Geotechnical Problems and Performance Studies - Chilla Power Scheme, Hardwar

D. N. Bhargava
U.P. Irrigation Research Institute, Roorkee, India

Dwarka Nath
U.P. Irrigation Research Institute, Roorkee, India

S. N. Kapoor
U.P. Irrigation Research Institute, Roorkee, India

Sunde Singh
U.P. Irrigation Research Institute, Roorkee, India

Geotechnical Problems and Performance Studies - Chilla Power Scheme, Hardwar

D. N. Bhargava
Director, U.P. Irrigation Research Institute, Roorkee

Dwarka Nath
Research Officer

S. N. Kapoor
Assistant Research Officer

Sunder Singh
Research Supervisor

SYNOPSIS Thin plastic clay seams existing in the upper Shivalik formations caused hazardeous geotechnical problems during the construction of 144 MW capacity Chilla Power House Scheme, Hardwar, India by initiating several land slides. In addition, serious seepage problems occurred with the commissioning of the scheme. The paper describes in detail, the events of slides & seepage problems faced and the remedial measures adopted to counteract them. The data observed on the instruments installed to keep a vigil on the performance of the structure has also been analysed and discussed.

INTRODUCTION

The Chilla Power House Scheme has been constructed during the years 1973-80 in the sub-mountaineous region of the Himalayas with power house sited 9.0 km. north of the holy city of Hardwar (U.P.), India and a barrage about 14.5 km. further up near Rishikesh on River Ganges. The power house utilizes a drop of 33m. for operating four units of 36 MW capa-city each. The general ground level at the power house site varies from 305 to 310m.while the foundation level in unit bays and intake structure are 277.5 and 304.424 m. respectively. The 14.5 km. long lined power channel with discharging capacity of 567 m³ has a bed level of 321.5 m. and pond level of 330.0m. in the forebay channel. The location of barrage,power channel and power house is shown in Fig. 1.

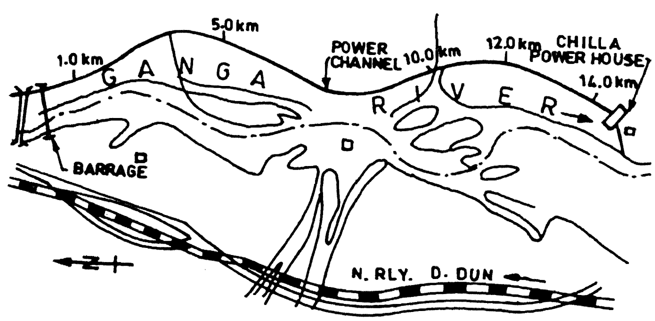

Fig.1 : Index Plan

TOPOGRAPHY & GEOLOGY

The entire scheme is located in upper Shivalik region of the Himalayas. The power house is situated at the base of a hillock with ground elevations (305-310 m.), gently rising at an angle of 8° to about 340 m. The foundation strata comprises characteristic clay shale interspersed with pervious bands of coarse sand, gravel and pebble dipping at an angle of 10-15° towards the power house. The saturated gravelly bands are erratically deposited in the area. Unit 1 and 2 are founded on clay bands whereas unit 3 and 4 are on sandy gravels. Layers of thin plastic clay seams daylighted (CCC-1977) at El. 304.0 and 300.5 m. on the penstock slopes and at El. 300.4 m. in the power house pit during their excavation. The orientation of these seams are shown in Fig.2

