as 28 m, depth from the river bed (Figs. 21, 22).

Design Philosophy of RCC Diaphragm Wall

Let us understand, design philosophy of the R.C.C. diaphragm which has been identified to serve two purposes, i.e. upstream and downstream cut off and the coffer dam to divert the river during construction period through the diversion channel.

Looking to the wide range of variation in the availability of rock in the river bed i.e. from 2 m depth to a maximum depth of 28 m, several alternate proposals were discussed and suggested but on economical and practical considerations two types of designs were identified, the first one was applicable to diaphragm wall having depth from 2 to 12 m and second one having depth from 12 to 28 m (Fig. 23).

Design of Upstream Coffer Dam—R.C.C. Diaphragm Wall up to 12 m Depth

It can be seen that critical stability condition of this R.C.C. diaphragm wall structure shall be subjected during the construction period, when upstream side of upstream coffer

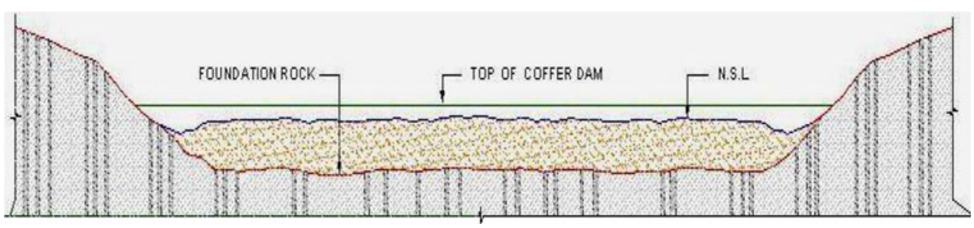


Fig. 20 Geological section of Bisalpur dam foundation

Table 2 Permeability of sand and rock strata details

Bore hole 1	Depth of Bore hole (m) 2	Type and depth of strata 3	Core recovery (%) 4	Permeability 5
1	0–10	Sand	–	>10^{-3} cm/s
	10–15	Rock Aravalli gneis	88	15 Lugeon
	15–21	Rock Aravalli gneis	94	12 Lugeon
2	0–11	Sand	–	>10^{-3} cm/s
	11–14	Rock Aravalli gneis	85	21 Lugeon
	14–19	Rock Aravalli gneis	93	18 Lugeon
3	0–12	Sand	–	>10^{-3} cm/s
	12–14	Rock Quartz mica schist	82	23 Lugeon
	14–18	Rock Quartz mica schist	89	17 Lugeon
	18–22	Rock Aravalli gneis	94	11 Lugeon
	22–26	Rock Aravalli gneis	98	5 Lugeon

1.3 Lugeon = 10^{-5} cm/s

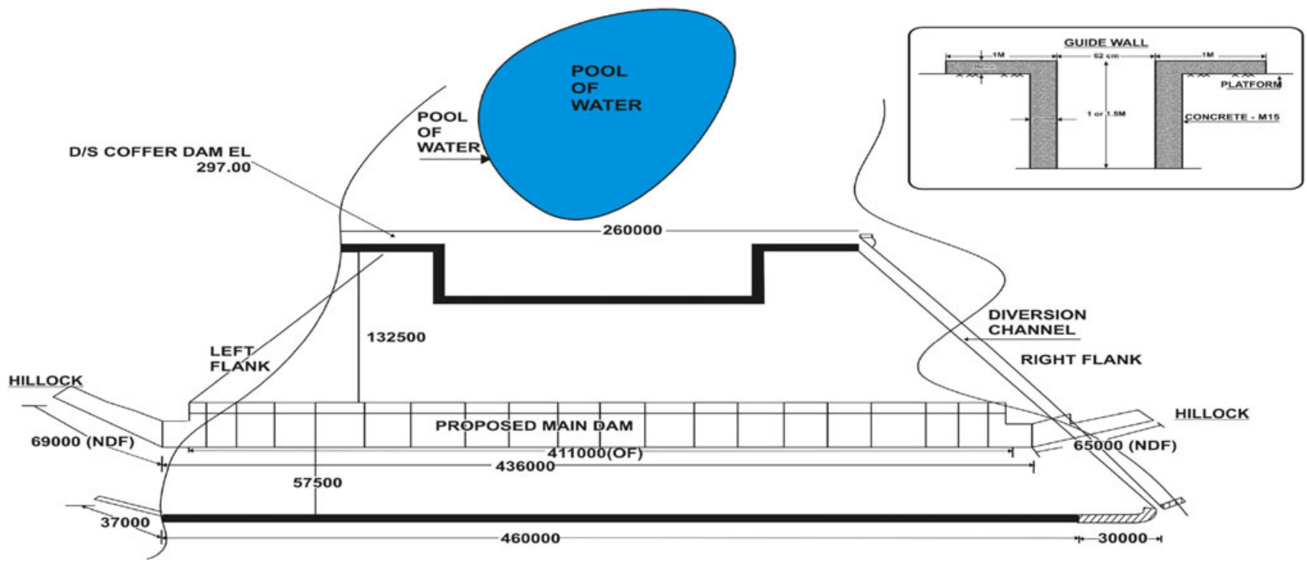


Fig. 21 Layout plan of Bissalpur dam

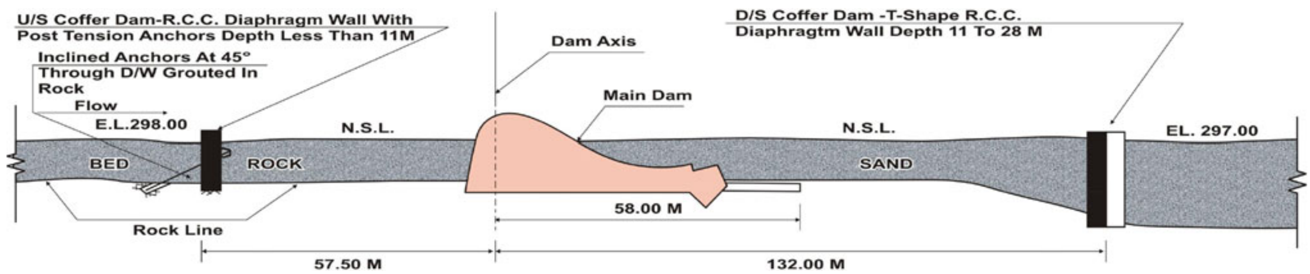


Fig. 22 Cross section through coffer dam and main dam

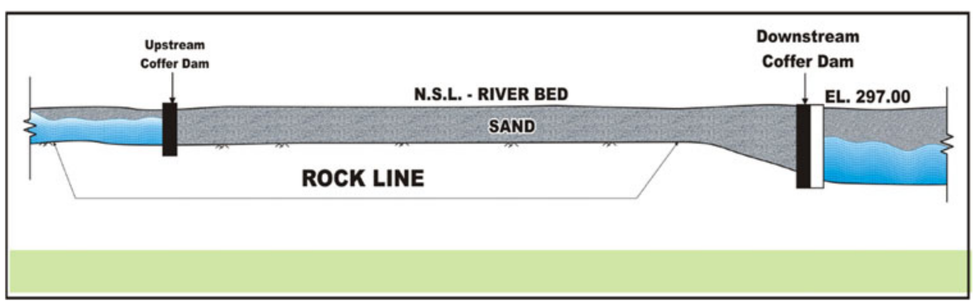


Fig. 23 RCC diaphragm wall coffer dam upstream and downstream

dam shall have the full head of water up to top of coffer dam plus the surcharge of submerged weight of sand and on downstream side there is nothing to support this wall as the excavation of main dam shall commence. Hence a RCC diaphragm wall of 60 cm thick with 60 kg reinforcement steel and in M20 concrete, was found satisfactory (Fig. 24).

