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DISCUSSION

Proc. Paper 11534

TECHNICAL NOTES

Proc. Paper 11538

[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

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of these indicators point out changes in rock or cutter conditions, forward thrust or cutter spacing should be promptly changed in order to attain an optimum working condition.

Appendix.—References

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Tehri Rockfill Dam

By Bhagwat V. K. Lavania[1]

Introduction

The Tehri Project is an irrigation-power project of the State of Uttar Pradesh, in India, envisaging construction of: (1) A 260-m high rockfill dam across the Bhagirathi River to store 2,880,000 acre-ft of water; (2) underground power houses, having installed capacity of 1,800 MW; (3) spillway for design discharge capacity of 12,000 m^3/s; and (4) four diversion tunnels of 11.25-m diam each. 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 of highly jointed and sheared phyllitic rocks and its abutment contact slopes be steep. The area is also seismically active. The problems examined are: (1) The location of the dam axis; (2) the dam section; (3) the approach to slope stability analysis under static and dynamic conditions; and (4) the foundations treatment.

Geology of Area

The rocks exposed in the area of the project site are phyllites of Chandpur-Series. 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 predominently arenaceous, massive in character, and distinctly jointed at phritiferous places that have lenticular elongated streaks of brown colored calcarlous material. The foliation planes are least developed. A number of bands of this variety occur in the Tehri gorge as pronounced ribs.

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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. The rock in this unit is considerably impregnated with quartz veins, both along and across the foliation planes.

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 puckers.

Sheared Phyllite.—The sheared phyllites constitute the weakest bedrock unit in the gorge and have resulted from crushing. They show pinching and swelling

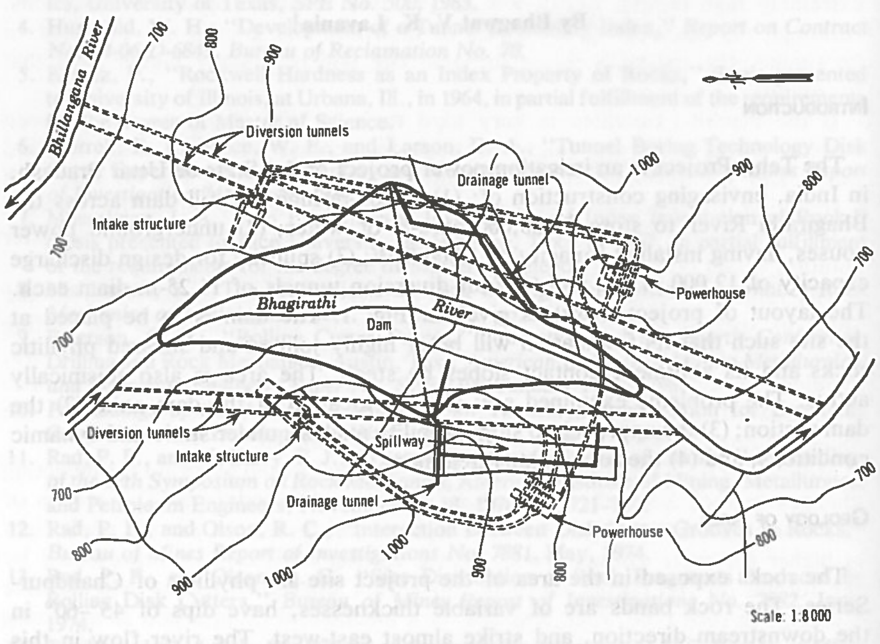

FIG. 1.—Layout of Project Works

along their trace. Almost all the gullies seen in the Tehri gorge have been guided by the structural control offered by the various types of shear zones. Calctuffa has also been deposited along some of these gullies, testifying to their capability to permit seepage. The rock in the shear zones vary from brecciated to gouged.

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 alined 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 a fraction of a 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. The first type is alined parallel to the gorge, cuts across the foliation, and dips almost vertically. The second type is developed along planes cutting across the foliation planes and dips in an upstream direction. The third type is oriented across the gorge, trends some what oblique to the foliation, and dips approximately vertically. The thickness of these shear zones is also variable. The cross and transverse shear zones have been considered more important for the study of the layout of project structures:

1. Transverse Shear Zones—There are a number of transverse shear zones in the area. During earlier investigations, a very prominent, about (10-m to 15-m thick) shear zone was interpreted from drill hole data in the river bed.

