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Performances of Some Embankment Dams in India

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SYNOPSIS The performance in terms of pore pressure and deformation of five earth and earth-rockfill dams is briefly presented. These dams are located in different geologic formations. Their pore pressure response has been low due to relatively longer construction periods. The deformation behaviour has been similar to the dams constructed earlier. The valley ratio seems to influence the strain condition along the height and longitudinal axis of the embankment dam.

INTRODUCTION

Over the last 35 years a large number of earth and earth rockfill dams have been constructed in India as components of multipurpose river valley projects. The maximum height so far constructed is only 132.5 m and the one with a maximum height of 260 m is under construction. These dams have been constructed using essentially locally available soils. The choice and guidelines for the selection of soils to be suitably placed in different zones, the laboratory and field studies, compaction control during the construction stage including the thickness of layers before compaction and control of moisture content essentially followed the guideline as laid down by USBR. A number of these dams have been instrumented for the measurement and control of construction pore water pressures and deformations effectively. Foundation settlements were also recorded wherever soils and soft rocks were encountered. Both pore pressure and deformation devices similar to those adopted by USBR were installed. The construction period in general was longer than that achieved in the USBR dams. The valley ratio, i.e., width of the valley to the height of the dam varied from little over one (in the case of Ramganga) to over three hundred (as in Pochampad).

This paper presents brief case histories of five earth and rockfill dams, namely, Beas, Ramganga, Linganamakki, Pochampad, and Tawa, whose heights vary from 36 m to 132.5 m. These dams are located in different geologic settings with foundation materials varying from soft lateritic soil to weathered rock. The types of soils used, the kind of instruments installed and the response of these dams under construction loads and reservoir filling are presented in comparison with the performance of some of the dams in USA.

BEAS DAM

Beas dam located in the foothills of the Himalayas is founded on Siwalik series consisting of alterations of sand rock and clay shale. A bed of boulders was located from 27 m to 76 m below the ground level. The clay shale bands are generally massive but include silt and sand fractions. The sand rocks are coarse grained and showed considerable loss of strength upon soaking.

The height of the dam above the foundation level after completion is 132.5 m completed during the period 1969-74. The valley ratio is about 17. The central clay core with gravel shell was adopted as shown in Figure 1. Core essentially consists of a mixture of clay shale and sandstone. For the filter and the pervious zone hard quartzite with 22 to 31% sand and 40 to 77% gravel was used. The core was compacted

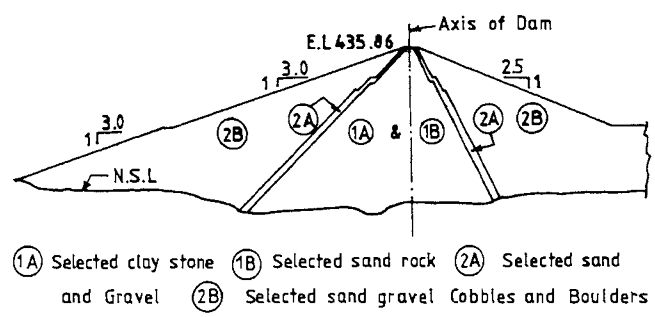

Fig.1. Cross-section of Beas Dam

at -2% to +1% of the optimum moisture content to achieve 98% of Standard Proctor density, (Berry,1976).

In order to study the performance of the foundation and dam a total of 145 USBR type piezometer tips (58 foundation type and 87 embankment type) were installed in the trenches dug, filled with the same material and carefully compacted.

For the design initial pore pressure was estimated assuming undrained condition for laboratory compacted specimen under one dimensional consolidation using Hilf's approach (1948). For impervious zone this would work out to be 30% of the total stress. Tests conducted on undisturbed specimens revealed high pore pressure and used for recheck of the downstream slope as under

σ_v kg/sq.cm.	Pore pressure, percent
< 5.0	50
5.0-9.5	67
> 9.5	100

where σ_v is total stress.

