Performance Evaluation of Rarem Dam

M. C. Goel
Investigation and Planning Circle (WB), Roorkee, India

Djoko Mudjihardjo
Institute of Hydraulics Engineering, Bandung, Indonesia

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M.C. Goel
Superintending Engineer, Investigation and Planning Circle (WB), Roorkee, India

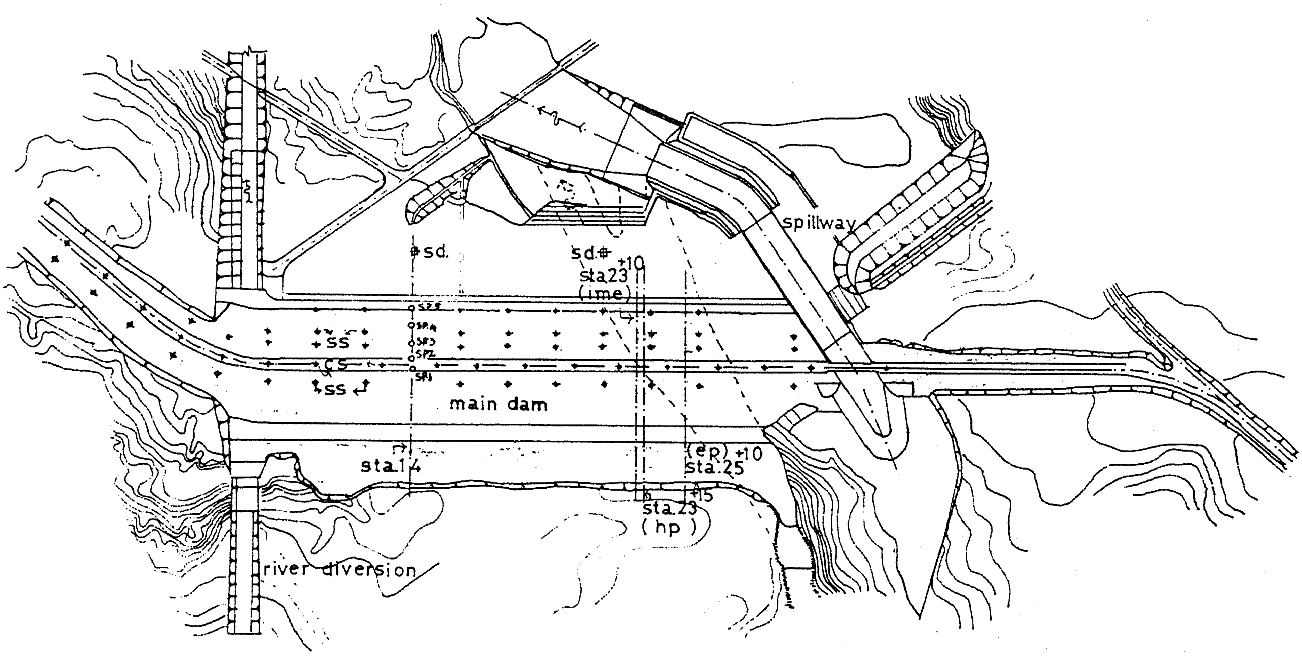
Djoko Mudjihardjo
Institute of Hydraulics Engg., Bandung, Indonesia

SYNOPSIS - 28.0 m high zoned Rarem dam in Indonesia was instrumented with hydraulic piezometers., electrical Carlson type piezometers Cassagrande type vertical stand pipe piezometers, inclinometers, and surface settlement points. The analysis of observational data has indicated that settlement took place almost simultaneously with construction of dam and reservoir filling. Very low construction pore pressures were observed and phreatic line developed almost simultaneously with reservoir filling. The results of efficiency of grout curtain based on electrical analogy model studies are also discussed in the paper.

INTRODUCTION

With a view to irrigate 22,000 ha area in North Lampung, Sumatra, in Indonesia, 62 Million Cum reservoir capacity has been created by constructing 28.0 m high and 1550 m long zoned earthen dam section at Pekurun located about twenty five kilometers southwest of town Kotabami. The construction of dam embankment was started in November 1981 and completed in early October 1983. Impounding of water into the reservoir started two weeks after completion of dam. The reservoir filling was complete by mid January 1984. The layout of works together with instrumentation plan is shown in Fig. 1. The project area is covered by young sediments of andesite tuff formation - a product of quarternary volcanic activities. The andesite tuff(bed rock) is a complex of various layers varying in thickness less than 1.0 m to 10.0 m comprising of braccia, conglomerate tuff, tuffaceous sandstone and tuffaceous clay stone. Geological profile along dam axis is shown in Fig. 2.

Vertical central clay core of Rarem dam section having upstream and downstream slopes as 0.2H:1V is protected by suitable filter transition zone. The shells are of rockfill having upstream and downstream slopes as 3H:1V and 2H:1V respectively. Typical cross-section of dam alongwith instrumentation details is shown



sp : Stand pipe piezometer ep : Electric piezometer hp : Hydraulic piezometer cs : Crest measuring point

ime : Inclinometer/magnetic extensiometer sd : Seepage discharge ss : Surface measuring point