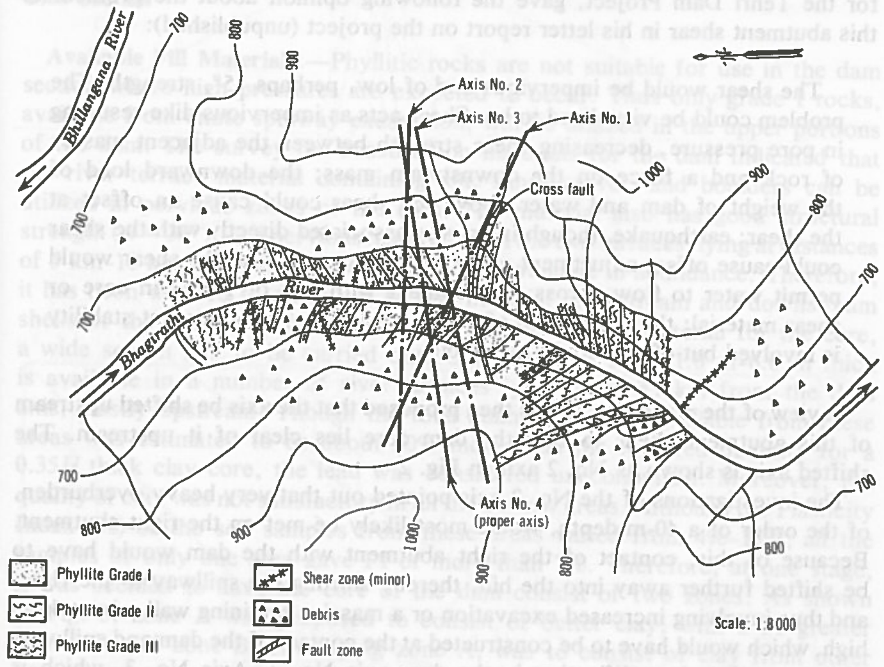

FIG. 2.—Geology of Dam Site

However, the recent drilling and electrical logging has not indicated the presence of such a prominent feature.

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. Like all other shear zones in the area, this also shows considerable pinching and swelling in thickness along its trace. It appears at both banks of the river, as shown in Fig. 2, and its thickness in this area is about 4.0 m.

Dam Axis

A number of alternative axes were considered for the dam. The main considerations for the suitability of the dam axis were safety of the structure and economy. The safety of the dam became important due to complex geology of the foundations and the area being seismically active. The economy considerations included: (1) Quantity of fill materials; (2) foundation treatment; and (3) cost of other structures of the project with respect to the axis.

Initially dam axis No. 1, as shown in Fig. 2, was considered the suitable axis. It provided excellent seating for the dam. But with it, the major cross fault was coming under the core of the dam and therefore any movement along the fault due to earthquake or otherwise would have created some damage or even rupture of the core. The problem became more important in view of steep valley slopes. In 1972, J. B. Cooke, a member of the Board of Consultants for the Tehri Dam Project, gave the following opinion about the problem of this abutment shear in his letter report on the project (unpublished):

> The shear would be impervious and of low, perhaps 15°, strength. The problem could be visualized to be: Shear acts as impervious dike resulting in pore pressure, decreasing shear strength between the adjacent masses of rock and a force on the downstream mass; the downward load of the weight of dam and water above the shear could cause an offset at the shear; earthquake, though in no way associated directly with the shear could cause offset adjustment at the shear; any offset at the shear would permit water to flow across core contact with risk of piping in core or shear material; the orientation of shear is such that no abutment stability is involved but offset would be serious.

In view of the preceding, it was then proposed that the axis be shifted upstream of this abutment shear so that the dam core lies clear of it upstream. The shifted axis is shown as No. 2 axis in Fig. 2.