But another important aspect to be considered was that, during the flood time after construction of diaphragm wall, there was a possibility that RCC diaphragm wall may get tilted on the downstream side, due to active earth pressure and velocity of water, and sand bed getting scoured on the downstream up to the rock level. This important design aspect was overcome by providing a suitable post tension anchor of 135 Tons from 3.6 m from top of coffer dam at 45° at 2.5 m c/c in the entire length of diaphragm wall where depth of rock was available up to 12 m depth. This anchor was to be drilled and grouted at least 8.5 m in rock i.e. it involved a drilling at 45° through the sand strata of 12 m depth, then to drill further 8.5 m in rock which consists of quartz mica schist.

Design of Downstream Coffer Dam—R.C.C. Diaphragm Wall with 12–28 m Depth

RCC diaphragm wall was required to be provided in the reaches where rock level was from 12 to 28 m depth. It

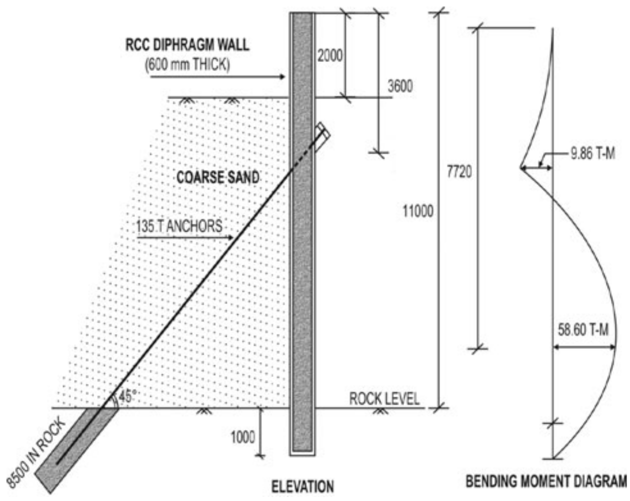


Fig. 24 Cross section of upstream coffer dam

may be observed that post tension Anchors were not suitable for RCC diaphragm walls having more than 12 m depth because of stability reasons and design requirement of 1 m thick diaphragm wall which was not practical to construct due to non-availability of proper equipment and on economy consideration.

RCC diaphragm wall having more than 12 m depth, has been designed on the principle of counter fort retaining wall. The design of this diaphragm wall is differing from the earlier one in two aspects due to its deeper depth.

During flood time, after Construction of diaphragm wall it shall be subjected to the load exerted by surcharge of sand and water. Due to deeper depth anchors are not viable, hence T shape diaphragm wall are provided on the principle of counter fort retaining wall.

To overcome the turning moment vertical anchors of 170 T were provided at the cross arm of T-Section at 2.65 m from front face of diaphragm wall (Fig. 25).

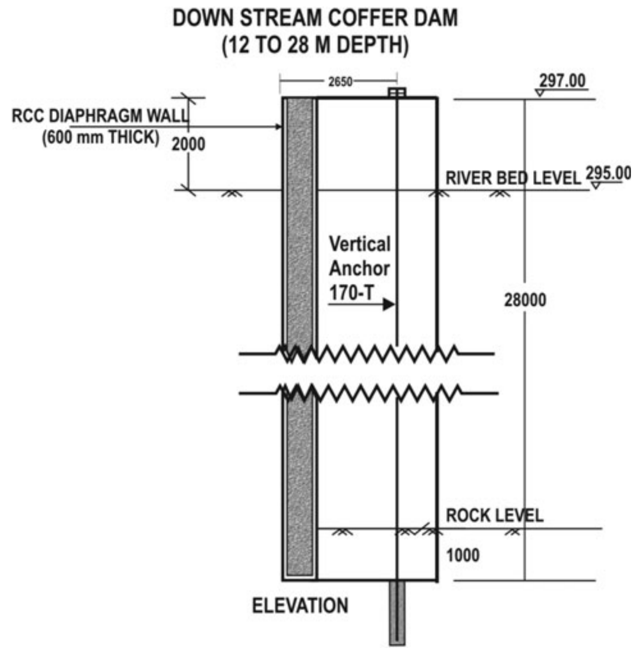


Fig. 25 Downstream coffer dam (12–28 m depth)

Construction Sequence for RCC Diaphragm Wall and Anchors

RCC Diaphragm Wall

Following is the construction sequence:

Construction of Platform

A platform using selected soil is constructed along the alignment of diaphragm wall for the movement of cranes, vehicles etc. The width of platform is as per requirement but generally kept as 20 m. The top of platform is kept at least 1.5 m above the water table in the area.

Panelling and Construction of Guide Walls

The total linear length of diaphragm wall is generally kept as 2–6 m and in this project it is 5 m length. A guide wall in M15 Concrete is constructed along the alignment. This guide wall (Fig. 26a, b) prevents subsidence of trench at the surface and guides the grab for trenching of the panel.

Trenching and Chiselling of Panels

Generally, excavation is carried out with an earth cutting grab, mechanical or hydraulic, handled by a crawler mounted crane or by reverse or direct mud circulating drilling machine mounted on a traversing trolley, travelling on rail tracks. As the excavation advances the trench is filled up with bentonite slurry of correct consistency and hydration to provide support to the excavated faces. The level of slurry in the trench is maintained by supplying fresh slurry from the slurry storage tanks. When hard strata or obstruction due to boulders is encountered during excavation, a heavy fabricated chisel is used to break through the rock or boulders. The grab is once again used to remove fragmented rock pieces and thus trench in a panel is taken up to designed depth which is generally 0.6–1 m in rock (Figs. 27, 28).

Lowering of Stop End Pipes

Fabricated tubes confirming to the thickness of the wall known as stop end pipes is lowered one each at the end of the panel trench up to the bottom of the trench with the

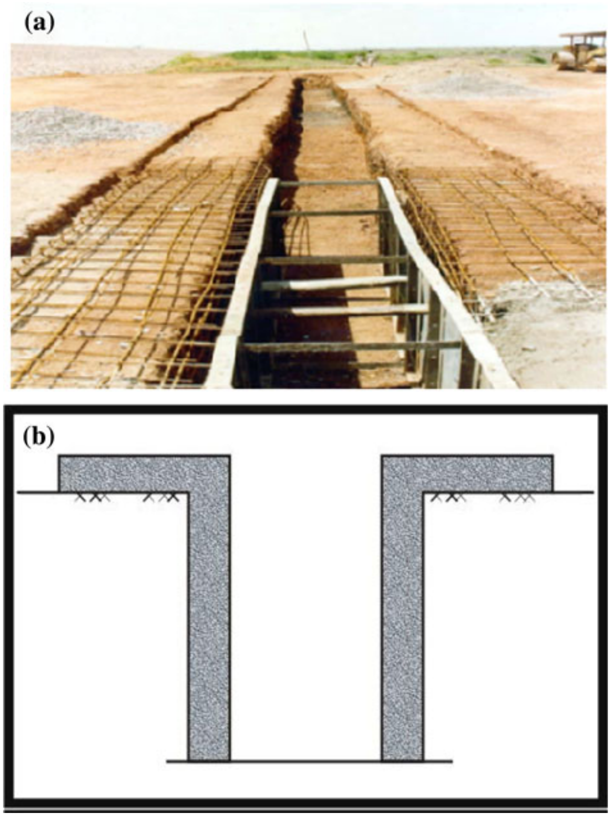


Fig. 26 a Guide wall, b guide wall cross section

Fig. 27 Trenching and chiselling of panels

Fig. 28 Trenching and chiselling of panels

Fig. 29 Lowering of reinforcement in panel

help of crawler mounted crane. The purpose of insertion of these tubes is to provide smooth semi-circular ends to the panels after concreting, as also to confine the spread of concrete only to designated dimensions of the panel. The convex surface of the panel end after retraction of the stop end pipes after concreting of panel also helps in guiding the tool for excavation of adjoining panel. The joint between the two panels shall be kept imperfect shapes other circular for stop end pipes are also in use to provide different shape to joints between the panels.