Fig. 1 : Plan of Instrumentation Installation at Rarem Dam

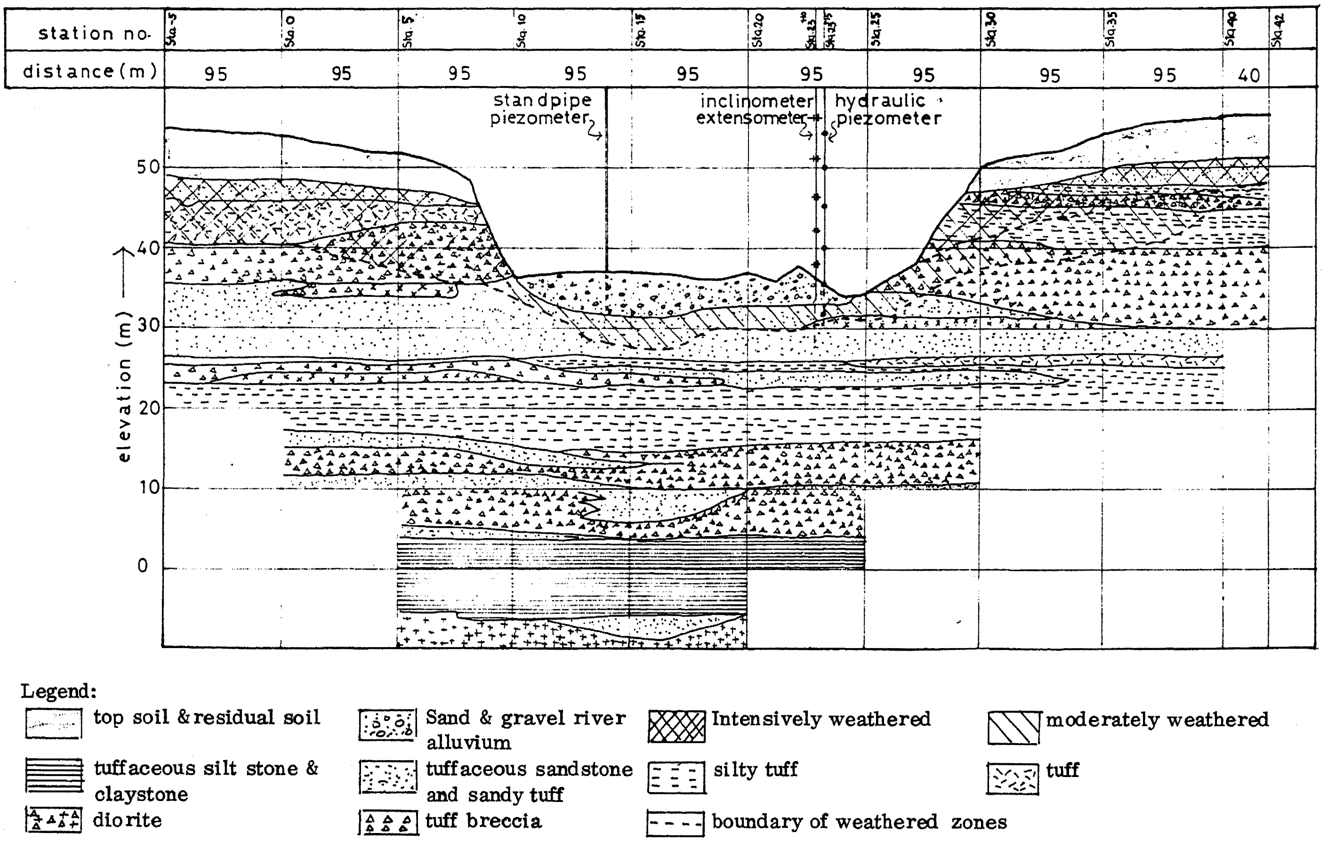


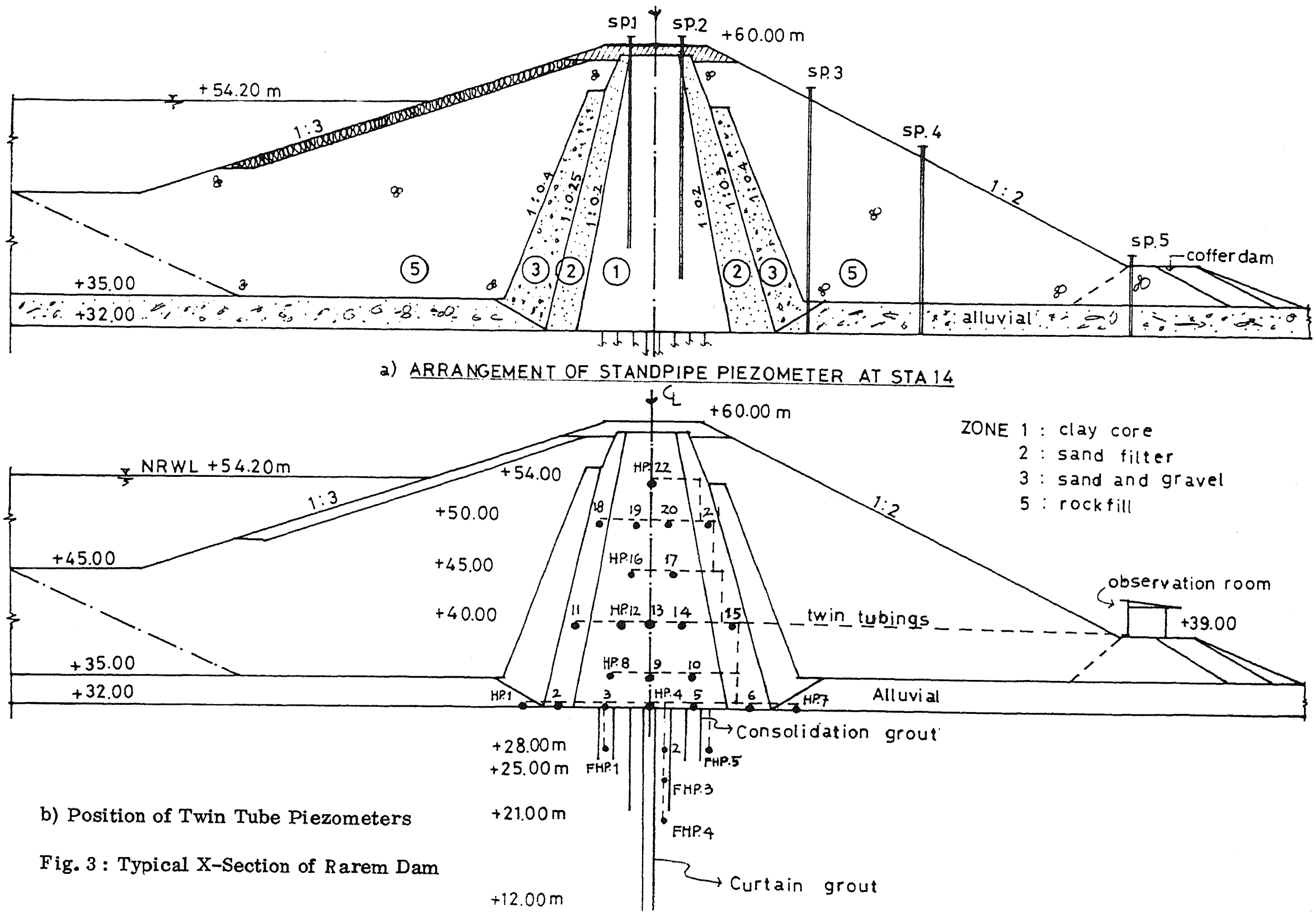
Fig. 2 : Geological Profile Along Dam Axis

Table - 1 Properties of Material

Zone	Gradation			Atterberg limit (%)			Compaction		Shear strength		Permeability
	Soil type	Particle size(mm)	% finer	LL	PL	PI	Dry density t/m^3	OMC %	Cohesion t/m^2	ϕ^o	cm/sec
Clay core zone(1)	Clay Silt Fine sand	$\leqslant$ .002 .002-.075 .075-0.45	29 64.91 7 - 9	96.4	48	48.9	1.10 to 1.12	48.80 to 50.50	1.0	28	5.9×10^{-6}
Filter zone(2)	Fine sand Medium sand Coarse sand	.075-0.45 .45-2.0 2.0-4.75	20-40 50-48 12-14				1.987	9.64	0	36	1×10^{-3}
Transition zone(3)	Sand Gravel Cobbles	$\leqslant$ 4.75 4.75-75.0 75-150	18-28 67-62 38				2.192	7.25	0	38	
Rockfill zone(5)		0.3-3.50					2.0		0	40	

in Fig. 3. Material for clay core comprises reddish brown clay dominated by Kaolinite with high plasticity belonging to CH classification of soil. Material for sand filter and sand gravel transition zone was borrowed from Rarem river deposit whereas rockfill was quarried from andesite bed from a distance of about 6 km from dam site. The properties of material used in the dam are given in Table-1.