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 50-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. 3, 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. A curvature of about a 3,000-m (10,000-ft) radius was proposed to be given in the axis resulting in maximum displacement of 26.0 m at the center of the axis, i.e., about 4.6% of the axis length and 10% of the total height of the dam; such a convexity has been provided in the axis of a number of other dams as well. But Cooke is of the view that such a large radius of curvature cannot be useful for the purpose of providing arch action for preventing the transverse crack formation. Also, according to Cooke,

a little more money would have to be spent for giving the layout of the dam with a curved axis compared to a straight one. However, technically, he does not see any disadvantage in providing a curved axis. Some other members of the Board of Consultants for Tehri Dam Project are of the opinion that convexity in the axis should be provided—even if there is doubt about its positive advantage. Therefore, it is possible that a curved axis may be considered again; especially if it is coupled with another advantage, such as better abutment contact (mainly at the right abutment) with the topography and rock levels at the site.

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. The material also has good structural strength ($\phi = 38°$). The pervious material from the two terraces, lying at distances of 5 km–10 km upstream of the dam axis, is available in abundance. Therefore, it has been decided to make use of this material in upstream and downstream shells of the dam and transition filters. For impervious material for the core, a wide search had to be carried out. Clay as a top layer 1.0 m–6.0 m thick is available in a number of river terraces lying 5 km–35 km from the dam axis, mostly upstream. Though the total quantity of clay available from these areas was estimated to be about 50% more than the required quantity for a $0.35H$ thick clay core, the lead was considered uneconomical. Moreover, the quality of clay was not satisfactory in all the borrow areas. Although the Plasticity Index (PI) of the soil samples from these areas varied from 4%–18%, all the samples of only one area gave PI of more than 7%. Therefore, at one stage, 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. With such a core material, it would be possible to obtain fill materials for the dam from a borrow area within 10 km of the dam site. The proposal to make the core of the blended material has been considered economically and technically sound. The proposed gradation envelope of such a blended core material is given in Fig. 4. The core material would be comprised of a mixture of 20%–40% minus No. 200 U.S. Sieve size clayey silt, and the rest of sand to gravel particles of sizes from 75 mm up to 150 mm or so. Such blended materials have already been used in the construction of the core of high dams like Mica (244 m) in Canada and Oroville (234 m) in the United States. In 1960 Bleifuss and Hawke (1) recommended the use of a well-graded mixture of fine and coarse material for the core of the dam.

The project lies in the seismically active area of the country. Earthquakes with an 8-MM scale intensity have occurred on this seismic belt. Thus, the blended material of the core is also in accordance with the recommendations of Sherard (10) for leak-resistant material. Webster (12), while describing the crack resistant measures taken at Mica Dam, mentions the use of blended material in the core as one of them.