Actual measurements from the piezometers installed in the deepest river bed suggested higher pore pressure (about 80%) than assumed for the design in some regions due to packets of impervious soil. In some other regions the pore pressures were lower. The moisture checks after compaction revealed varying water content between + 0.4% to - 2% optimum moisture. The average moisture content of 86% of samples showed lower than optimum moisture. Maximum pore pressure zone developed was close to the foundation since the permeabilities of the foundation and core material were of similar magnitudes. The dissipation of pore pressure resulting stiffening of soil structure slowed down further development after shutdown period or for periods following very slow progress of fill construction.

For the measurement of settlement of foundation and compression in the dam, seven USBR telescopic cross-arm were installed at 3 sections of the dam. To measure the cross valley deformations, six of the telescopic cross-arms were provided with internal horizontal movement devices at 30.5 vertical intervals. An inclinometer was installed at the deepest section through the impervious core. Surface monuments to measure both settlement and horizontal alignment were set up on the upstream and downstream faces and on the upstream parapet wall on top of the dam.

During the early construction period consolidation was faster which slowed down with time. On an average 22.5% of the total consolidation took place during the first one month after placement. Generally lower layers showed higher consolidation than the upper layers. The rate of consolidation soon after completion of dam remained very small almost uneffected with the filling up of the reservoir. The rates of consolidation varied upto 0.22 mm/day. The submergence of pervious layers resulted in a settlement of about 0.9%. During the three years period of shut down on the right abutment section little compression took place. The maximum settlement occurred at mid-height of the dam. The shape of the settlement versus height of embankment curves were essentially parabolic in shape.

The foundation settlements continued to increa with the embankment rise and also during the shut-down period on the right abutment. The maximum foundation settlement was about 0.4%, see Figure 2, (Berry,1975). The cumulative em bankment settlement was about 2.5%. The verti cal strain versus effective stress relationshi was essentially linear on log-log plot confirm ing Gould's findings,(1954). In comparison to USBR dam where vertical compression under 7 kg cm^2 ranged from 1 to 4%, the Beas dam experien ced 1.1 to 1.7% while at 0.70 kg/cm^2 the range was 0.1% to 0.4%. The maximum upstream-downstream deformations observed in the lower half and upper half of the dam were 300 mm and 250m respectively. The maximum deformations increa sed along the axis on the right flank side was 215 mm and 88 mm on the left flank.

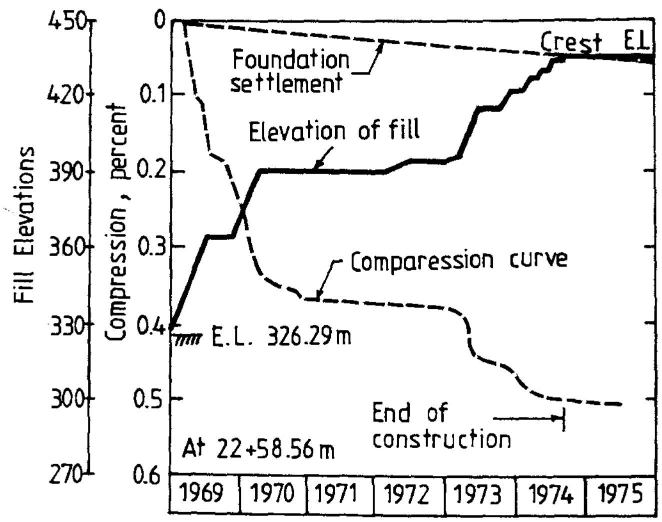

Fig.2. Typical Foundation and Embankment Response of Beas Dam

RAMGANGA DAM

The main earth and boulder fill dam (127.5m high) constructed across Ramganga rests on foundation consisting of alternate bands of sand rock and clay shale of middle Siwalik age The main body of the dam is composed of judicious zoning keeping in view the available material to form zones of crushed clay shale, crushed sandstone and river bed material of varying sizes. Wherever coarse fractions were present, attempt was made to achieve 100% proc tor density for the fractions finer than 4.8mm Fines smaller than 0.074mm for clay stone, sandstone and river bed material were 46%, 21% and 5% with respective maximum sizes of 15 cm,