Foundation treatment comprises curtain grout as well as consolidation grouting. Curtain grouting was done along dam areas in two rows, each 1.0 m apart in staggered position and spacing of the holes on each line was 1.5 m with depth ranging from 5



b) Position of Twin Tube Piezometers

Fig. 3 : Typical X-Section of Rarem Dam

to 20 m. Consolidation and blanket grouting was done in upstream and downstream sides of curtain grout at the bottom of cut off trench for sections between Station 1 and Sta. 28. The grout holes were arranged at 3m interval on six parallel lines, 1.5 m apart from each other. The holes were drilled upto 10m depth on the lines adjacent to the grout curtain and 5 m on other lines. Consolidation and blanket grouting was done prior to curtain grout.

2.0 INSTRUMENTATION

Rarem dam has been instrumented with the following instruments:

i) Electrical resistance Carlson type 27 piezometers at Sta. 25 + 10 to measure pore pressures.
ii) Bishop type hydraulic piezometers 27 numbers embedded at Sta. 23 + 15.
iii) Cassagrande type vertical stand pipe 5 piezometers, installed at Sta. 14, about 180 m away from the section of Bishop's type hydraulic piezometers.

Out of 27 nos. electrical piezometers, 7 were damaged during calibration stage and out of remaining 20 nos. 14 nos. became out of order during construction period. The readings obtained by these six Carlson type piezometers, were irratic and unreliable. The causes of damage were suspected as under:

i) The wire gauges were broken due to vibrations or sudden shocking or hitting during transportation from manufacturer to the site.
ii) After embedment, the seal might have leaked thus allowing the water entry into the cell causing short circuiting in the system.

Hydraulic tips were embedded both in embankment fill and in foundation of dam. Tips in foundation were installed in drilled holes while in embankment, were placed at specified locations as fill progressed. Tips were carefully saturated with deaired water and taken to the site under water. The connecting tubes which were also deaired from the observation room by flushing with deaired water, were connected to the tip under water. The location of tips in dam section is shown in Fig. 3(b).

From this figure, it can be seen that five foundation tips have been installed in tuff rock foundation. Piezometers FHP 1, 2 & 5 have been embedded at El. + 28.0 where as FHP 3 is at El. +25.0 and FHP 4 at El. + 21.0. In all 22 piezometers have been installed in embankment in 6 tiers viz. 7 nos. (HP 1 to HP 7) in first tier (El. + 32.0 m), 3 nos. (HP 8 to HP 10) in second tier (El. + 35.0), 5 nos. (HP 11 to HP 15) in third tier (El. + 40,0), 2 nos. (HP 16 & HP 17) in fourth tier (El. + 45.0), 4 nos. (HP 18 to HP 21) in fifth tier (El. + 50.0) and one piezometer HP 22 in sixth tier at El. + 54.0. Piezometers HP 1 & HP 7 are installed in alluvium, HP 11, HP 15 HP 18 and HP 21 are in sand filter zone and balance are in clay core. Though the construction of dam started in November 1981 but the installation of foundation type piezometers could be started in July 1982 after completion of grouting. By that time, the fill attained the elevation of + 37.0 m.

Stand pipe piezometers were installed by drilling holes SP1 and SP2 at the crest in core zone, SP3 and SP4 on downstream slope of rock zone and SP5 on crest of downstream coffer dam. The location of these tips is shown in Fig. 3(a). These piezometers were installed in January 1984.

Inclinometers

One set of uniaxial inclinometer/extensiometer was installed at Sta 23+10 vertically on the centre line of core zone. The instrument consisted of access tubes of alluminium with grooves in four directions perpendicular to each other, magnetic settlement plates surrounding the access tube, a torpedo with electric cable and a digital read out unit. By lowering the probe into the access tube, it can show the inclination of the access tube on the read out digital unit and the signals from magnetic plate would show the settlement. The installation was completed during 20th July 1982 to 20th March 1983.

Surface Settlement Points

For monitoring the displacement at the surface of dam, 37 points at crest and 61 points on slope of dam (24 on upstream slope and 37 on downstream slope) were provided. These points were made of suitably embedded 2.0 m long galvanized steel pipe of 100 mm and 150 mm diameter for crest and slope surface measurement respectively. On slope, these points were located in rows at approximately 40 m spacing on lines parallel to the centre line of dam at elevation + 55.0 for upstream and at El.53 and 39.00 for downstream slope.

Because of non-availability of reliable observational data(magnitude of movement being within the limit of error of the measuring method), no further analysis was done for surface movement points.

Monitoring

For observation of instruments, three periods have been specified viz. (i) loading, (ii) stabilization and (iii) maintenance. Loading test means one month prior to impounding to one year after filling the reservoir. Stabilization period follows upto four years after filling whereas maintenance period is after stabilization. The recommended frequency of observations is shown in Table-2.

Table - 2 Frequency of Observation

Measuring device	Frequency		
	Loading period	Stabilization period	Maintenance period
Piezometers	twice a week	once a week	once a month
Inclinometer	once a week	once a month	twice a year
Surface monuments	twice a month	once a month	twice a year