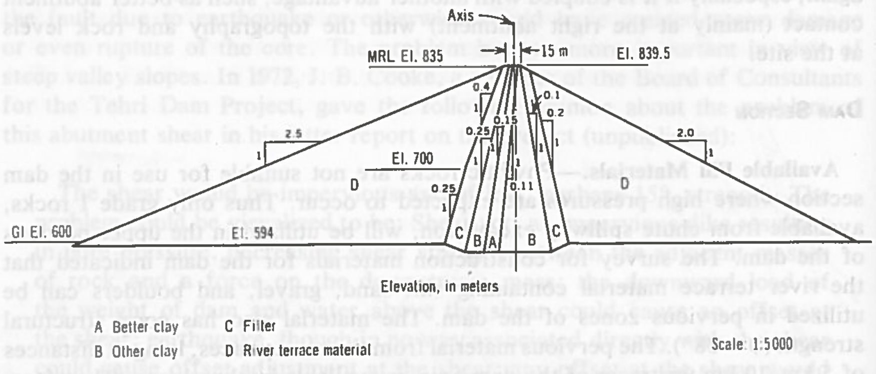

FIG. 3.—Initially Proposed Section

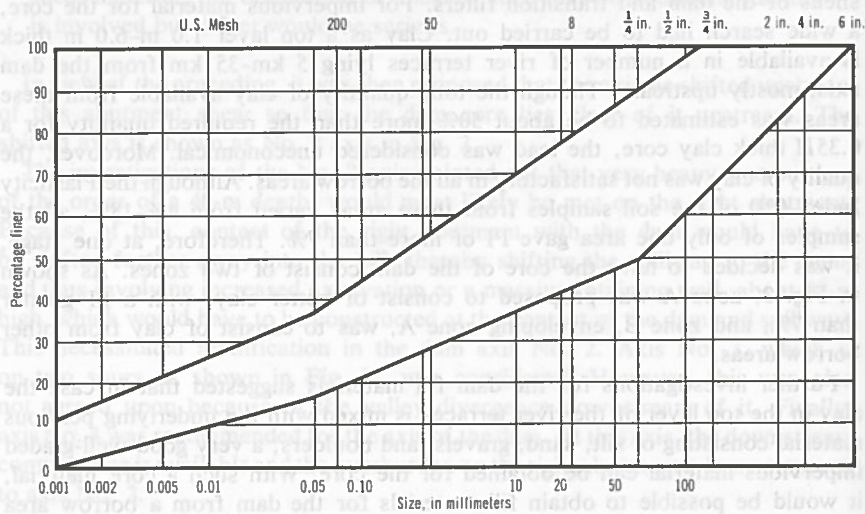

FIG. 4.—Proposed Gradation Envelope for Core Material

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 is minimum so that the 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. The proposed dam section is shown in Fig. 5.

Shape of Core.—Both the vertical and inclined cores have their advantages

TABLE 1.—Fill Material Characteristics

Characteristics (1)	River terrace material, Zone C (2)	Core material, Zone A (3)	Filter material, Zone B (4)
Dry unit weight, in kilonewtons per cubic meter	19.0	18.5	19.0
Moist unit weight (at OMC), in kilonewtons per cubic meter	20.0	19.5	20.0
Saturated unit weight, in kilonewtons per cubic meter	22.0	21.5	22.0
Buoyant unit weight, in kilonewtons per cubic meter	12.0	11.5	12.0
Angle of friction, in degrees	38	27	32
Cohesion, in kilonewtons per square meter	—	10.0	—