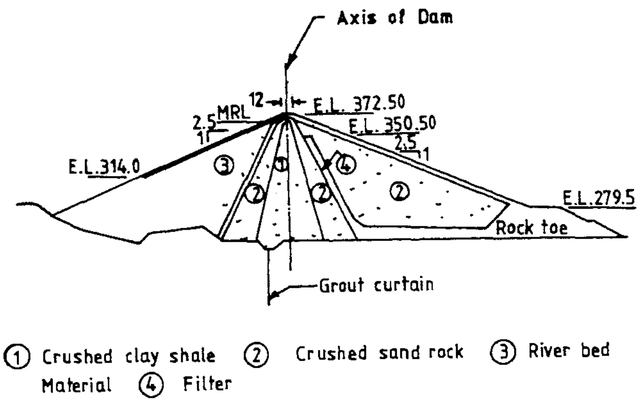

Fig.3. Cross-section of Ramganga Dam

20 cm, and 43 cm. The claystone was compacted by 12 to 14 passes on 15 cm thick layers. For sand stone the number of passes varied from 17-21 on layers 30 cm thick. The river bed material was compacted by 2-3 passes of 10 tonne vibratory roller on 60 cm thick layer. A section of the dam is shown in Figure 3; the construction of the dam was completed during 1970-74. The valley ratio is just above one.

Twin tube hydraulic piezometers, USBR type (43 in the embankment and 6 in the foundation) were located in a cross-section at the deepest river bed where the rock was loose jointed and fissured and took heavy grout intake. Five electrical piezometers were also installed to check the reliability of the hydraulic type.

Based on the one dimensional consolidation tests carried out on crushed clay shale and crushed rock (both less than 4.8 mm) compacted at +2%, + 1% and at optimum moisture content and using Hilf's equation construction pore pressures were estimated as (Manglik and Gupta,1977),

(i) for crushed clay shale = $0.50\ \sigma_v$

(ii) for crushed sand rock = $0.250\ \sigma_v$

(iii) for downstream shell = $0.125\ \sigma_v$

where σ_v is the fill total stress.

The construction pore pressure strictly followed the pattern of construction schedule. A general trend is indicated in Figure 4 for crushed rock. The pore pressure ratio in the early stage of construction was high but with increase of height of fill, it decreased to 0.25 as was assumed in the design. The pore pressure built up in this zone closely followed with the reservoir water level even in the first filling, see Figure 4. The permeability of the sand rock at maximum dry density was less than 20×10^{-6} cm/sec. In the case of clay shale the minimum pore pressure ratio was 0.45 and maximum was 0.80 but most values were below 0.55. This ratio was 0.3 for clay shale and 0.1 to 0.15 for sandstone. In the case of

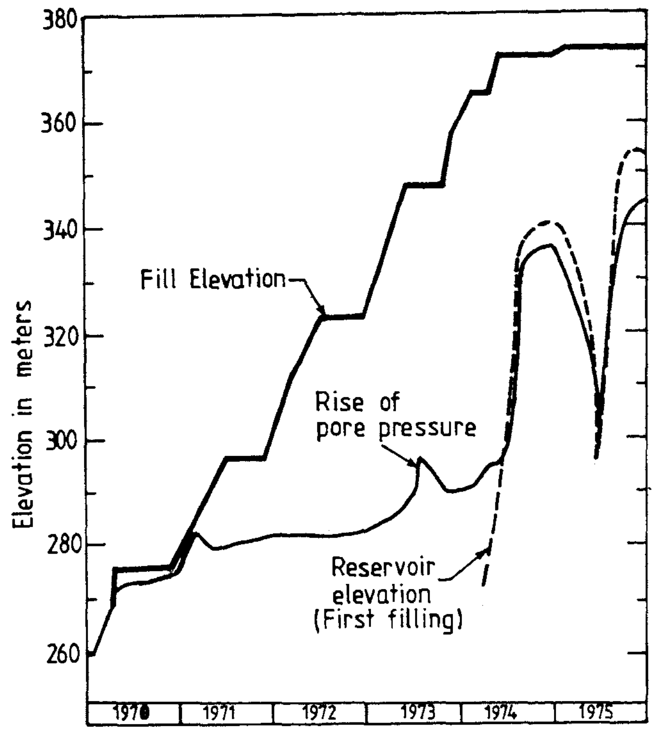

Fig.4. Pore Pressure Response in Shell Zone of Ramganga Dam due to Embankment Construction and Reservoir Filling

clay shale the pore pressure responded well with the reservoir fluctuations only after the 2nd year of filling. Initial shutdown period extending beyond 4 months considerably helped in restricting the further development of pore pressure to construction load. The contours of highest intensity of pore pressure developed at the central region of the lower part of the core. In the upstream portion of the core, pore pressure did increase by about 8% during the first reservoir filling load on the upstream slope.