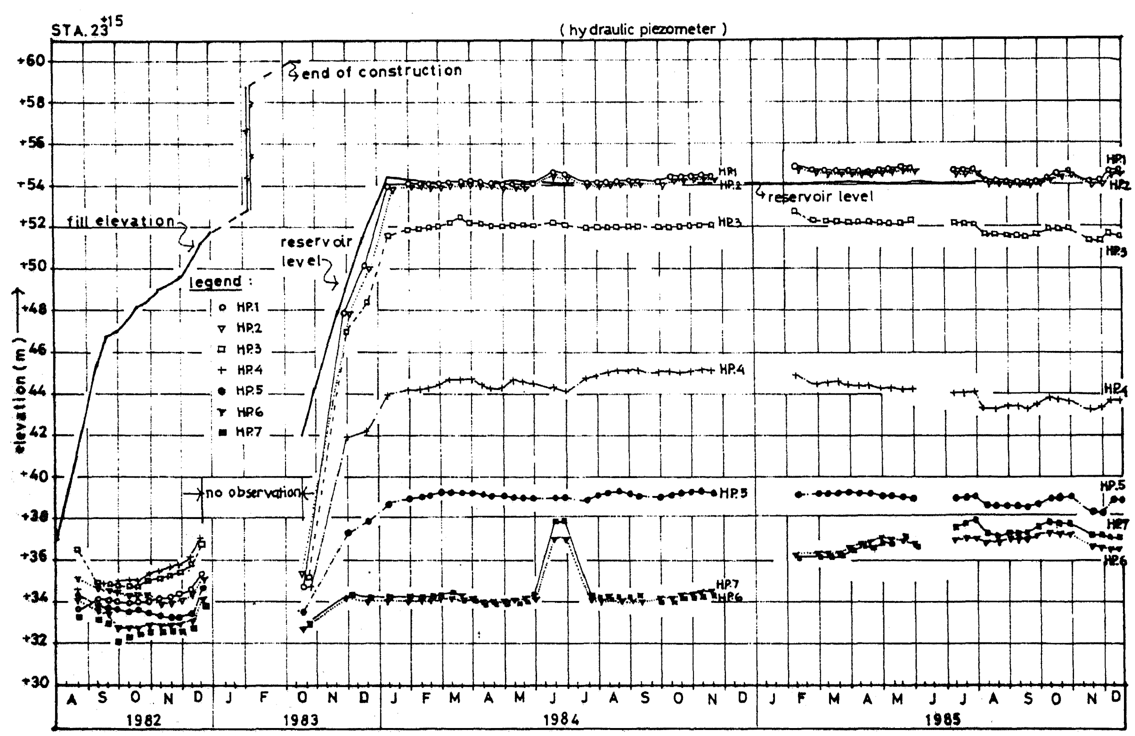


Fig. 4 : Pore Pressure Observations in First Tier

The observations on Bishop's type hydraulic piezometers were taken from August 1982 and continued upto December 1985 except a gap from January to October 1983. Readings on vertical stand pipe piezometers were started from January 1984.

ANALYSIS OF OBSERVATIONAL DATA

Construction Pore Pressures

Observed pore pressures from August 82 to December 1982 and from mid October 1983 to December 1985 for embankment piezometers HP1 to HP7 are shown in Fig.4. A study of pore pressures recorded from August 82 to December 1982, indicates that tips no.HP1 and HP2 embedded in river alluvium and filter zone respectively upstream of clay core at El.+32.0m, have recorded almost constant pore pressures of the order of 2 to 2.5m of water head. Since the water level in the river, was at El.+34.0m, obviously these tips recorded pressures corresponding to river water level. Pore pressures recorded by tips No.HP6 and HP7 burried downstream of core in filter zone and alluvium respectively, are of the order of 0 to 1 m and are thus lesser than those recorded by tips located upstream of clay core in similar type of strata. Pore pressures measured by HP3 and HP4 embedded in the clay core show that their values tend to follow the increasing trend as height of embankment fill increases. Their values vary from El. 35.0 to 37.0 m i.e. pore pressure head of 3 to 5 m of water head. At 5m pore pressure head, corresponding dam height is 19m indicating thereby development of pore pressures of the order of 0.4H only. Other tips viz. HP8 to 10, HP12 to 14 and HP16 & 17 located in clay core at different elevations have recorded nil or very insignificant construction pore pressures.

The core material consists of high plasticity reddish clay belonging to CH group having PI about 48% indicating thereby that clay core should be highly compressible and high construction pore pressures should have developed but observed data has shown low pore pressures. Had the observations continued till March 1983, the period during which the dam height was raised further by 8m, more positive conclusions could have been drawn.

Theoretical analysis by Hilf's approach(1948) using one dimensional consolidation test data, has shown that corresponding to dam height of 19m, without any drainage, the pore pressures should have been of the order of 12.5m whereas maximum observed value is about 5.0m. This shows the dissipation of construction pore pressure by 60%. Since core material consists of high plastic clay, this dissipation factor value seems to be too high specially when time duration for dissipation was only 4 months and the rate of construction was considerably high(3.25m/month). Development of low construction pore pressures may be due to fill placement on dry side of OMC(-1.5%), low placement densities(r_d=1.12 t/m^3), nearness of filter zones and possibility of arching action because of lesser thickness of core and dam being of low height.