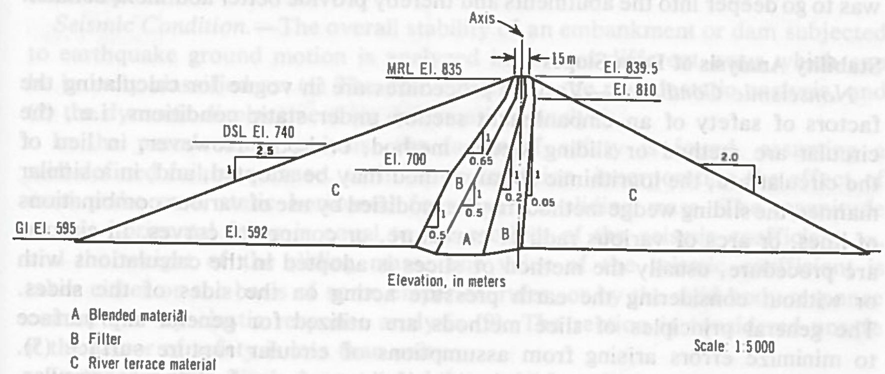

FIG. 5.—Proposed Section

and disadvantages. Vertical core provides better contact with the foundation and provides slightly more thickness of the core for the same quantity of the core material. In the case of small height dams, grouting of foundation can be done throughout the post-construction period, if needed. The settlement of the core is independent of the settlement of the downstream shell. An inclined core provides the facility for placing bulk quantity of downstream shell before placing of the core, and also for carrying out foundation treatment during this period. This is a positive advantage of the inclined core, especially in countries where the dry weather period for placing the core is rather short (this is not true in the case of Tehri Dam). The drawdown pore pressures developed are also less than those in the central core. The model studies carried out at the University of California in 1958 (2) and at University of Roorkee in 1962 (4) have indicated that an inclined core is somewhat more earthquake resistant. However, if there is excessive post-construction settlement in the downstream fill, due to any cause, a thin sloping core may crack. Also, due to core material being the structurally weakest of all fill materials, the upstream slope may become flatter. Because of this consideration, a vertical clay core was initially proposed for the section shown in Fig. 3. But after deciding to have the core of blended material, this consideration becomes unimportant. In view of all these considerations, it is now proposed to have a moderately inclined core for the dam, as shown in Fig. 5. To keep the core away from the upstream slope, for about the top one-tenth of the dam height, the proposed core should be vertical. Looking at the other high dams of the world, the core of the Mica Dam is also moderately inclined, with an upstream slope of 0.4:1 and a downstream slope (sloping towards upstream) of 0.1:1; the core of the Oroville Dam has an upstream slope of 0.9:1 and a downstream slope (sloping towards upstream) of 0.5:1; and the core of Nurek Dam (300 m) in the Union of the Soviet Socialists Republic is a central core, with both upstream and downstream slopes of 0.25:1.

Thickness of Core.—As shown in Fig. 5, the proposal is to have the thickness of Tehri Dam core equal to 0.3 times the height of the dam, H, plus the top width. The core thickness of the Mica, Oroville, and Nurek Dams are $0.3H$, $0.4H$, and $0.5H$ plus top width, respectively. In the case of the Tehri Dam, with the relatively thin core adopted for the dam, the decision of the engineers was to go deeper into the abutments and thereby provide better abutment contact.

Stability Analysis of Dam Slopes

Nonseismic Condition.—Various procedures are in vogue for calculating the factors of safety of an embankment section under static conditions, i.e., the circular arc method or sliding wedge method, or both. However, in lieu of the circular arc, the logrithimic spiral method may be adopted, and, in a similar manner, the sliding wedge method may be modified by use of various combinations of lines, or arcs of various radii of curvature, or composite curves. In circular arc procedure, usually the method of slices is adopted in the calculations with or without considering the earth pressure acting on the sides of the slices. The general principles of slice methods are utilized for general slip surface to minimize errors arising from assumptions of circular rupture surface (5). As reported by Terzaghi and Peck (11), if the surface of sliding is circular, the improvement in accuracy by considering the side forces acting on the slices is not likely to exceed 10%-15%. On the other hand, if the surface of sliding is not circular, the error due to omission of the forces may be significant.

The preceding information, in brief, indicates that the factor of safety is affected by the shape of the failure surface and the method of analysis. The values of the factor of safety also vary according to the way the shear strength parameters are selected for the design, i.e., whether minimum, maximum, or mean values are taken. Actually, during construction there is variation in these values also due to variation in: (1) Material quality and water content in different borrow areas; (2) placement rate, method, and water content, and (3) climatic

conditions. There are other factors that also affect the value of the factor of safety, e.g., the unknown stress distribution inside the body of the dam, the method of taking into account the pore-water pressures, the assumption of treating the analysis as a limiting equilibrium problem, etc. In view of this, the factor of safety obtained with any of the methods of analysis can at best be regarded as a "working" factor of safety. The analysis of the Tehri Dam Section by the Fellenius method of slices (of circular slip surface) and by wedge method is in hand. The dam section is being tested for the following usual conditions of loading:

Conditions of analysis	Slope
End of construction	Upstream and downstream slopes
Steady Seepage Condition	Downstream slope
Sudden draw down	
Up to DSL	Upstream Slope
Up to levels up to DSL (partial pool)	Upstream Slope

The unit weight, angle of friction, and cohesion values of the fill materials that have been adopted for the analysis of the dam section, on the basis of preliminary testing of the borrow materials, are given in Table 1. These values are subject to modifications in the light of subsequent test results.

The allowable factor of safety for all the three conditions of analysis has been kept as 1.5.

Seismic Condition.—The overall stability of an embankment or dam subjected to earthquake ground motion is analyzed in several different ways which can be broadly classified as: (1) The conventional or the pseudostatic analysis, and (2) the dynamic displacement (or deformation) analysis.