Lowering of Reinforcement Cage

A properly fabricated reinforcement cage generally welded at all joints is then lowered in the trench with the help of the crane (Fig. 29). Many times when the cage is more than 10–12 m length, the cage is made in more than one piece with adequate length of the longitudinal bars for proper lapping and insitu welding. 80 mm dia. m.s. pipes are left @ 3 m for grouting foundation rock and joints of diaphragm wall and foundation rock. Finally after insertion to the proper depth it is suspended on the guide walls. All inserts, cut outs in final wall and dowel bars for future connection of slabs to walls are fabricated with the cage.

Fig. 30 Lowering of tremie pipes

Lowering of Tremie Pipes

Tremie pipe are made up in section 1.2–1.5 m length, for concreting are lowered through the cage, at pre-determined position with the help of crane (Fig. 30). Generally two tremies are used, but if the panel length is 2–3 m single tremie is used. Sometimes due to particular shape of the panel 3 tremie pipes are also used. Tremie pipes are then fitted with hoppers to receive the concrete in batches.

Mixing, Conveying and Pouring of Concrete in Panels

Concrete is mixed in a central batching plant and transported to the site of excavated panel using tippers, fitted with special chute or if the lead is more and travel time is also longer the transportation is done in transit mixers. Generally M-20 or M-25 Grade of concrete with 150–200 mm slump is being used. The coarse aggregate of 20 mm and down size is being used for better workability.

The surging concrete in hopper, drives out all the bentonite slurry from the tremie pipe due to its higher density from the bottom of pipe, and finally spreads at the bottom of the trench. After deposition of concrete of a batch at the bottom of the trench there is no bentonite slurry left in the pipe and next load of concrete shall not come in contact with bentonite slurry in the pipe. The fresh batch of concrete shall push the earlier laid batch upwards if the bottom of the tremie pipe is kept embedded in the spread of earlier concrete of first load at the bottom of the trench. The concrete mix is designed to be fluid and cohesive. The w:c ratio is generally kept as 0.45–0.55 and slump more than 150 mm.

Gradually the level of the concrete rises at the bottom of the trench. The displaced bentonie is pumped away to a collection pit. As the concrete level rises, the tremie pipe in pieces are withdrawn section by section (Figs. 31, 32, 33).

Fig. 31 Preparation of concrete

Fig. 32 Pouring of concrete in diaphragm wall panel

Jacking of Stop End Pipes

The stop end pipes are withdrawn once the concrete starts setting, using hydraulic jacks and hydraulic power pack slowly. A special jacking arrangement is required to pull out stop end pipes without disturbing concrete resting against it. At any given point of time the bottom of the tremie pipe is ensured to be embedded in surrounding concrete by at least 1–2 m to avoid contamination of concrete at the interface of bentonite slurry and concrete. Only the first load comes in contact with slurry. The same interface keeps on raising with advancing concrete and the concreting of panel is thus completed. Generally this contaminated concrete is made to flow out the final stages of concreting or chipped out later on to expose the fresh concrete (Fig. 34).

Sequencing of Panels

Once the first panel is completed, then either the alternate panel method is adopted for the next operation or

Fig. 33 Pouring of concrete in diaphragm wall panel

Fig. 34 Jacking of stop end pipes

consecutive panel method is used. If the sequence of construction of panel to be followed is alternative panel method, then in that case first alternative panels are constructed, known as 'primary panels' using two stop end tubes. After completion of these primary panels for certain length of wall, the panel left out between two panels, known as 'secondary panel', are constructed. No stop end pipe is required for secondary panels since the ends of secondary panel is already defined by convex surface of concrete of primary panels on either side of it, thus giving a continuous wall. The concrete of secondary panels flows into the convex ends of the primary panels to give joint between primary and secondary panel a perfect half round shape.

Another sequence of construction equally popular is know as, 'consecutive panel method' requiring one stop end pipe only at the next end of the consecutive panel. However the next consecutive panel is not started earlier than 48 h of concreting of previous panel, to allow enough setting time and strength to the concrete of earlier casted panel to prevent it from damage.

TAM Grouting of Panel Joints

It is necessary to grout from the upstream side of the panel joint to make the joint absolutely leak proof. This calls for tube-A-machette grouting. Here one or two holes are drilled on the upstream of the panel joint, a tube-A-machette is inserted and sheath grouting is performed. After 7 days of sheath grouting, individual rubber sleeves are grouted with cement bentonite grout with $<5\ \%$ bleeding. A measured quantity of grout is required to be pumped through individual sleeve. Due to this grouting the panel joint becomes absolutely water tight (Fig. 35).

Grouting of Foundation Rock and Joints Between Diaphragm Wall Bottom and Rock

It is essential to leave 80 mm dia. m.s. Pipe in diaphragm wall to achieve two objectives (a) to grout the joints between RCC diaphragm wall and rock (b) to grout the rock below diaphragm wall. For this purpose 6 m drill holes are done in the rock through 80 mm m.s. pipe, left in the diaphragm wall and after washing and taking permeability test, it is grouted as per IS-6066.

Post-Tension Anchoring

The most difficult and tricky part was to provide the post tension anchors of 135 T and 170 Tons at 45° inclination at every 2.5 m length. After 28 days of concreting of a panel of 5 m, the process to provide 135 and 170 Tons Anchors were started (Fig. 36).

The first step was to drill at 45° inclination a suitable dia. hole through the RCC panel in the sand up to the rock and encase the hole with 100 mm M.S. casing pipe, it was very essential to case the hole up to rock, because of sandy strata and that too inclined at 45′ from horizontal. This feat was performed through indigenous rotary drill machines and bentonite slurry was used for circulation while drilling.