Pore Pressures in Foundation

Pore pressure observations in FHP1 to 5 alongwith reservoir

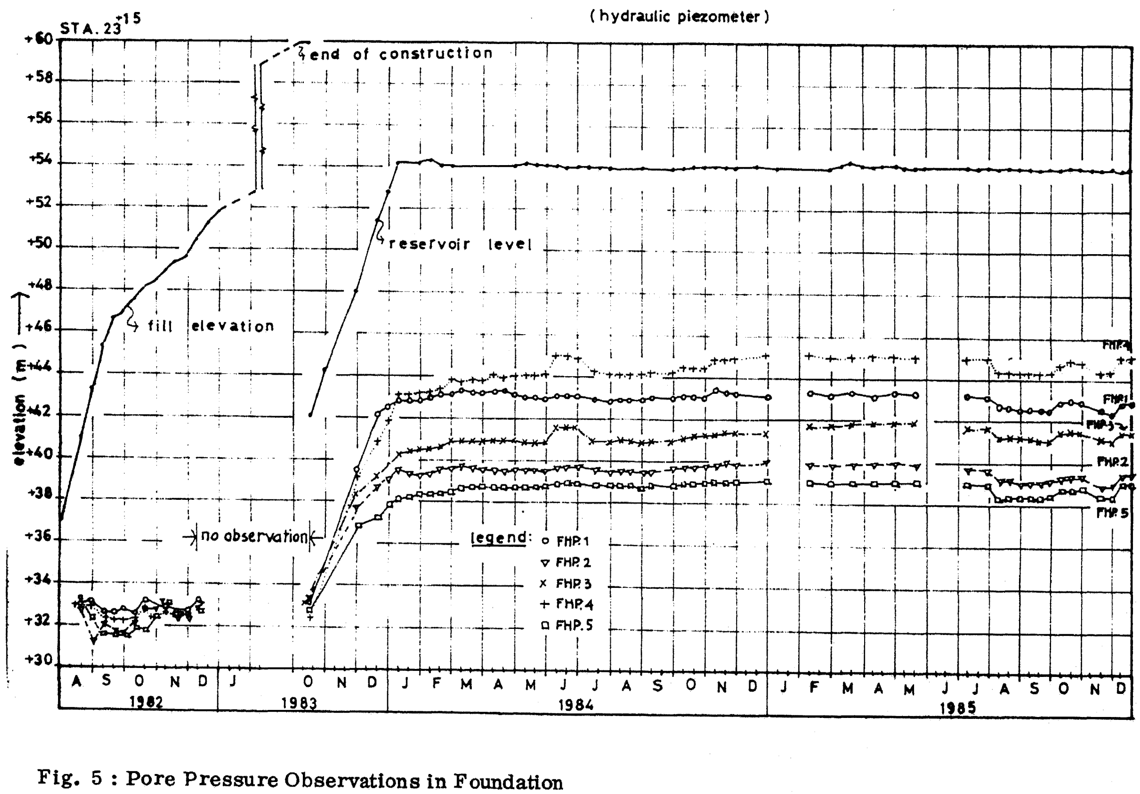


Fig. 5 : Pore Pressure Observations in Foundation

filling pattern are shown in Fig. 5. During reservoir filling period (October to December 1983), these tips have recorded increase in pore pressures following the pattern of reservoir level raising almost simultaneously. Since the reservoir level has remained almost constant from January 1984 onward, the pore pressures recorded by these five foundation tips are also almost constant during this period and the variation of the order of ± 1.0 m of water head may be due to inherent limitation of the instrumentation. Observations during and after reservoir filling indicate that for FHP1, 2 & 5 located at El. +27.0m, the pore pressures reduce from tip no. 1 to 5 indicating thereby the effect of consolidation grouting done upto El. +27.0. The pore pressures at tips Nos. FHP1 & 5 are of the order of 16m and 12m. respectively against reservoir head of 28.0 m. Tip no. FHP4 located downstream of grout curtain and at lowest level (El. +21.0m) has recorded highest pore pressures which are higher by about 2m head of water from those recorded by FHP1. Similarly tip no. FHP3 also located downstream of grout curtain at El. +25.0m, has also recorded higher pore pressures by about 2.0m as compared to tip no. FHP2. Recording of higher pore pressures by tips no. FHP3 & 4, is contrary to the expectations.

Effectiveness of Grout Curtain

The effectiveness of grout curtain is judged by the reduction of seepage discharge (Cassagrande 1961). In the absence of any data regarding seepage quantity, the efficiency of grout curtain was determined from decrease in pressure heads (Hari Krishna and Goel, 1978) using following formula -

$$\text{Efficiency} = \left(\frac{P_1 - P_2}{P}\right) \times 100 \text{ (in \%)} \qquad (1)$$

where

P_1 & P_2 are the pressure heads upstream and downstream of grout curtain respectively.

P is effective reservoir head.

The efficiency of grout curtain has been calculated by taking total head corresponding to observed piezometer head in tip no. FHP1 and tail water level as recorded in a stand pipe (SP5) embedded at toe of dam at Sta. 14, which is about 180 m from the section of these piezometers. The recorded tail water level is at El. +32.50m and recorded pore pressure in FHP1 is at El. +43.30m. Based on this approach, the results of calculations of grout curtain efficiency are shown in Table-3.

Table 3 - Grout Curtain Efficiency Based on Observed Data

Elevation	Tip No.	Obs. pore pressure (m)	P_1-P_2 at El. 28.0	Efficiency (%)
+28	FHP1	15.3	0	0
+28	FHP2	11.5	3.8	35.19
+28	FHP5	10.5	4.8	44.45
+25	FHP3	15.7	2.6	24.07
+21	FHP4	23.2	-0.9	-8.33

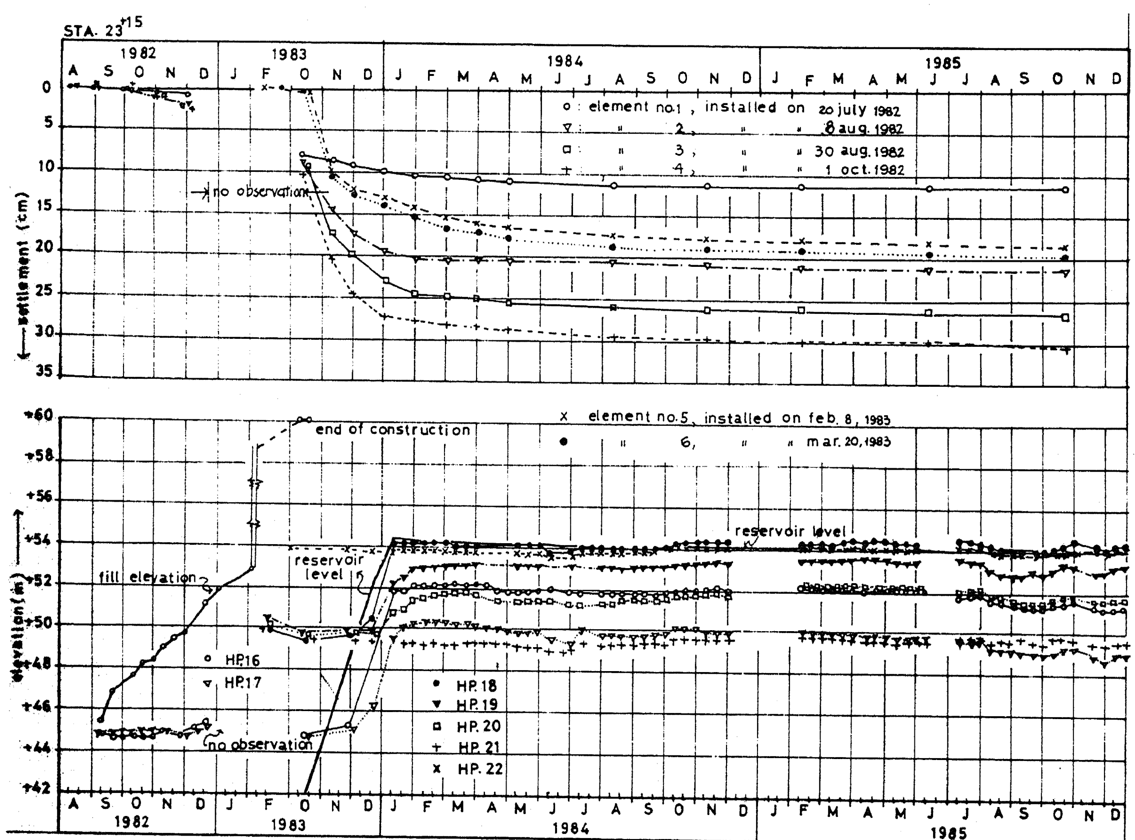


Fig. 6 : Pore Pressure and Settlement Observations

Since the base of core has been provided with consolidation grouting and thus, it is a case of multiple permeabilities in zoned dam section and foundations, and the excess recording of pore pressures by FHP3 & 4 is not explained, electrical analogy experiments to simulate the anisotropy in permeability, were conducted.

