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History of Tehri Rockfill Dam Design

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SYNOPSIS: A 260.5 m high rockfill dam is one of the main components of Tehri Dam Project which is under construction for the last over seventeen years. During this period detailed site investigations have been carried out and design of the dam is kept under constant review. A number of studies are conducted to provide feasible solutions to anticipated problems. The paper describes main design features of the dam mentioning the changes necessitated time to time with regard to these.

INTRODUCTION

The Tehri Project is an irrigation-power project of the State of Uttar Pradesh, in India, envisaging construction of: (1) A 260.5 m high rockfill dam across the Bhagirathi River to store 2,880,000 acre-ft of water; (2) underground power house, having installed capacity of 2000-2200 MW; and (3) spillway for design discharge capacity of 11,700 m³/s. The four diversion tunnels of 11.00 m diam each have already been constructed. The layout of project works is given in Fig. 1. The dam is to be placed at the site such that its foundation will be highly jointed and sheared phyllitic rocks and its abutment contact slopes be steep. The area is also seismically active. The construction of the project was taken up in 1970. However, the activities did not move fast mainly because of the paucity of funds. But this provided the design engineers enough time to tackle the complex problems of this gigantic structure. The problems examined are: (1) The location of the dam axis; (2) the dam section (3) design of coffer dam; (4) the approach to slope stability analysis under static and dynamic conditions; and (5) the foundations treatment.

GEOLOGY OF AREA

The rocks exposed in the area of the project site are phyllites of Chandpur-Series (Lavania, 1975, Gupta et al., 1979). The rock bands are of variable thicknesses have dips of 45°-60° in the downstream direction, and strike almost east-west. The river flow in this reach is north-south. On the basis of their physical condition, the argillaceous and arenaceous materials present, and the varying magnitude of tectonic deformations suffered by them, have been broadly graded as follows:

Phyllites of Grade I - This rock unit is predominantly arenaceous, massive in character, and distinctly jointed at phylliferous places that have lenticular elongated streaks of brown colored calcareous material.

Phyllites of Grade II - This rock is conspicuously banded due to the rapid alterations of arenaceous and argillaceous material. In physical quality and competence this unit is considered next to grade I phyllite.

Phyllites of Grade III - This unit is composed mainly of the argillaceous components with lesser amounts of arenaceous material. It carries quartz veins and is traversed by closely spaced foliation planes, cleavages, and joints. The unit is generally weathered. It is involved in minor folds and pucksers.

Sheared Phyllite - The sheared phyllites constitute the weakest bedrock unit in the gorge and have resulted from crushing.

Shear Zones - A number of shear zones have been noticed at the dam site and its vicinity. Some of them are quite prominent. Shear zones are aligned mostly along the foliation direction and trending in N 40° W-S 40°E to N 70° W-S 70°E direction with dips of 30°-45° in the southwest direction, i.e. the downstream direction. These shear zones vary in thickness from fraction of centimeter to about half a meter. Apart from these, transverse shear zones, cross shear zones, and longitudinal shear zones (with respect to axis of dam) are also present.

1. Transverse Shear Zones - There are a number of transverse shear zones in the area. During initial investigations, a very prominent, about (10-m to 15-m thick) shear zones was interpreted from drill hole data in the river bed. It was therefore proposed (design of Dam, 1973) to construct an across the river tunnel to confirm its presence. However, the subsequent drilling and electrical logging did not indicate the presence of such a prominent feature, and the tunnel was not constructed. But, now, after a period of fourteen years, its construction has been started so as to leave no doubts about its presence before construction of the dam.

2. Cross Shear Zones - There are also a number of cross shear zones in the area. The most prominent of these is exposed in the vicinity of the dam. It has a strike
of N 85° W-S 85° E and dips at 55° in south-north direction. It appears at both banks of the river, as shown in Fig. 2, and its thickness in this area is about 4.0 m.