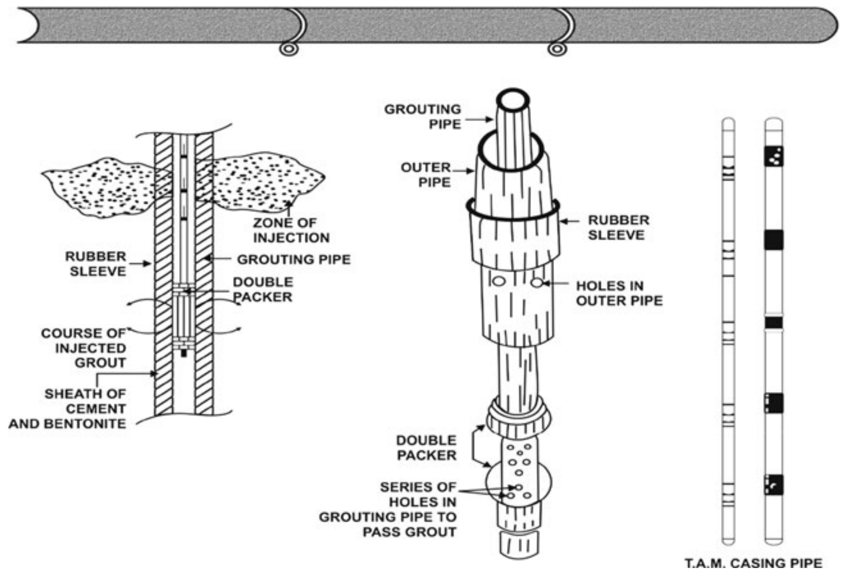
The second step was to drill 75 mm dia. hole in rock up to 8.5 m depth at 45° through the already installed 100 mm casing pipe. This drilling in rock was performed by using percussive drilling rigs powered by compressed Air at 686.58 kPa.

The third step, was to clean the entire stem of drill hole by alternate jet of water and compressed air.

The fourth step was to prepare a Tendon, made out of 9 nos. of 12.7 mm dia. high tensile strand wire. This tendon was prepared in such a way, that, it provides enough convergent and divergent points in tendon length, to be anchored in the rock.

The fifth step, is to insert this tendon in the drilled hole in rock, through the diaphragm wall and the 100 mm

Fig. 35 TAM grouting of panel joints



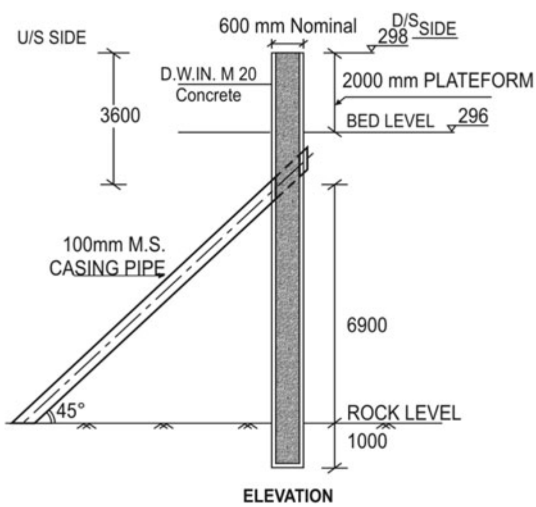


Fig. 36 Upstream coffer dam with anchor

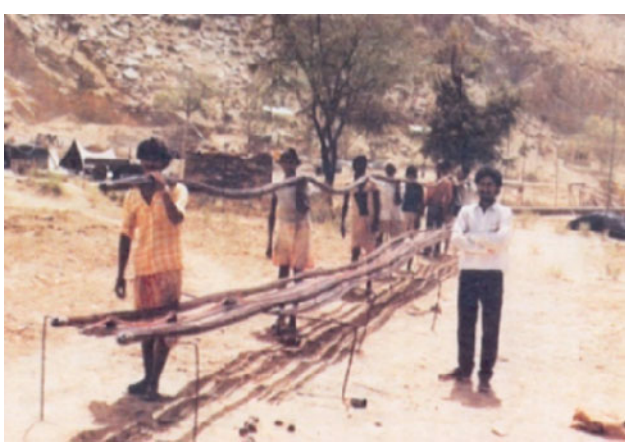


Fig. 37 Reinforcement Tendon

casing pipe. There is a plastic pipe of 12 mm inner diameter, which also goes along with the Tendon, up to the bottom of drilled hole. The sixth step is to pump the measured quantity of grout of 1:2 consistency cement to water through the 12 mm plastic pipe. The grout quantity is so measured, which is enough to fill the entire length of hole in the rock. The grout is allowed to set for minimum 2 weeks before post tensioning of this anchor begins (Figs. 37, 38, 39).

The sixth step is to stress this tendon with the help of a special hydraulic jack. The individual strand is stressed one by one and is locked at the upstream surface of the RCC diaphragm wall by special conical locking device. Like wise anchors at every 2.5 m c/c was provided in the entire length of the upstream and downstream RCC diaphragm wall. Further vertical anchors of 170 Tons were provided at the junction of T beam.

Thus the work of coffer dam cum cut off having a total length of 680 m (420 m upstream + 260 m downstream) for Bisalpur dam was completed in a record period of 12 working months. During foundation excavation of main dam between the two diaphragm walls it was observed that maximum seepage was <2 cusecs, due to which construction of main dam could be completed in 3 years.

Instrumentation

Instrumentation for measuring drop of head across diaphragm wall, following arrangements were made.

Four Piezometer lines were installed in the river bed portion @ 100 m c/c on the upstream of upstream diaphragm wall and downstream of downstream diaphragm wall.

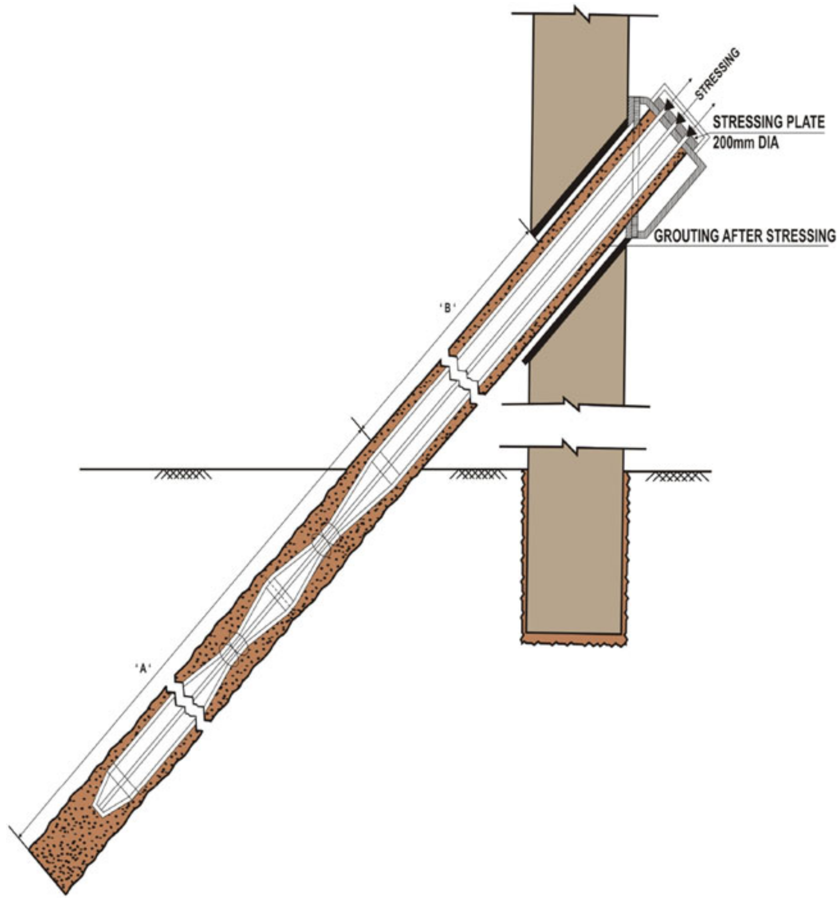


Fig. 38 Completed anchor through diaphragm wall

Fig. 39 Post tensioning of anchors

In each line about 6 Piezometers were installed. Two Piezometers on upstream of diaphragm wall and four on the downstream of downstream diaphragm wall.