Compression characteristics of the embankment closely followed the sequence of construction. The total compression in the fill was about 1.8% of the height of the dam. In the clay stone forming the core, the percentage compression at 10 kg/cm^2 was about 3.3%, in the sandstone this value was 2.9%. The post construction rate of compression was about 0.12% for 3 years, which is less than that has been observed by Gould (1954) for USBR dams. The shape of the pore pressure and compression curves were essentially same, as the theoretical curves obtained by Hilf's method for 10.5% and 12% moisture content with placement density of 2 T/cu.m. In general core underwent larger compression than the shale resulting differential movements at the intersurface.

The maximum horizontal strain in the clay core was only 0.25% suggesting compression occurring essentially in the vertical direction. Compres-

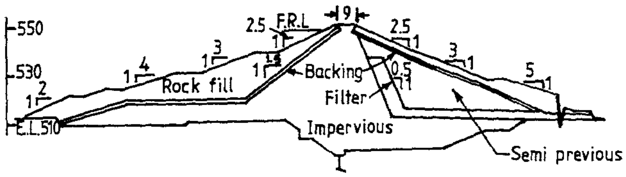

Fig.5. Cross-Section of Linganamakki Dam

sive strains at lower elevation and tensile strains at higher elevations were observed. The shape of the valley resulted compressive strains along the longitudinal axis of the dam suggesting intermediate principal stress larger than that corresponding to plane strain condition. The magnitude of this stress appears to vary along the length and height of the dam.

LINGANAMAKKI DAM

The right flank of Linganamakki dam is composed of earth and rockfill with a maximum height of 51m rests on lateritic soil and weathered rock. Granitic gneiss and hornblend schist form the main rock foundation for the masonry-cum-concrete portion of the dam. After eliminating the top 0.5m, the residual soil, formed along the hill slopes due to deep sub-areal weathering and leaching action of parent rock, was used for the semipervious zone and the fine grained soil lying below this coarse grained material was used for impervious core. Granite gneiss formed rockfill. Since most part of the foundation consists of fine grained, plastic, low strength lateritic soil flatter slopes upto 3:1 were adopted for the section, see Figure 5. The valley ratio is about 19.

The fill compaction was achieved with the use of sheepsfoot roller with the thickness of layer before compaction varying from 25 to 30 cm. The degree of compaction to be achieved was specified as 98% of proctor compaction with water content varying from -3% to optimum moisture content. The field control maintained moisture content to achieve on the drier side to keep the pore pressure under control.

In order to observe the performance of the dam, piezometers and vertical settlement devices with cross-arms were installed. The piezometers consisted of nine twin tube embankment tip and installed in two rows at one section only. The USBR type tips were made of plastic stock with two porous discs fitted with inlet and outlet twin tube connections. Pore pressure measurements were recorded from a terminal well located downstream.

Piezometer readings could only be taken after one year of the completion of the construction. With the result that the construction pore pressure and its response to rise and drop of reservoir level could not be recorded. In the subsequent years the pore pressure response clearly followed the reservoir level fluctuation. A valuable data was missed due to the delay in the installation of piezometers.

Three number cross-arm systems were installed at different sections located at distances from the axis of the dam. Cross-arm pipes of 91.5cm and spacer pipes of 122cm length were chosen to get a spacing of 152.5 cm, cross-arms. To have free sliding of cross arm pipe in the spacer pipe usual precautions were taken to prevent soil getting in and locking the system. A torpedo attached to the end of a steel tape was used to measure the settlement of each of the cross-arm

The construction of the earth-rockfill portion took place from November 1960 to June 1964. The measurements of settlements continued till the end of 1968 suggested that the primary compression of the fill took place during the construction period closely following the progress of the fill construction curve. During the first three years, the reservoir level drop increased the post construction settlement. Later on the settlement remained essentially constant uneffected by the fluctuations in the reservoir level, (Char,1979). Table I shows the settlements at three different locations in the dam.