The investigations of the No. 2 axis pointed out that very heavy overburden, of the order of a 40-m depth, would most likely be met on the right abutment. Because of this, contact of the right abutment with the dam would have to be shifted further away into the hill, thereby shifting the spillway to the right, and thus involving increased excavation or a massive retaining wall about 30-m high, which would have to be constructed at the contact of the dam and spillway. This necessitated modification in the dam axis No. 2. Axis No. 2, which is on two spurs as shown in Fig. 2, was considered. However, this was also not agreed upon because of the valley divergence downstream of it. Finally, axis No. 4 was recommended for the axis of the dam. At this axis, the downstream convergence is available and the overburden on the right abutment is less compared to axis No. 2.

Curvature in the dam axis was proposed to give a little convexity to the dam axis upstream, with a view toward providing some sort of arch action in the dam which might help in reducing the tendency of transverse crack formation in the body of the dam.

DAM SECTION

Available Fill Materials - Phyllitic rocks are not suitable for use in the dam section where high pressures are expected to occur. Thus only grade I rocks, available from chute spillway excavation, will be utilized in the upper portions of the dam. The survey for construction materials for the dam indicated that the river terrace material containing silt, sand, gravel, and boulders can be utilized in pervious zones of the dam. Therefore, initially it was decided to have the core of the dam consist of two zones. As shown in Fig. 3, zone A was proposed to consist of better clay, with a PI greater than 7% and zone B, enveloping zone A, was to consist of clay from other borrow areas.

Further investigations for the dam fill materials suggested that in case the clay in the top layer of the river terraces is mixed with the underlying pervious material consisting of silt, sand, gravels, and boulders, a very good well-graded impervious material can be obtained for the core. The proposed gradation envelope of such a blended core material is given in Fig. 4. Such blended materials have already been used in the construction of the core of high dams like Nurek (300-m) in USSR, Mica (246 m) in Canada and Oroville (234 m) in the United States. In 1960 Bleifuss and Hawke recommended the use of a well-graded mixture of fine and coarse material for the core of the dam.

Zoning of Dam Section - To be practical, all the materials used in the dam fill must be obtained from river terraces. The core material has to be obtained from the terraces where the thickness of upper clay layer is sufficient to impart the desired impermeability to the blended material without adding fine material from other areas. The pervious material for upstream and downstream shell of the dam has to be obtained from terraces where the thickness of upper impervious material from it has good drainability; this must be done without screening off the fine material. The material for the filters between core and shells has to be obtained either from a river bed where clean gravels are available, or screened from the terrace material. Thus the zoning of the dam section became a very simple job.

Fig. 2 - Geology of Dam Site

Fig. 3 - Initially Proposed Section

Fig. 4 - Proposed Gradation Envelope for Core Material
The proposed dam section is shown in Fig. 5.

Fig. 5 - Proposed Section

Shape of Core - Both the vertical and inclined cores have been provided in high dams. The core of the Mica Dam is also moderately inclined with an upstream slope of 0.41 and a downstream slope (sloping towards upstream) of 0.12; the core of the Oroville Dam has an upstream slope of 0.91 and a downstream slope (sloping towards upstream) of 0.51; and the core of Nurek Dam is a central core, with both upstream and downstream slopes of 0.251. The model studies carried out at the University of California in 1958 and at University of Roorkee in 1962 have indicated that an inclined core is somewhat more earthquake resistant. A vertical clay core was initially proposed for the section shown in Fig. 3. It was thereafter proposed to have a moderately inclined core for the dam, as shown in Fig. 5.

Thickness of Core - Site investigations revealed the presence of sound rock in the river bed at EL 579 m. The core was therefore extended up to this level. Also, it was considered to have a seepage gradient of 0.3 and therefore the thickness of the core has been increased, as shown in Fig. 6, at the foundation contact level to 0.5H.