In the pseudostatic analysis, the factor of safety is found, assuming a well-defined failure surface, as in static condition, incorporating the effect of earthquake as a static horizontal force on the sliding mass. The magnitude of this horizontal force is equal to the product of the seismic coefficient, α, and the weight of the sliding mass. The value of the seismic coefficient is taken either on the basis of some emperical rules, or by the rigid body response concept, or by elastic response analysis (9). The section is considered unsafe if the factor of safety is less than unity.

In the dynamic displacement (or deformation) analysis, an attempt is made to find the displacement in the dam section due to an expected earthquake. This seems rational, because the inertia force due to an earthquake may cause the stresses to exceed the strength intermitently for short intervals of time only, and the factor of safety along the assumed failure surface during these short intervals may become less than unity; but the result of this may only be some displacement in the embankment and not total failure. Newmark (6) has given an approach to find the dynamic displacement considering the embankment material rigid plastic in nature and the movement of the soil mass occurring along a well-defined surface, i.e., the surface of least resistance as obtained for static analysis. Seed (8) has suggested an alternative approach for estimating deformations of the slope from the observed deformations of the soil elements comprising it.

It is proposed that pseudostatic and dynamic analysis, for the Tehri Dam, be carried out using the preceding approaches. The pseudostatic analysis using the Swedish Slip Circle Method and the Fellenius method of slices taking into account an empirical seismic coefficient equal to 0.15, is being carried out. The value of allowable factors of safety for different reservoir conditions and the value of the proposed seismic coefficient in each case are given as follows:

Condition	Minimum factor of safety	
	$\alpha = 0.075$	$\alpha = 0.15$
End of construction	1.0	—
Steady seepage and partial pool		1.2
Reservoir drawdown	1.0	—

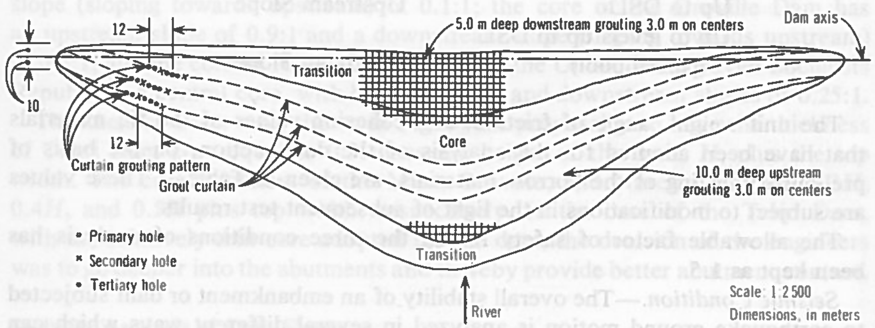

FIG. 6.—Foundation Grouting Plan

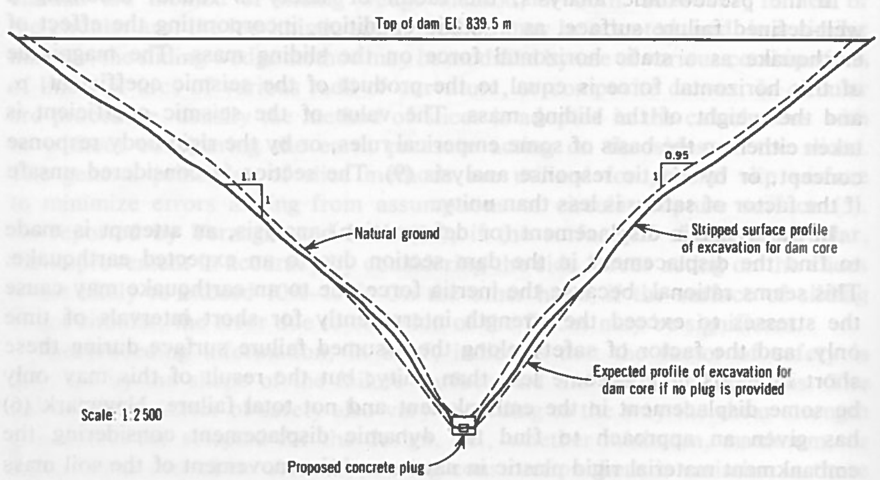

FIG. 7.—Proposal of Concrete Plug

As indicated, the value of the seismic coefficient—for the condition which is likely to be temporary—has been adopted to be equal to 0.075, i.e., half of the normal value.