Steady Seepage Pore Pressures

A study of pore pressures recorded by tips nos. HP1 & HP2 located in upstream sand filter at El. +32.0m shown in Fig. 4 & that of tip no. HP18 again located in upstream sand filter at El. +50.0m (Fig. 6) indicates that the pore pressures are rising with reservoir filling with no time lag. This indicates that upstream filter is working satisfactorily. Recording of negative pore pressures by tip no. HP21 situated downstream of core in filter chimney at El. +50.0m (Fig. 6) and that of tip no. HP15 at El. +40, confirms that downstream filter chimney has been able to lower the phreatic line within core of the dam. Pore pressures recorded for tips no. HP6 and HP7 located downstream of core in filter are of the order of +2.0m (El. +34.0m) upto November 1984 but thereafter the pressures have further increased by 2 to 3 m and by end of December 1985, the excess pore pressures are of the order of 5m.

A study of pore pressures recorded by piezometers no. HP3, HP4 and HP5 (Fig. 4) located at tier no. 1 at El. +32.0m, HP8, HP9 and HP10 at tier no. 2 at El. +35.0m, HP12, HP13 and HP14 at tier no. 3 (El. +40.0m) (plot not shown in this paper), HP16 and 17 at tier no. 4 at El. +45.0m and HP19 and HP20 at tier no. 5 at El. +50.0m (Fig. 6), embedded in clay core, reveals that during water impounding period, pore pressure increased simultaneously with reservoir filling with practically no time lag. A close look of the pore pressures recorded by these tips indicate that from January 1984 onward when the reservoir has been filled upto El. +54.2, the readings of pore pressures of all the tips are almost constant and the steady seepage pore pressures developed follow the flownet pore pressure pattern.

As expected, vertical stand pipe piezometers SP3 to SP5 located in downstream shell of dam section, have not recorded any pore pressures thus indicating that downstream shall is dry and chimney filter is working satisfactorily. Piezometer SP2 located in clay core at El. +37.0 has recorded pore pressures of the order of 7.0m (El. +44.0m). Hydraulic piezometer tip no. HP10 located at El. +35.0m and almost at the location of SP2, has also recorded pore pressures of the same order (El. +44.0m). Thus, the observational data of hydraulic twin tube Bishop's type piezometers is confirmed by the observational data of Cassagrande type porous tube vertical stand pipe piezometers. Tip no. SP1 has been reported to be chocked and hence no reliable data is available from this tip.

Equipore pressure contours for readings taken after fifteen months of reservoir filling (March 1985) are shown in Fig. 7. A study of this figure would indicate that the effect of anisotropy in the permeabilities of clay core, foundation treated by consolidation grouting and that of untreated foundation, is noticeable.

Free surface line as obtained from observations of twin tube hydraulic piezometers and that obtained by vertical stand pipe piezometers no. SP2 is shown in Fig. 7. From this, it can be seen that free surface line obtained by twin tube piezometers is higher as compared to that given by vertical stand pipe no. SP2. Free surface line has also been drawn by L. Cassagrande's approach presuming foundations to be impervious. Free surface line with this method lies in between the two observed free surfaces. Free surface line has also been drawn by electrical analogy method taking permeability of clay core, consolidated grouted foundation zone and ungrouted foundations as $6x10^{-6}$, $6x10^{-4}$ and $1x10^{-3}$ cm/sec. respectively and the same is also shown in Fig. 7. It is seen that the free surface obtained by electrical analogy, coincides fairly well with the free surface obtained by twin tube piezometers. Thus the observational data of Bishop's type piezometer can be treated as reliable.

Vertical Movement

A study of Fig. 6 showing settlement data, indicates that during construction period (August to December 1982), the settlement recorded, varied from 0.5 cm to 2.5 cm, the smallest value was recorded by settlement plate no. 1 embedded at bottom most level at 32.773 and highest by plate no. 4 at El. +46.736 m. No significant settlement was recorded by plates no. 5 & 6 from their installation in February and March 1983 until October 1983 when reservoir filling started. At the end of construction in October 1983, the settlement of the order of 8 cm to 10.5 cm was recorded by plates no. 1, 2, 3 and 4.

During reservoir filling from October to December 1983, a steep rise in settlement is recorded by almost all plates. In case of plate no. 4, the settlement increased from 10 cm to 27 cm. For plate nos. 5 & 6 (El. 50.90 and 56.411m), increase in settlement was of the order of 12-13 cm by end of December 1983. Plates no. 1, 2 (El. 37.439) and no. 3 (El. 41.924), indicated rise during this period from 8.5 to 10 cm, 9 cm to 20 cm and 10 to 23 cm. respectively. This shows that depending upon the location of plate, maximum settlement has been recorded during reservoir filling which is of the order of 13 to 17 cm for plates located in

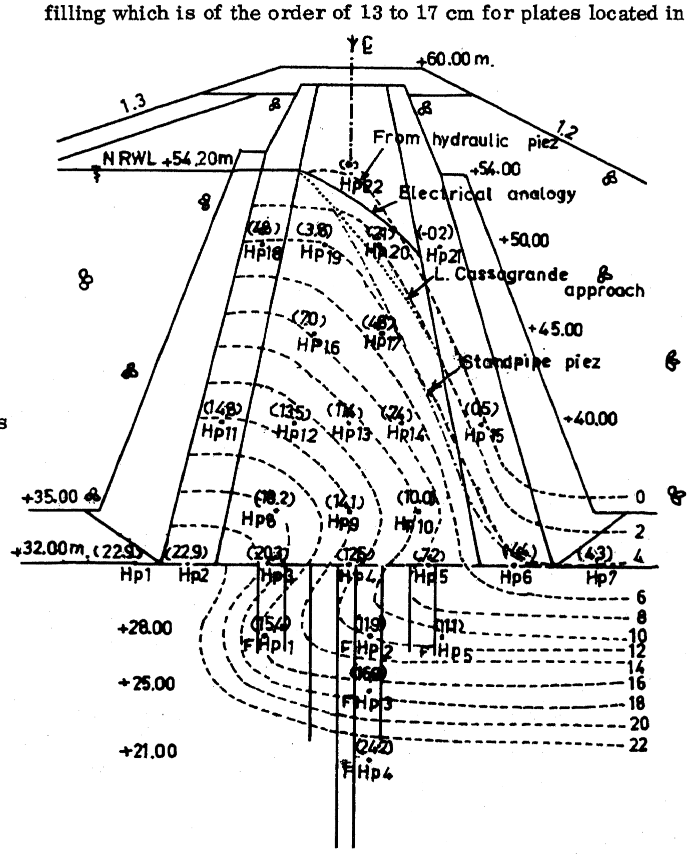


Fig. 7 : Equipore Pressure Contours and Phreatic Surface

upper height of dam. Non recording of settlement by top plates nos. 5 and 6 during construction period (March, September 1983) may be due to some arching action of core by adjoining shell zones which might have reduced during reservoir filling because of lubrication due to saturation and thus during reservoir filling, settlement is recorded by these plates as well. It is interesting to note that for plates no. 1 to 4, settlement after completion of filling (December 1983 upto October 1985) has increased only by about 2 to 3 cm whereas in case of plates no. 5 & 6, the settlement recorded in post filling period is of the order of 5 to 6 cm. This shows that from the date of starting reservoir filling, the total settlement recorded by plates nos. 4, 5 & 6 is of the same order i.e. about 19-20 cm. It may further be noted that beyond April 1984, settlement recorded is practically negligible and thus the settlements are also taking place almost simultaneously or with little time lag.