Fig. 6 - Main Dam and Cofferdam Section

Design of Filters - For high dams, specially in seismically active areas, well designed thick filter layers are preferred as compared to transition zones that roughly satisfy the filter criterion. However, it was not possible to design filter layer with respect to the proposed blended material for the core using Terzaghi or U.S.B.R. Criterion. Sherard (1979) provided solution to this problem of broadly graded coarse soils that are internally unstable and accordingly fine to medium sand layer has been provided on upstream and downstream faces of the core as shown in Fig. 6. The rest of filter layers on upstream and chimney drain on downstream have been designed with respect to this sand layer as the base layer.

DESIGN OF COFFER DAM

It was estimated that a 104.5 m high coffer dam would be needed to safe-guard against 1 in 1000 year frequency flood with a preceding flood of 1 in 25 year and 2.7 m freeboard. Fig. 6 shows the placement of the coffer dam as part of the main dam. Its upstream and downstream slopes are 21 sl and 1.851 respectively. There have been a number of exercises with various combinations of hydro-logical parameters to determine the height of the coffer dam corresponding to 1 in 1000 year frequency flood and it has been found to vary between 83.0 m to 115.0 m. There have been discussions also regarding the design flood frequency and the decision has been in favour of 1 in 1000 year mainly because of not taking any risk for expected unprecedented flash flood in the absence of long-term hydro-logical data of the region. For the coffer dam section shown in Fig. 6, about 2.3 million m$^3$ of fill is to be placed in one non-monsoon period of about six months in a very narrow gorge of the river. Corresponding to 1 in 100 year flood the fill quantities reduce to half. Alternate designs for placing the fill in two working seasons were worked out (upto et al., 1980). The sections were designed to withstand, (i) the overtopping (Fig. Nos. 7a and 7b) and, (ii) through flow and overflow simultaneously (Fig. Nos. 8a and 8b).
There have been also suggestions for construction of concrete overflow section, below the core of the main dam, to serve as coffer dam or to reduce the fill quantities. There is yet another problem with the placing of coffer dam or the first stage dam as shown in Fig. 6. The axis of coffer dam, about 300 m upstream of the axis of main dam, would have all the problems anticipated for axis No. 2 of the dam as mentioned before.

Study to construct the first stage dam by directional blasting technique was also conducted in 1978. The inference drawn from the study was not in favour of adopting the technique mainly because of presence of very weak (sheared phyllites) rock bands, effect of heavy blasting on other structures of the project and, proximity of the site to the town.

STABILITY ANALYSIS OF DAM SLOPES

Nonseismic Condition – The outer slopes of the dam have been fixed by carrying out the stability analyses by the Fellenius method of slices (of circular slip surface) and by wedge method for the conditions of, (i) end of construction, (ii) steady seepage, (iii) sudden drawdown up to dead storage level and, (iv) sudden drawdown up to partial pool levels.

For determining the properties of fill materials, extensive sampling and testing has been done. In the borrow area for shell material (Dobata), 53 number of pits having depth of 9-10 metres were excavated (Gupta, et al., 1979). The gradation envelope of the material as in the borrow area is shown in Fig. 9. Initially the shear parameters of this material were determined by 30 cm x 30 cm size direct shear box. The maximum size of rock particle which could be used in the test apparatus was 25 mm only. The gradation curve for the specimen material was made in such a way that the curve for the coarser 70% of the material is parallel to that of borrow material as shown in Fig. 10. In order to find out the effect of particle size, plot was obtained for maximum particle size tested versus angle of friction value, as shown in Fig. 11. Extrapolation of this curve can be done for higher sizes of materials along any of the three indicated paths. The lower most curve (No. 3) indicates angle of internal friction as 42.5°. The effect of confinement was later studied by conducting tests on 38 cm dia size samples in the triaxial apparatus for shell and core materials (Lavania 1982). The angle of internal friction for the shell material (Zone III) adopted for the design is 38°. For core material friction angle is 27° and Cohesion 10 kilonewtons per square metre. For transition zone or filter layers friction angle adopted is 32°.