Type of Piezometers installed were porous tube type.

Pre and post diaphragm wall installation readings were taken regularly.

Based on these Piezometer readings pre and post mean hydraulic gradient were drawn.

It was observed that drop of head across the diaphragm wall at 90 m distance is more than 90 %, practically in the entire length of diaphragm wall coffer Dam (Fig. 40).

Salient Features of Coffer Dam RCC Diaphragm Wall

Total length of coffer dam diaphragm wall.

Upstream	420 m
Downstream	260 m
Total quantity of 60 cm thick RCC diaphragm wall	12,000 m^2
Reinforcement steel used	790 mt
Cement consumed	4,230 mt
Post Tension Anchors used 135 Tons	296 Nos
170 Tons	204 Nos
Total cost of work (1988 level)	Rs. 8.40 Crores
Completion period	12 months

Conclusions

Coffer dam for the Bisalpur dam had been successfully constructed and performed well during construction period for more than 5 years.

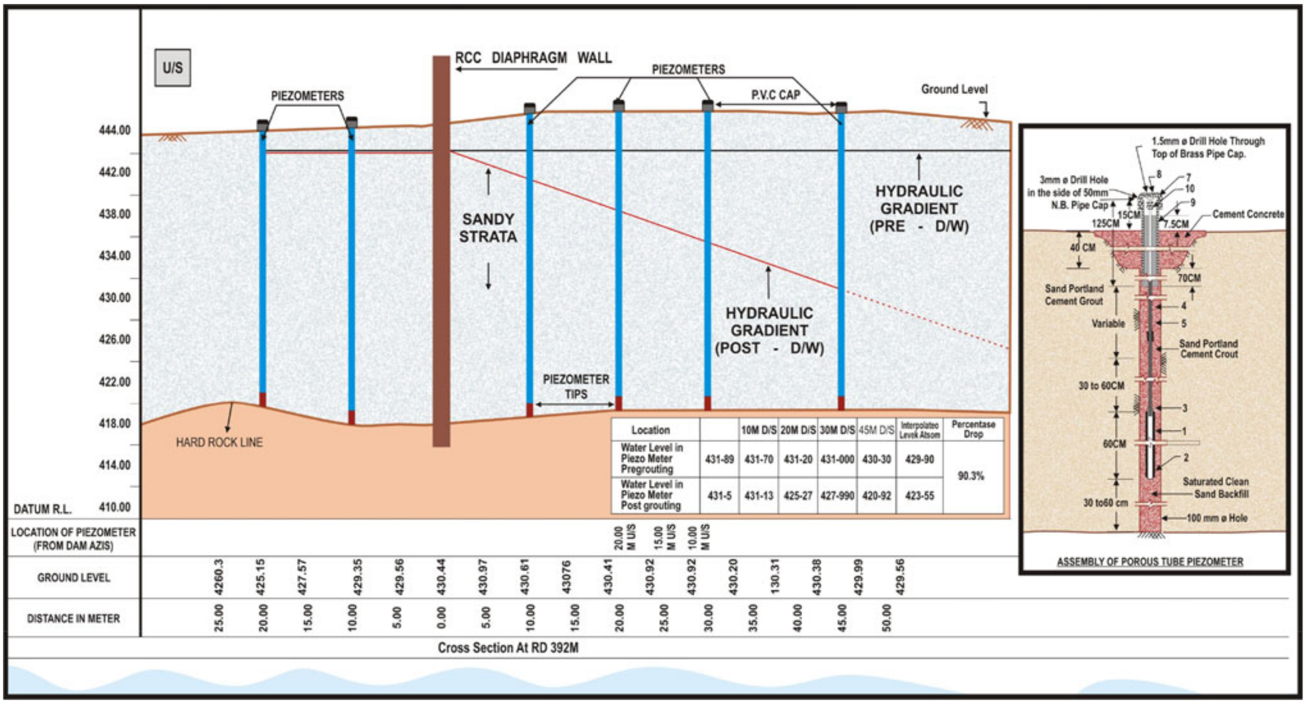


Fig. 40 Piezometer observation at Bisalpur dam

RCC Diaphragm Wall is successfully used as positive cut off for dams but can also be used as a coffer dam to divert, run off of the river and can be constructed in a very short period.

The total seepage through the Diaphragm wall covering 12000 SM area, was <2 cusecs.

Case Study III: Restoration and Rehabilitation of Old Pagara Masonry Dam by Grouting Technique

Abstract Pagara Masonry Dam was constructed about 94 years from now during the period 1911–1917. After construction, it has breached/damaged and rehabilitated three times. The third rehabilitation was done in present time which has called for a rational approach of dam safety, The strengthening of dam using guniting, pointing and grouting has been discussed in detail along with other deficiencies.

Concrete and masonry dams generally perform well. Many of the old structures, which have masonry block hearting and concrete/masonry facing have shown little sign of deterioration. Many of the old masonry and concrete dams do not have provision for relief of uplift pressure below the foundation but the stability of these older structure has not been seen as a problem as these were constructed with a fairly massive cross section. But this is not the case with all old dams which have deteriorated due to various causes like internal erosion, sliding, inadequate spillway capacity etc. The internal erosion results in serious failure of dams. To bring such deteriorated dams within the acceptable safety standard is a difficult task since every such problem with these concrete or masonry dam has tended to be specific to the site condition and can not be generalized for a solution.

Pagara dam across Asan river is a composite dam. It consists of 244 m long stone masonry non flow dam, 73 m long masonry waste weir and 1,439 m long earthen dam. The maximum height of dam is about 27 m above the deepest river bed level.

The spillway was fitted with 6 falling shutter type of crest gates of 12.19 × 1.83 m^2 which are in-operative and remain open it addition there is an open cut for a length of about 244 m on the right flank. There are two sluices located in the non-overflow section of the dam. One sluice fitted with a vertical gate of 0.91 × 1.83 m, size and another with a gate of 1.83 × 4.88 m size. Pagara dam is situated near village Joura, nearly 39 km from Morena town in north M.P. (Fig. 41).

History of Construction

This dam was constructed in a period of about 95 years from now, during 1911–1917 when knowledge and computational methods in structural design of dams and in hydrology were less accurate than now-a-days. A geological fault zone existed at the junction of earth dam and masonry dam which went unnoticed at the time of design

Fig. 41 Pagara dam

and construction of this dam. The open jointed sandstone formations on which the masonry dam foundation was rested, were not considered for uplift pressure. Drainage galleries to release uplift pressure were unheard of, in the old days. The cut off in the earthen dam had not been anchored to the impervious stratum. All these deficiencies resulted in considerable subsequent trouble and required thorough rehabilitation and upgradation of dam. The designed flood also needed reconsideration for spillway capacity. This needed a rational approach of dam safety including hydraulic studies, analysis of the stability and mechanical behavior of dam and foundation treatments, gate operation etc.

This dam has breached/damaged and rehabilitated three time since its construction in 1917. In following para they are discussed in brief.