TABLE I. Settlement at Different Locations

Cross-arm location	Height of fill,meter	Settlement, percent
1	12.8	1.89
2	22.0	2.17
3	24.7	2.39

At location 3 the foundation settlement was only about 0.74%. The total settlement of the fill, 2.39%, occurred in a period of 6 years, of this 2.2% occurred during the first 2 years of the construction period. The post construction settlement of about 0.19% in 4 years is comparable with the observations from the study of USBR dams (Gould,1954) wherein it was observed that the post construction settlement was less than 0.2% during the 3 years and about 0.4% during the 14 years after the end of construction.

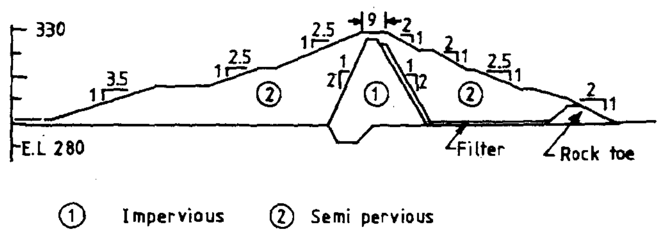

Fig.6. Cross-section of Pochampad Dam

No estimate was made in advance to establish the possible development of pore pressure using Hilf's or any other theory. Even for the settlement analysis, no tests were conducted even of oedometer type to predict the settlements. The role of foundation soil in dissipating or otherwise of the construction pore pressures could not be assumed. The coefficient of permeability of the foundation soil varied about 6 x 10^{-6} cm/sec to 41 x 10^{-4} cm/sec and appear to have been sufficiently effective in dissipating the construction pore pressure inspite of a heavy core and cut off.

POCHAMPAD DAM

Across river Godawari Pochampad dam, 36m high, was constructed; it is one of the largest dam in India. The total earthern portion is 13.64 km. The foundation soil on the flanks is essentially clay with medium to high compressibility underlain by disintegrated rock at shallow depths. The foundation treatment was essentially provision of a cut off trench excavated to the sound rock, backfilled and compacted by impervious soil. Wherever disintegrated rock still existed in the foundation grouting was adopted. A section of the earth dam is shown in Figure 6. The central impervious core with slopes of 1:2 was encased in semipervious shell zone having slopes of 2.5:1 to 3:1 on the upstream and 2:1 to 3:1 on the downstream. An inclined filter was provided on the downstream slope of the core connected to a horizontal filter. A heavy rock toe is included to support the downstream slope. The valley ratio is about 340.

The performance of twin tube hydraulic piezometers (USBR type) was somewhat erratic. As many as 65 piezometers were installed. Some of the piezometers located in the foundation showed high pore pressure to the extent of 91% of the height of the fill, (Char,1979). Much of the data is not available for a comprehensive understanding of the performance of the earthern dam.

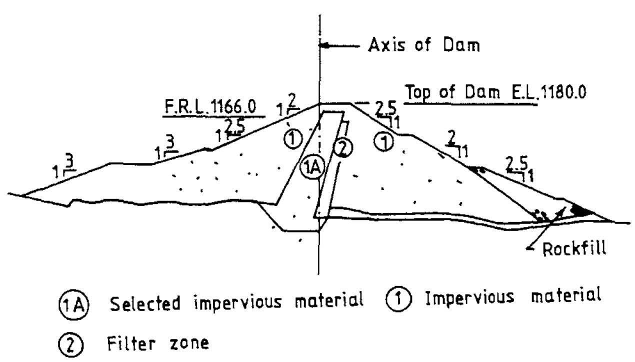

Fig.7. Cross-section of Tawa Dam

USBR type cross-arms were installed at three different sections. Two such devices were installed at each section such that one device penetrates both semipervious and impervious zones and the other semipervious zone alone on either downstream or upstream. Of the six devices installed two of them at a section did not function and measurement of deformations could not be carried out. The foundation settlement was 1.12% till the end of 1976. The maximum cumulative settlement of the fill was 3.36% which was rather high. Due to reservoir filling, water load resulted a further settlement of 0.25m. Since the valley had been flate the longitudinal displacements were negligible.