Seismic Condition – The conventional pseudo-static analysis was also carried out along with nonseismic condition analysis and the outer slopes were fixed on the base of minimum safety factor of 1.5 and unity for nonseismic and seismic condition with seismic coefficient of 0.15. Thereafter, a number of studies were taken up for seismic stability of the dam. Design earthquake parameters were determined for the site (1983). Fig. 12 shows the normalised plot of the design time history for the site. The recommended values (max.) of design base earthquake (DBE) and maximum credible earthquake (MCE) are 0.125 g and 0.23 g respectively.

Attempt has been made to estimate the non-recoverable deformations (1983) due to MCE, on the basis of evaluation of yield acceleration for rigid body movement of the sliding mass. As shown in the Fig. 13, only about 15.0 m and 10m depths of the upstream and downstream slopes are likely to be affected. Therefore, it has been proposed to place 20 m deep grade I rock (instead of gravel and boulder fill), which has angle of friction of about 45°. With this provision, as shown in Fig. 6, there is no likelihood of any slope deformation.

Tests were conducted at site on fill and foundation materials for determining the numerical values of dynamic shear modulus and damping coefficients. The plots of shear modulus versus strain level for the shell and core materials are
given in Figs. 14 and 15 (1983). The damping ratio of 10% has been adopted for these.

![Fig. 13 - Affected Wedges of Dam](image)

**Fig. 13 - Affected Wedges of Dam**

The shell material as available in borrow areas has quite high percentage of silt and sand (Fig. 9). Though it has been decided to restrict the silt size particles to 6% by selective borrowing but still the development of excess pore water pressure during earthquake could not be ruled out without tests on such a material. Similarly, there could be development of excess pore water pressures in the core. Therefore tests were conducted (1983) on shake table by placing the material in saturated condition with desired density and subjecting it to the sinusoidal cycles equivalent to the MCE design time history. It was observed from these tests that there would be no excess pore water pressure development (and thereby reduction in shear strength) for more than 70% R.D.

A number of finite element analyses have been carried out for the dam. Two dimensional linear and non-linear analyses do not show any sign of distress to the proposed dam section. There is, of course, slight concentration of stresses upstream of the core (in the shell) where it changes slope (from 0.9 to 1.1). But this does not seem to be of any consequence. The three dimensional analysis, which are in progress, are likely to present a more rational picture as the valley is very narrow and confinement effect would be substantial.

**Foundation Problems**

The foundation problems, which include abutment contact problems, are of the following two types: (1) Routine problems of earth dams; (2) special problems of the Tehri Dam arising out of the geological topographical conditions (Design Memo. 1973, Lavana 1975 and Gupta et al., 1979).

Routine problems that are characteristic to any earth dam generally relate to: (1) Seepage through foundation rocks (2) development and accumulation of water pressure in abutments; (3) removal of weathered or undesirable zones in foundation; (4) easing off the overhangs and local steep slopes in the area and; (5) treatment of local pockets of weak material, seams, and joints.

**PROPOSED REMEDIAL MEASURES**

**Routine Foundation Problems**

**Foundation Grouting** - For making the foundation sufficiently impervious so as to prevent excessive seepage, curtain grouting and blanket grouting in the core and transition filter contact was earlier proposed. In view of the highly jointed and sheared phyllitic rocks in the foundation, a three-line grout curtain was proposed. The depth of the primary holes at 12.0 m spacing would be 0.3 H, with a minimum of 25.0 m at the crest. The secondary and tertiary holes would be about two-thirds and one-third the primary hole depth. This depth can be modified depending on the grout intakes.

Area grouting for the core contact area and downstream fine filter has been planned at 3.0 m spacing 10.0 m deep upstream of the curtain and 5.0 m deep downstream. The foundation grouting plan is given in Fig. 16.