First Breach/Damage and Rehabilitation: Year 1924–1927

The dam construction was completed in the year 1917. Floods of year 1917 and 1919 caused some damages to the dam. (Exact damage done is not on record). In 1924 floods, excessive leakage developed under the earth dam and the downstream slope of the dam slumped. A sand belt was detected in the shale foundations of the earthen dam and so a concrete cut off was provided at the heel of the earth dam and upstream face of earthen dam was paved with RCC slab up to MWL and top of earthen dam was raised by 0.6 m. With these repairs earthen dam was put in operation in 1927.

Second Breach and Rehabilitation: Year 1943–1948

In the floods of 1943 water levels rose to RL 202.69 m which was the TBL of non overflow section of masonry dam. Probably due to excessive uplift pressures in the foundations of masonry dam, masonry dam section failed and breached in about 30 m length. This breached portion was rebuilt with wider base width on the basis of stability considerations then prevailing. These stability considerations were of a very general nature. The dam was put back into operation in 1948.

Third Damages and Rehabilitation of Dam: Year 1988–1990

Gravity section of Pagara dam was constructed with lime mortar in 1917 and subsequently partly reconstructed after its breach in 1943 and thus it is more than 87 years old dam on date. The dam had also been once over topped. It has shown distress in form of wetness of slope, slushiness etc. Body of masonry dam was profusely leaking. Hence it was felt necessary in the year 1988 to accurately assess the present condition of dam in respect of hydrological and structural adequacy. In this paper this third time damage and rehabilitation has been discussed in details.

Identification of Causes of Present Distress (1988–1990)

Based on the Investigations carried out by Geological Survey of Indian and Central Soil and Materials Research Station, following conclusions were drawn.

Seepage through foundation of earth dam was not effectively controlled resulting the downstream slushy condition, bulging on the downstream slope of earthen dam.

Existence of geological fault zone at the junction of masonry and earth dam which was not treated at the time of construction.

The masonry dam was founded on sand stone formations which had open joints (Fig. 42). These joints had not been treated. There existed high permeability in the upper layer of foundation rock.

Distress due to ageing, which comprises of deterioration of construction materials (i.e. cracking, erosion and weathering etc.) and foundation failure. The failure may occur even under usual operating conditions due to increase in normal loads (i.e. silting, uplift and decrease in resistance i.e. by leakages crack propagation, weathering, erosion etc.) leading to complete failure of structure.

On the downstream face of masonry, very heavy spouts of water were coming out (Fig. 43). The entire downstream face was wet and most of the masonry joints were leaking. This further confirms that mortar from the joints has been washed out and lot of voids has been created in the body of the masonry. The body of the masonry dam was porous and safety of this structure was under danger zone.

Fig. 42 Open joints in dam foundation

Fig. 43 Seepage through masonry dam

There are problematic reaches in the earthen dam, predominant leakage locations were at/ch. 15, 17, 32, slushiness between Ch. 13–17, 22–23 and 26. 28 and 31–41.

Inadequate spillway capacity, which in turn may increase the surcharge height above the stipulated MWL overtopping the NOF portion, leading to complete failure of dam.

Vertical cracks in non overflow section at few locations.

Erosion at the heel of masonry dam due to clear overflow of spillway discharge.

Missing agreement of the statistical computation of that time to the present Indian standards.

Remedial Measures

To compute the 1,000 year flood as the normal design flood and ensure that the dam and the auxiliary structures of the spillway would not undergo any damage with the computed PMF. The encroachment in the free board and extent of damages that are likely to occur may be estimated. However hydrological rehabilitation is not being discussed in this paper. Only structural rehabilitation is being discussed here.

To ensure the structural adequacy, following remedial measures were recommended and carried out.

Masonry Dam

To gunite the entire up stream face of dam with wire mesh reinforcement. It was recommended that the reservoir be emptied and dam face thoroughly cleaned and the gunite applied in one continuous operation to ensure proper bonding.

Raking of all masonry joints on downstream face of dam and re-pointing them with good cement mortar.

Grouting the body of the dam and upper layers of the foundation to fill in all the voids created by leaching etc. using cement grout.

The test grout pattern is proposed in two rows 1 m apart and holes being spaced at 3 m c/c staggered. The first row of grout holes being one meter from the upstream face of dam. After test section, different reaches of masonry dam was proposed to be grouted accordingly. For layout of grout holes (Fig. 44).

Earth Dam

The existing earth dam section should be brought to a proper and uniform profile in the entire length.

To provide downstream toe loading with inverted filter in the identified seepage reaches.

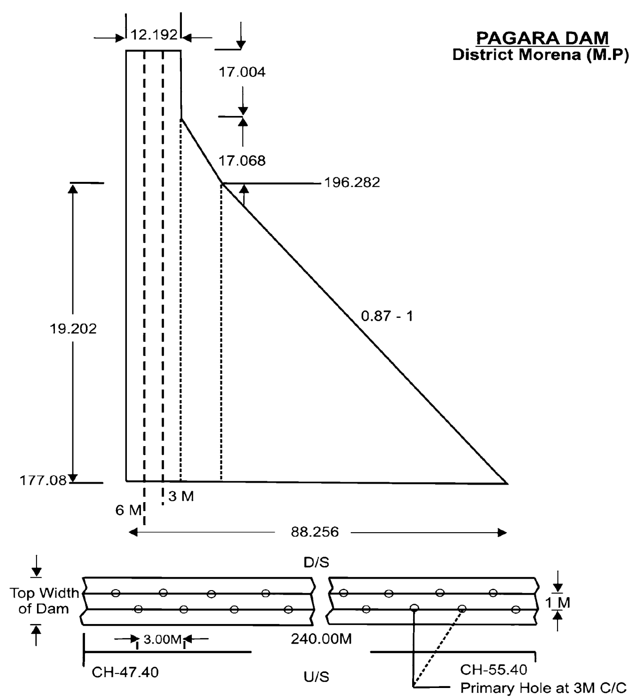


Fig. 44 Pattern of drilling and grouting

Relief wells shall be provided in the predominantly seepage zones.

The drainage ditch on the downstream side of earth dam had caved in and discontinuous in places. It was suggested that a continuous drainage trench $1 \times 1\ m^2$ may be excavated for the full length of the earth dam and back filled with broken rock and gravel to lead the seepage as well as rain water to the existing natural Nallah.

Structural Rehabilitation of Masonry Dam

Details of execution of guniting of upstream face, pointing of downstream face and grouting plan of main body of dam foundation are discussed below :-

Guniting of Upstream Face of Dam

In the first phase, guniting on the upstream face was started. The main component of guniting work as executed were as below.

Raking of the masonry joints on the upstream face:

Raking of joints has been done by lowering down the cradle platform (Jhula) from top of dam along with one skilled mason. With the help of chisel and hammer, mason has skillfully opened the joints and removed the old mortar about 50 mm in depth. With the help of 10 nos. of such set up, it was possible to tackle the entire face within a short time of 3 months.

Simultaneously with the help of jet of water and air, entire upstream surface of dam was cleaned along with the raked joints.

$50 \times 50\ mm^2$ hard drawn wire mesh of 10 gauge wire, has been fixed to the upstream face of dam with the help of long bolt cum nail of 250 mm length, in a such a way that top remains protruded about 40 mm from the masonry face to receive the wire mesh. The spacing of bolt cum nails were 1 m c/c both ways.

Wire mesh is tightened to the bolt cum nails with the help of $100 \times 100\ mm^2$ m s plate of 10 mm thickness with the help of nut.