TAWA DAM

Across the river Tawa, a tributary of Narmada,an earth dam 36.6m high was constructed to flank a masonry spillway located in the river portion. Two saddles which exist on the left flank are also connected by the earth dam. The total length of the embankment is about 1.64km. The geologic formations at the site consists of sandstone and shales. Over the most part of the flanks where earth dam is located, coarse friable sandstone with thick and extensive beds of clay shales overlain by plastic/silty clay are present. Conglomorate with interbedded grits and red shale are present at the top levels on the right and left flanks. The valley ratio is about 45.

A section of the dam is presented in Figure 7, showing the distribution of various zones. The construction was completed during 1975. Out of a total of 67 piezometers, (hydraulic twin tube USBR type) 14 were placed in the foundation and the remaining in three sections of the dam. Six USBR cross-arm devices coupled with horizontal movement plates were placed at three sections. Serface settlement points (66 on the right flank and 71 on the left flank) were installed on the crest and slopes of the embankment. The earthern section consists of clay core flanked by semipervious soil with their permeabilities of 12 x 10^{-6} cm/sec and 42 x 10^{-6} respectively. The construction pore pressures were less than 20% of the fill pressure, (Verma and Raj,1978). The maximum settlements before and after reservoir impounding were 3.32% and 3.48% respectively when the core and shell zone depths were in the ratio of 2:1. In the shell portion the settlements at the end of construction was 1.84% which increased to 3.2% upon impounding the reservoir. The foundation settlements were negligible. Embankment settlements were larger than those observed in other dams.

DISCUSSION

In the foregoing a brief case study of five earth and earth rockfill dams constructed in the recent years located in the different geological settings and using different soils has been presented. In the construction of these dams thick and thin cores have been used with relatively longer construction periods adopting essentially the guidelines followed for such dams in USA. The performance of these dams has been satisfactory as per the past experience. Even in the case of Beas dam having a maximum height of 132.5 meters, the maximum vertical deformation was only about 2.5% of the height of the dam, which is lower, in comparison to EI Infiernillo dam (Marsal & Arellano,1967) wherein this deformation was of the order of 5.5% of the height of the dam. The valley ratio of the latter dam is of the order of 2.4 whereas that of Beas dam is about 17. In the case of Ramganga dam where the valley ratio was small, i.e. just over one, the vertical compression was about 1.8%. Wherever the valley ratio was large, the longitudinal deformation seems to be very small suggesting a very near plain strain condition existing over the entire height of the dam except near the abutments. When the valley ratio was small, there has been a tendency for the displacements being directed towards the centre of the valley. With the result that larger compression took place along the longitudinal axis and it varied from abutments to the centre of the valley. This definitely indicates that the stresses along the longitudinal axis of the dam have been larger than the stresses in the transfer direction. Even though adequate instrumentation has been carried out in these five dams, unfortunately, comprehensive data could not be obtained and, therefore, effective analysis could not be carried out. It is hoped, in the future dams well planned instrumentation will be carried out to improve the design and analysis and to be able to predict the performance of such structures more realistically.

CONCLUSIONS

From the study of the performance of five dams which were constructed essentially following the guidelines laid down by USBR one concludes that the construction pore pressure response has been generally low due to longer construction or shut-down periods. The deformation patterns and their limits in vertical, longitudinal and transverse directions were within the limits observed for the dams constructed in USA. It has been conclusively indicated from the measurements that compression exists along the longitudinal axis of the dam and its magnitude increases along the length of the dam (from the abutments to the centre of the valley) and also with the height. For large valley ratios these deformations suggest near plane strain conditions (as in Pochampad and Linganamakki) for smaller ratios as in the case of Ramganga and El Infiernillo dams, the compressive stress along the dam axis was more than that corresponding to plane strain condition. The distribution of this stress with height is parabolic with higher intensities towards the base of the dam. Consequently it becomes necessary that constitutive relationships and the strength parameters to be adopted for various elements in the dam vary with its lengt and height, and so these variations should appropriately be considered in the analysis to predict realistic performance of earth and rock fill dams.

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