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.

Routine problems that are characteristic to any earth dam generally relate to: (1) Seepage through foundation rock; (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.

The special foundation problems which are involved in Tehri Dam relate to: (1) Provision of a secure junction between the core of the dam and the foundation bed rock; and (2) provision of a secure junction between the core of the dam at the right abutment and left side wall of the chute spillway.

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 area have been proposed. In view of the highly jointed and sheared phyllitic rocks in the foundation, a three-line grout curtain has been proposed. The depth of the primary holes at 12.0-m-spacing would be $0.3H$, with a minimum of 25.0-m spacing 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. 6.

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, as shown in Fig. 1. A rock cover of 30.0-m (100-ft) minimum has been proposed over the tunnel below the dam fill. A portion of these drainage tunnels is alined in the direction of strike of the rock bands, which may not be very effective as it would not intercept the various rock bands and water pockets along bedding planes.

Foundation Stripping.—Removal of weathered rock is desirable to improve the shear properties of foundation. However, it may not be necessary to remove the weathered rock in the entire area and in the entire depth below the dam; in the interest of economy, the removal of weathered rock in the dam core and filter contact areas only may be sufficient. The ASCE Committee (3) 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. At some places, rock is exposed while at other places, overburden of the order of 10 m–15 m, or even more (at the right abutment), is expected to be encountered. In the river bed, the depth of silt, gravels, and boulders has been estimated to be above 10 m. In view of this, it has been proposed that in the area under upstream and downstream shells 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 a depth of 3.0 m below sound rock level to provide an effective cutoff. As the heavy overburden is expected only at higher elevations where the height of the dam would be less, the stripping of only 6.0 m is not likely to endanger the stability of the dam section. However, the danger of liquefaction during an earthquake has to be investigated. 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. 7, the gorge section has slopes of right and left abutments as 1.1:1 and 0.95:1, respectively, and no overhangs in the rock are indicated. 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 10.0 m (33.0 ft). The gorge in this depth is likely to have almost vertical walls. Therefore, in case the easing of the valley slope is done at a 70° angle, as mentioned previously, from the core foundation level in the river portion, 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.

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. The areas outside the core and filter zones will, however, be leveled as such to the riquired profile for placing and compacting the fill.

Special Foundation Problems

Concrete in River Section.—As mentioned under "Routine Foundation Problems—Abutment Easing," the river bed is filled up to a depth of about 10.0 m, with loose material and the valley walls at this depth are expected to be almost vertical. Therefore, this creates a serious problem of obtaining a secure junction between the core of the dam and its seat. The abutments of the Tehri Dam are steeper than those of the Mica, Oroville, and Bonnet 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 (7). 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 gallaries of transverse dimensions 3.5 m × 4.5 m (upstream gallery), 4.5 m × 5.5 m (central gallery), and 3.5 × 4.5 m (downstream gallery). A similar proposal (Fig. 7) is proposed for the Tehri Dam. The economical and technical aspects of the problem are being worked out.

Counterforts at Junction.—As the chute spillway is very close to the dam and the control structure is just downstream of the dam axis, a retaining wall of the order of about 25 m high is to be constructed at the junction of the two structures. The main problem to be encountered in this connection is to obtain a secure junction between the core of the dam and the retaining wall, so as to avoid any chances of free seepage between the two. This problem is likely to be of greater importance when the behavior of this junction is visualized during an earthquake. At present, the wall of counterfort type is proposed. The distance between counterfort projections may be kept sufficient so as to allow placing and compaction of the core material by machines. The optimum length of the counterforts to be embedded inside the core may be decided on the basis of required seepage length at the junction.

Conclusions

The paper describes the main features of a 260-m high Tehri rockfill dam with brief explanations as to how these have been achieved. This is to augment the rockfill dam technology and invite suggestions.

Acknowledgment

The design of Tehri Dam project structures is being carried out in the design wing of the Uttar Pradesh State (India) Irrigation Department under the direction of A. N. Harkauli.

Appendix I.—References

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DISCUSSION