Theoretical settlement calculations based on Terzaghi's one-dimensional consolidation theory were also done taking coefficient of consolidation c_v value as 6×10^{-3} cm^2/sec, initial void ratio e_0 as 1.407 and compression index c_c as 0.118 determined from laboratory tests. The analysis has indicated that for 90% consolidation, total settlement of 53 cm is anticipated in a period of 18 years where as for 50 percent consolidation 29 cm shall take 4 years period. The total observed settlement is of the order of 30 cm which is almost 50 percent of estimated total settlement. This has been attributed to possible deficiencies in experimental results and quick settlement is due to, two dimensional effect in the prototype.

Horizontal Movement

Horizontal movement was measured by a torpedo moving in four mutually perpendicular grooves in alluminium casing of inclinometer. Two grooves are aligned in upstream an-d downstream direction and the other two in diversion canal and spillway direction. Fig. 8 shows a plot in between observed displacement and depth measured initially on 16th November 1982 on the completion of dam and continued untill 25th Sept. 1985. During reservoir filling period at El. 54.20, maximum displacement of 4.1 cm (5.7-1.6) in diversion canal direction at about 14 m depth, was recorded on 25th Sept. 1985 and 6 cm(4+2) deformation in downstream of dam direction at about 17 m depth was also measured on 29th October 1984 and 27th March 1985. This shows that displacement of dam is taking place both along the dam axis as well as in upstream downstream direction.

A close look of Fig. 8 indicates that height wise, there are three distinct regions so far as horizontal movement is concerned. One is from top of dam to 4 m depth, second is 4 m to 10 m depth and third is 10 m depth to bottom most anchor point in foundation. In second zone, there is practically no movement either along the axis or perpendicular to it. In first region (top 4.0m), downward movement of about 1.5 cm is recorded after 4 months of reservoir filling but it has returned to normal position by Sept. 1985 confirming that there is no downward movement. Similar is the trend for movement along dam axis in this region. However, in region 3 (10 m from top until foundation), maximum downward movement of the order of 4 cm from normal has been recorded after 10 months of filling (29th October 1984) which has subsequently reduced to 2.0 cm after one year (29th Sept. 1985). The movement towards diversion canal is increasing with time and has attained deflection of the order of 6.0 cm.

Normally no lateral movement should be expected and the behaviour of inclinometer readings in third zone needs further investigations.

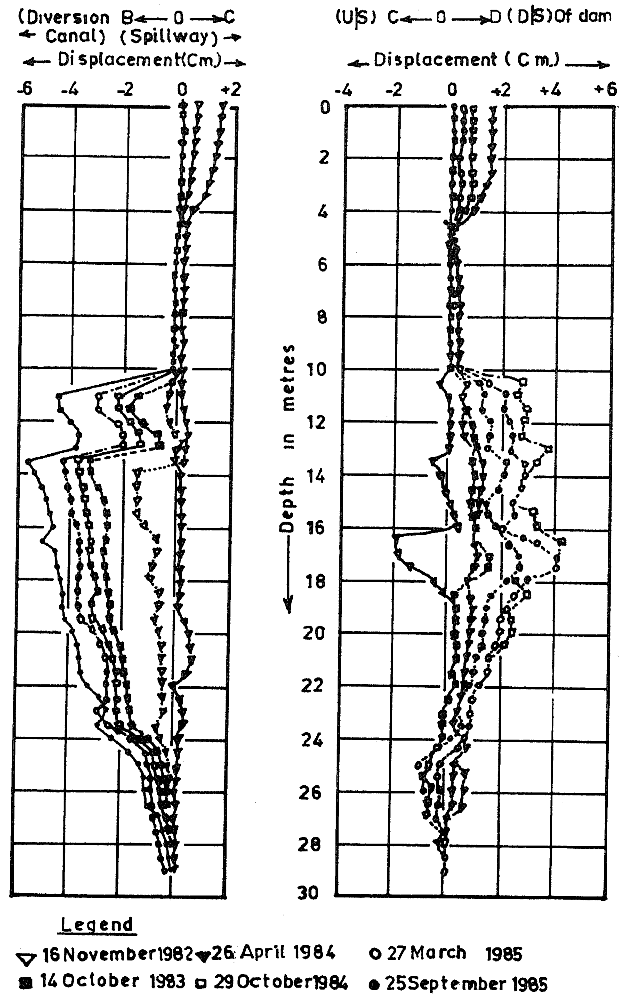


Fig. 8 : Observational Data for Horizontal Movement

ELECTRICAL ANALOGY EXPERIMENTAL RESULTS

Experimental Set Up

With a view to find answer for the falacy in the observational data of foundation piezometers FHP1 to FHP5 and to determine the efficacy of grouting, electrical analogy experiments using conductive tray were performed. The conductivities of four different electrolytes representing four different permeabilities in four zones (Table-4) were obtained by mixing sodium chloride in distilled water or tap water depending upon the range of conductivity. It was observed that maximum change in the conductivity of electrolite for clay core prepared in distilled water, was 7.4% during experimentation and thus electrolyte can be said to have remained stable even though the range of maximum conductivity was 300 times that of minimum tested conductivity.

The tests were conducted for six conditions of grout curtain efficiencies viz. (i) 100% effective, (ii) 75% effective, (iii) 50% effective, (iv) 25% effective, (v) grout curtain completely ineffective and (vi) grout curtain as well as consolidation grout completely ineffective. First case was represented by using perspex sheet showing grout line whereas for fifth and sixth case, no perspex sheets were used. For other cases, corresponding percent area of perspex sheet was perforated to

Table 4 - Permeabilities and Conductivities of Different Zones

Zone	Permeability(k) (cm/sec)	Relative K	Conductivity micro mho
Clay core	$6x10^{-6}$	1 K	$5.0x10^{2}$
Consolidation grouting	$5.6x10^{-4}$	100K	$5.0x10^{4}$
Foundation	$1x10^{-3}$	200K	$10.0x10^{4}$
Downstream filter	$1.8x10^{-3}$	300K	$15.0x10^{4}$

Table 6 - Model Test Results with Variable Permeabilities in Consolidation Grouting Zone