A dry mix of cement and sand in 1:3 proportion has been applied with the help of Guniting machine all over the surface. Water cement ratio of 0.38 has been used. Thickness of gunite applied was 50 mm in average.

Curing for 28 days has been done by sprinking water throughout the surface of the dam (Fig. 45).

Raking and Pointing of Downstream Face of Dam

Raking of joints has been done by lowering down the cradle platform (Jhula) from top of masonry dam with a skilled mason. With the help of chisel and hammer old mortar from the joints between the masonry stones were taken out up to 50 mm depth.

The face of masonry and joints were washed and cleaned with the help of air and water jet.

When water is gushing out from the various joints, seepage water is localized through 20 mm G.I. pipe embedded in the masonry joint so that water from the nearby surface starts coming through the pipe and later on, when pointing is done in surrounding area, this pipe can be closed and seepage water brought under control.

Pointing has been done in conventional manner in 1:3 mortar.

Curing for 28 days has been done on the entire downstream face surface.

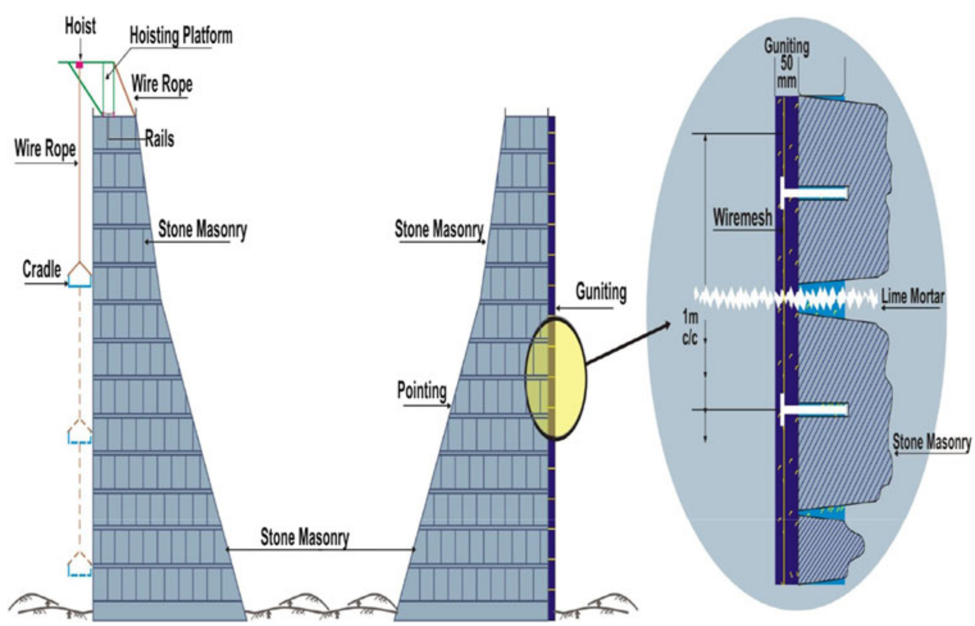


Fig. 45 Raking, pointing and guniting of upstream face

Grouting

Drilling and grouting operations has been executed as below (Fig. 46).

Drilling has been done by using Rotary cum percussive drilling rigs (Percussive method) with water flushing system. Though in some states Rotary (Diamond) drilling is recommended for drilling in weak and porous masonry dam, but it is very expensive and slow in progress.

First of all a casing of 80 mm diameter is fixed at the location of the hole, at least ½ m in the masonry by drilling a suitable dia. hole and fixing a m s casing by caulking and grouting it in the drill hole.

Drilling in masonry has been performed in descending order method in the front row first, where holes were marked at 3 m c/c in stages of 6–9 m. The first stage so drilled is washed with jet of air and water till all the drill cutting comes out.

Water test (cyclic) were taken in every stage. Permeability of a strata is measured in terms of Lugeon. Lugeon is a unit of water loss measured in litres, in a hole per meter at a water pressure of 980.83 kPa.

For grouting following machinery and equipment has been used.

High Speed Mixer

For mixing of cement with water, high speed mixers known as colloidal mixers are used (Fig. 47a, b). It is important to use these mixers so that grout does not bleed while grouting.

Double Drum Agitator

From colloidal mixer, grout is taken to double drum agitator, where measured quantity of grout is supplied to the grout pump. These are basically pedal mixer run by compressed air motor or electrically operated motor.

Electrically operated double cylinder pumps have been used to pump the grout in the hole. These pumps are reciprocating, piston type, capable of producing 4,904.13 kPa pressure and having a pumping capacity of 100–300 L/min liquid in the hole (Fig. 48a, b). From the agitator, grout comes to the pump and from pump it is injected in the hole through packer and grouting header fitted with a pressure gauge.

Grouting has been executed as per IS 6066 by starting with thin mix of 1:10 cement to water and thickening the same as the grouts intake increases and ending with 1:1 by weight.

The grout is left in the first stage to set after grouting for 24 h and then by re-drilling the set grout, second stage of 6–9 m depth is drilled and same procedure as described