![Fig. 16 - Foundation Grouting Plan](image)

**Fig. 16 - Foundation Grouting Plan**

There has been lot of thinking on this aspect of foundation treatment during last about ten years and a number of alternative proposals have been studied. Though, the groutability tests conducted at the site have not shwon any cause of concern; but the unprecedented height of the dam made some engineers to think for post construction grouting arrangements and accordingly the proposals of having foundation grouting tunnels as provided in Nurek Dam and Chicoasen Dam were prepared. Recently, proposals have been prepared that have grouting tunnel below the core either fully or partly (cut and cover) in the foundation rock and connected to shafts or abutment drainage tunnels on both the banks.

**Drainage Tunnels** - For preventing development and accumulation of pore-water pressures in the abutments, drainage tunnels are proposed at EL 700 m on both banks. A rock cover of 30.0 m (100 ft) minimum has been proposed over the tunnel below the dam fill.

**Foundation Stripping** - The ASCE Committee (1972) found, for the dams studied, that the overburden-having depth of ± 10.0 ft (3.0 m) and less was removed from the entire area. In case of the Mica Dam, the overburden (called common material) was removed only below the core foundation; at Oroville Dam, the removal was done over the entire area; and at Bennet Dam it was in the area below...
core and filters. The depth of overburden at the Tehri Dam area is not uniform. It has been proposed that in the area under upstream and downstream streams where overburden is met, stripping of a maximum 6.0 m (20.0 ft) be done. But the area under the dam core would be excavated up to sound rock level to provide an effective cut-off. The core foundation level and shell foundation level will be joined by excavation at a gentle slope not steeper than 1H:1V.

Abutment Easing - As shown in Fig. 17, the gorge section has slopes of right and left abutments as 1:1.1 and 0.9:1, respectively. Therefore, it is proposed that the local steep slopes and overhangs if any, be eased at a slope of 70° (0.36H:1V), as was done at the Mica Dam. However, the depth of silt, gravel, and boulder-fill in the river bed is expected to be of the order of 10-15 m. The gorge in this depth is likely to have almost vertical walls. Therefore, considerable excavation may have to be done. Keeping this in view, another proposal, as mentioned in the following section on "Special Treatment", is under consideration.

![Fig. 17 - Proposal of Concrete Plug](image)

Treatment of Joints and Seams - It is proposed that the foundation below core and filters be cleaned of loose and foreign materials, joints, faults, shear zones etc.; that it be cleaned up to a depth at least three times their width; and that it be back-filled with concrete or pneumatically applied mortar. Larger depressions would be backfilled with concrete.

Study has also been done to assess foundation settlements after the dam construction on the basis of plate load test data (Gupta et al., 1979).

Special Foundation Problems

Concrete in River Section - The abutments of the Tehri Dam are steeper than those of the Mica, Oroville, and Bennet Dams, and the water head is also greater than for these dams. However, the abutment slopes of the Nurek Dam (300 m) are of almost the same order, and the water head is also greater than that for the Tehri Dam. For the Tehri Dam, in case the excavation of abutments is done at the limiting slope of 70° from the core foundation level in the river bed, it would involve a huge excavation of the rocks and for a considerable height the abutment contact would be at a slope (70°) steeper than that available naturally from above the river water level. In the Nurek Dam, for a similar situation, it has been considered economically and technically sound to have the portion at the river bed filled with concrete (Odintov and Gladkov, 1971). This concrete block in the river bed, under the core contact area, is called a "concrete plug". It is 157 m long, 30 m-60 m wide, and accommodates three grouting galleries. Similar proposal (Fig. 17), with and without grouting galleries have been proposed for the Tehri Dam.

Counterforts at Junction - As the spillway is very close to the dam, a retaining wall of the order of about 25 m height is to be constructed at the junction of the two structures. The main problem is to obtain a secure junction between the core of the dam and the retaining wall. This problem is likely to be greater importance when the behaviour of this junction is visualized during an earthquake. At present, the wall of counterfort type is proposed.

CONCLUSIONS

The paper describes the design of main features of a 260.5m high Tehri rockfill dam with brief explanations as to what changes have been made time to time. This is to augment the rockfill dam technology and invite suggestions.

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