Tip No.	El.	Pore pressure of Relative Permeability of consolidation grout zone(%)				Observed pore pressure
		10K	50K	100K	200K	(%)
FHP1	+28	91	91	90	87	51
FHP2	+28	50	54	55	50	35
FHP5	+28	14	14	15	13	30
FHP3	+25	37	42	45	39	43
FHP4	+21	36	38	46	35	53

Table 5 - Percent Grout Efficiency and Percent Observed Pore Pressures for Foundation Tips FHP to 1 FHP5

Tip No.	Elevation	Observed Pore Pressure March 1985		% Pore Pressures from Model Experiments Efficiency					
		(m)	(%)	100%	75%	50%	25%	No curtain	No curtain, No consolidation
FHP1	+28	15.6	51	90	89	90	90	88	89
FHP2	+28	12.0	35	55	58	60	70	74	79
FHP5	+28	11.0	30	15	17	20	25	36	65
FHP3	+25	16.8	43	45	62	65	70	76	80
FHP4	+21	24.0	53	46	58	60	70	80	82

represent the efficacy of grout curtain. For example for case (ii), to represent 75% effectiveness of grout curtain, 25% area of perspex sheet was punctured by equi-spaced small holes.

Analysis and Comparison of Experimental Data

Electric potentials of grid points were obtained using wheat stone bridge for all the six cases and equipotential lines were drawn(not shown here because of paucity of space). From these equipotentials, percent potentials at location of piezometers FHP1 to FHP5 were determined for all six cases of grout efficiency. Using observational data on prototype of these piezometers, the relationship of percent grout efficiency and percent pore pressures are worked out in Table-5 and plotted in Fig. 9. The model test results shown in Table-5, are for relative permeabilities of clay core, consolidation grout zone and foundation zone as K, 100K and 200K respectively.

From Table 5, it is seen that percent pore pressures of FHP1 from model studies are almost constant for various cases of grout efficiencies and are of the order of 90% irrespective of the fact that even if the grout curtain as well as consolidation grout is fully ineffective where as observed pore pressure in field is only 51%. This observed value seems to be too low and is not explainable even by model test results. Recording of low observed values may be either due to malfunctioning of tip or due to creation of some what impervious zone due to grouting around the tip. Observed pore pressure of FHP2 located downstream of grout curtain, is of the order of 35% whereas those observed on model, vary from 55% to 79%. The pore pressures as observed in the model are increasing as the efficiencyof grout curtain is increasing. Reasons for low observed pore pressures record ed by tip no. FHP2 may be the same as for FHP1. Perhaps consolidation grout is very effective below dam core. Observed pore pressure of piezometer no. FHP5 located at same elevation as that of tip no. FHP1 and FHP2 (+28.0m) is of the order of 30% whereas those observed on model, vary from 15% to 65%. If it is presumed that actual field observation is correct, it may be concluded that grout curtain is efficient only to the extent of 21%

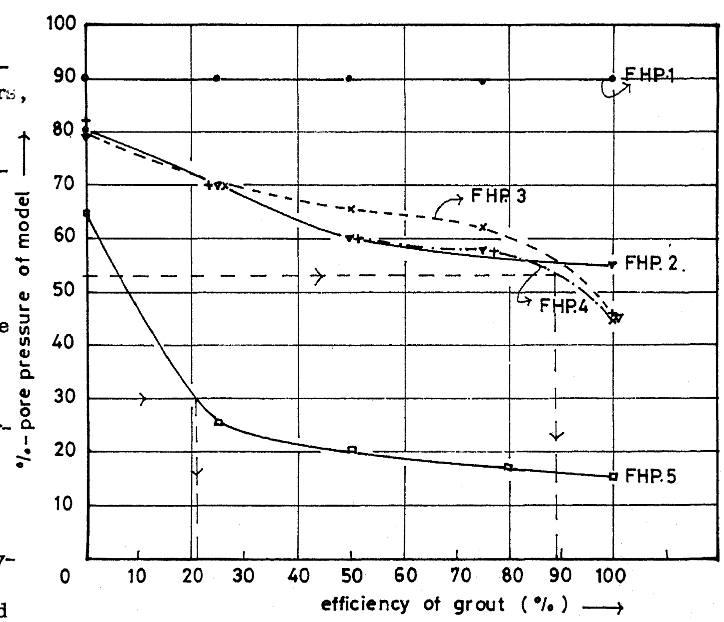


Fig. 9 : Grout Efficiency vs Percent Pore Pressure

(Fig. 9). Reason for higher observed pore pressures in this tip, may be poor quality of consolidation grouting in this region. Permeability of foundation varies from $7x10^{-5}$ cm/sec. to $0.84x10^{-3}$ cm/sec and the assumption of average permeability of consolidated area as $6x10^{-4}$ cm/sec(100K) uniformly throughout, may not be correct.

To explore this matter further, model studies were extended with relative permeability of 10K, 50K, 100K and 200K in consolidation grout zone and 100% grout curtain efficiency and the results of these studies are shown in Table-6.

From Table 6, it can be seen that percent pore pressures from test data for all the tips are practically of the same order for different permeabilities in consolidation grout zone indicating thereby that if permeability in consolidation zone is taken uniform, by and large, there is no significant change in pore pressures recorded by foundation piezometers.

Percent pore pressures measured by piezometer tip no. FHP3 from model increases from 45 to 80% as the grout curtain efficiency decreases (Table-5) whereas observed pore pressure is 43%. Thus it can be said that efficiency of grout curtain is almost 100%. Similarly percentage of pore pressure measured by FHP4 (located also in the downstream of grout curtain at El.+21.0) from model studies, increases from 46 to 82% with decreasing efficiency of grout curtain. Observed pore pressure is 53% which corresponds to 89% efficiency of grout curtain (Fig. 9).

Keeping in view the readings recorded by tips in foundation, it can be said that both grouting viz. consolidation as well as curtain, are by and large successful at Rarem Dam.

CONCLUSIONS

Based on the observational data of instrumented Rarem dam and the electrical analogy model studies, following conclusions are drawn:

i) Bishop's type twin tube hydraulic piezometers are more sturdy and reliable as compared to Carlson type electrical resistance and strain gauge type piezometers.

ii) At Rarem dam, the recorded construction pore pressures are of very low magnitude even though the core comprises of highly plastic clay.

iii) The steady seepage pore pressures have developed almost simultaneously with reservoir filling with practically no time lag.

iv) Major settlement has coincided with construction of dam and reservoir filling. Total settlement of 30 cm has taken place which is 50% of anticipated settlement. No futther settlement is expected in future.

v) There is no movement in direction perpendicular to dam axis. Whatever initial movement was there during initial reservoir filling, the same has been recovered back. However, there is some evidence of little movement along dam axis towards diversion canal side.

vi) Electrical analogy model studies coupled with observational data of piezometers in foundation, have indicated that grouting done at Rarem dam is effective.

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