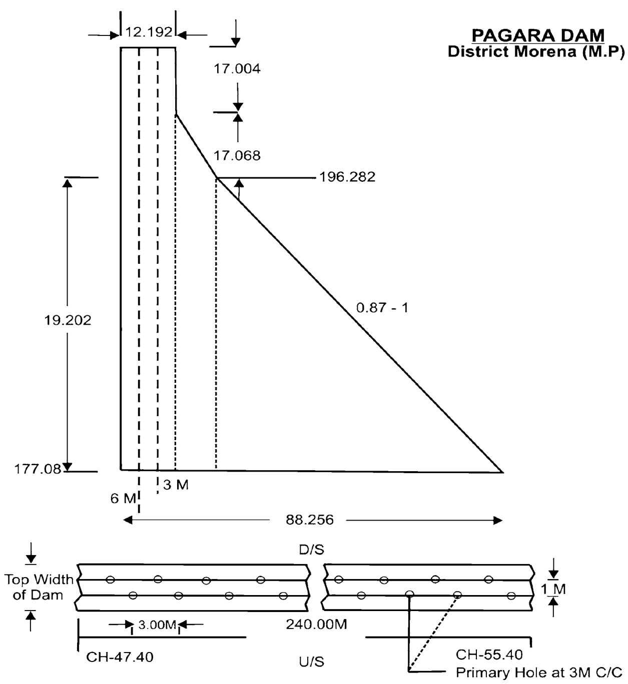


Fig. 46 Pattern of drilling and grouting

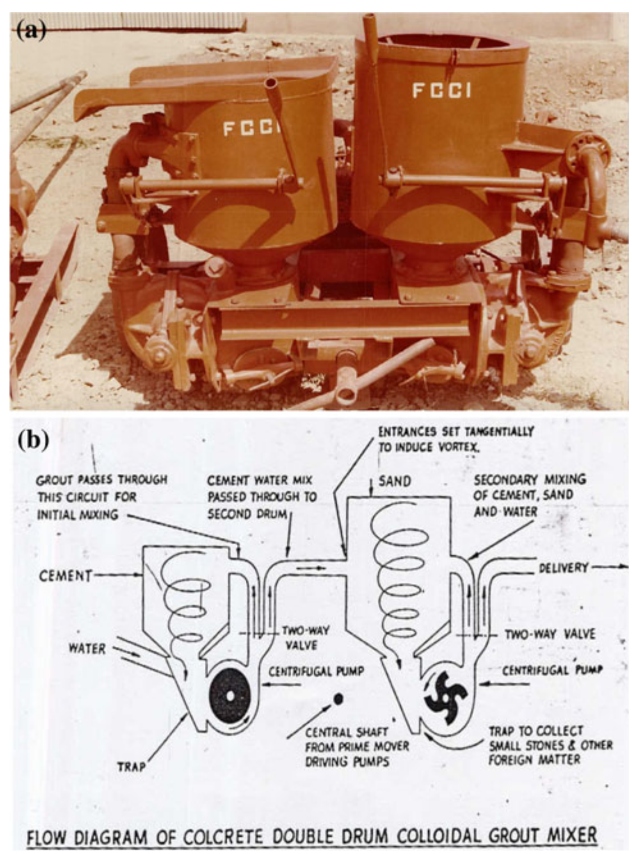


Fig. 47 **a** Colloidal mixer, **b** colloidal mixer

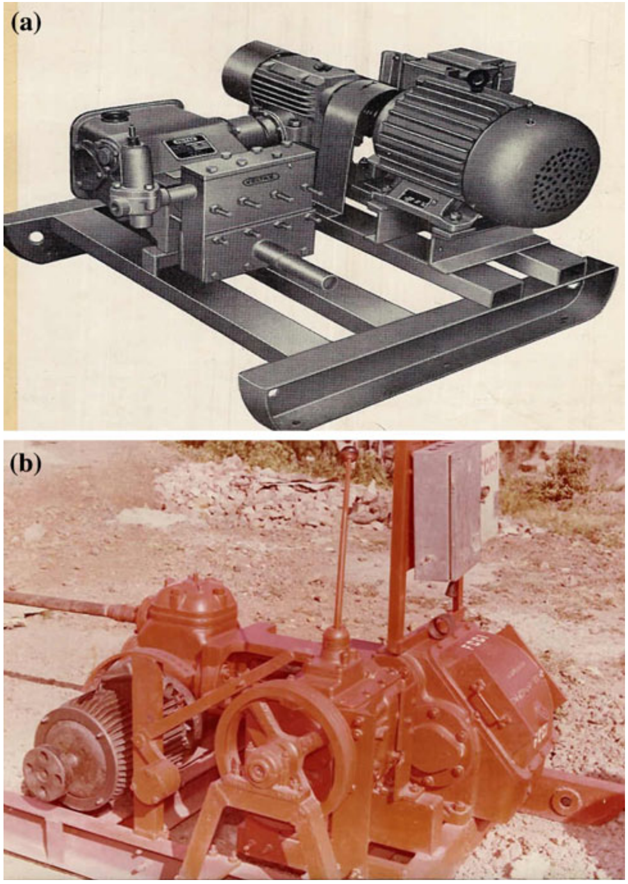


Fig. 48 **a** Grout pump, **b** grout pump

above is followed till a hole reaches to full depth. Drill holes were taken 3 m in rock foundation to grout the upper layer, which was highly permeable.

After grouting the holes of first row for about 30 m length, second row drilling and grouting is also performed simultaneously and the entire length of dam is completed.

Grouting Pressure

Maximum grouting pressure has been kept different in various stages as below:-

0–6 m	196.17 kPa
6–12 m	294.25 kPa
12–18 m	392.33 kPa
18–24 m and more	490.41 kPa

Near the sluices at Pagara dam, very loose masonry with lot of voids were observed which were taken care of by drilling and colgrouting 6 nos. drill holes, using sand cement grout with the help of colcrete mixture and colmono pumps (Fig. 49).

After completion of primary holes, 30 % of secondary holes were drilled in the centre of the two rows. These

Fig. 49 Sluice of Pagra dam

30 % holes were chosen near the holes, where grout intake was very high.

After secondary holes, a diamond drill hole at every 30 m c/c was drilled to find out the condition of masonry after grouting and water test taken.

It was found that good core recovery up to 97 % has been achieved and water test shows the result of <2–5 Lugeons. Following Fig. 50a, b shows condition of Pagara Dam before grouting and after grouting treatment (Fig. 51).

Salient Features and Data for Grouting of Pagara Dam

Length of masonry dam grouted	244 m
Average depth of grout curtain	18.00 m
Width of grout curtain	2.00 m
Volume of cement injected in entire dam	352.00 mt
Grout consumption per meter of drilling	50.00 kg
Quantity of percussive drilling executed	6934 m
Quantity of re-drilling executed	7197 m
Quantity of diamond drilling executed	144 m
Quantity of Guniting executed	2340 m^2
Quantity of pointing executed	9366 m^2
Pregrout permeability (average) more than	100 L
Post Grout permeability	2–5 L

Conclusions and Recommendation

With the use of cement grout, a very badly leaking masonry dam, having voids and high porosity was completely made water tight and structurally sound. It is essential to provide pointing on the upstream and downstream surface of dam before starting of grouting.

For grouting a badly leaking masonry dam, cement grout is sufficient. At few places wherever porosity is very high, colloidal grout of sand and cement can be used.

Fig. 50 **a** Before grouting treatment (Pagara dam), **b** before grouting treatment (Pagara dam)

Fig. 51 After grouting treatment (Pagara dam)

It is recommended to use high speed colloidal mixers to mix and prepare grout for the masonry dam. Conventional pedal mixers should be avoided.

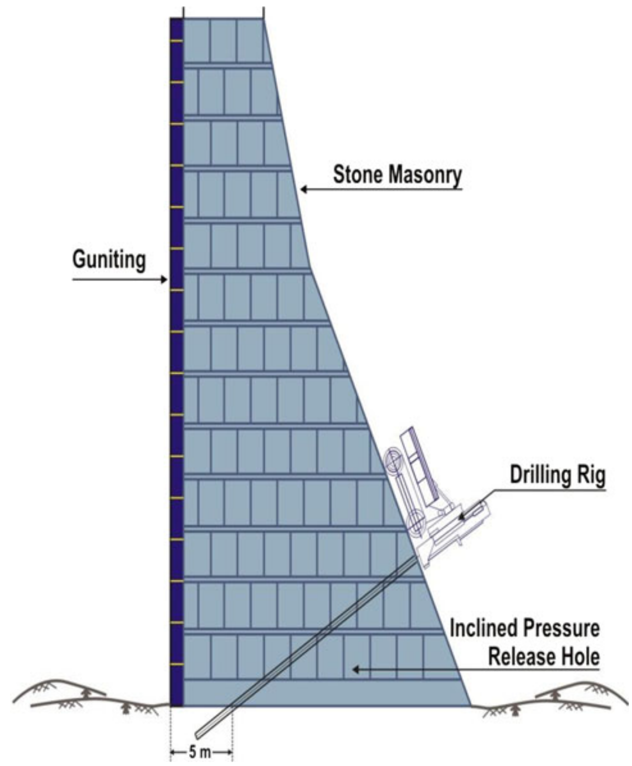


Fig. 52 Inclined pressure release hole

It is essential to provide inclined release holes in the masonry portion from downstream face of dam, to release the pore pressure because there is no foundation gallery. The holes must penetrate through body of the dam and 1 m into the foundation rock along a line about 4.57 m downstream from the toe of the dam. Spacing of hole may be 12.24 m centre to centre (Fig. 52).

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