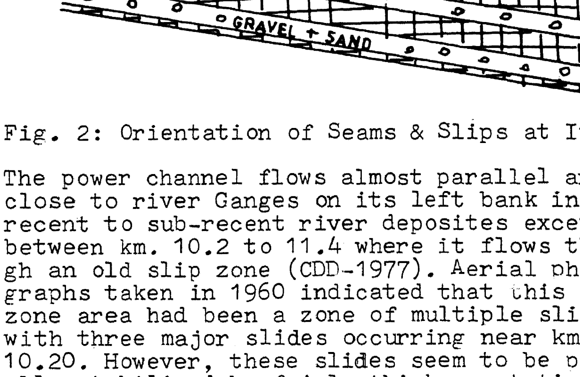

Fig. 2: Orientation of Seams & Slips at Intake

The power channel flows almost parallel and close to river Ganges on its left bank in recent to sub-recent river deposites except between km. 10.2 to 11.4 where it flows through an old slip zone (CDD-1977). Aerial photographs taken in 1960 indicated that this slip zone area had been a zone of multiple slides with three major slides occurring near km. 10.20. However, these slides seem to be partially stabilized by fairly thick vegetation as well as by drainages developed over them. In the head reach, the channel is in deep cutting after which the channel alignment is such that either the left bank is in cutting and right bank in filling or both banks are in filling or in cutting. Open pit excavations on the right, left and central line of the power channel indicated that the strata comprised erratically deposited layers of clay mixed with silt interspersed with layers of boulders, sand etc. These sequences varied with depth and chainage. Like power house site, layers of thin plastic clay seams daylighted at various elevations along the channel and were found dipping from left of power channel towards right in slopes varying from 6:1 to 10:1 (CCC-1978). Later during excavations of the right bank, inverted subsidiary seams were also found to exist on right bank of the channel. These seams were dipping from right towards left at a fairly steep angle of 45°. The disposition of the seams is shown in Fig.3.

along the slopes, a second slip occurred (C 1977) on 10.10.76 which enlarged the zone o previous slip on the upstream side. The sli ed mass moved towards the downstream side b about 2 m. in a period of about one week. I was then decided to regrade the overall slo to 2:1 with benches at El. 309.3 and 304.4 so as to minimise the earth pressure on the upstream side (Fig. 2). The easing of earth slope was started from the top and proceede downwards.

The power channel in reach km. 7.5 to 11.5 runs adjacent to a ridge and excavated eart was utilized for the construction of right embankment. When the excavation in the chan bed had gone down nearly to design level du

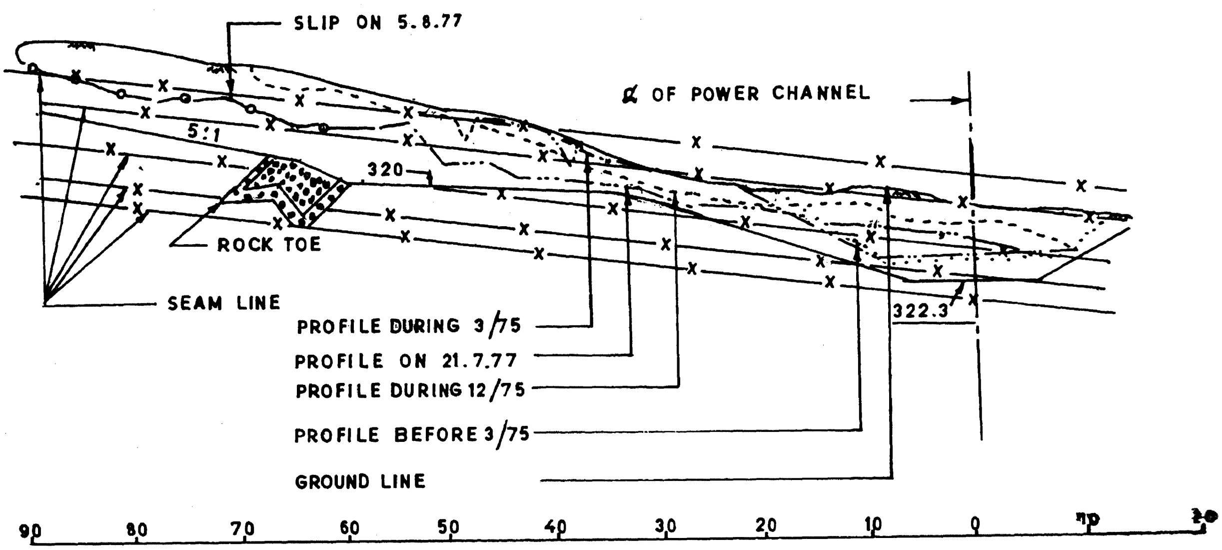

Fig.3 : Disposition of Seams & slips at power channel

GEOTECHNICAL PROBLEMS

The clay seams present at power house site acted as potential slide planes (Henkel-1967) and actuated a number of slips. The excavations for the power house structure involved about 2,70,000 m^3. of earth work and were started in Oct. 1973 with side slopes of 1/2:1. The foundation level of 277.50 m. was obtained just before the monsoons of 1975. During monsoons a part of the slope upstream of the pit between El. 295.0 and 310.0m. (Fig.2) slided down and filled the excavated pit (CCC-1977). The removal of this slipped mass was soon started and continued till Dec. 1975. Simultaneously, excavations for the intake structure were carried out, but only units 3 and 4 could be concreted before start of monsoons of 1976. On 13.7.76, when upstream key of units 1 and 2 were being prepared for laying concrete a land slide took place at El. 320.0 m. covering a width of about 37 m. across and 10 m. along the flow. As a result, the key and the foundation got filled with the slipped mass and slope sank down from El. 320.0 to 317.0 m. vertically (TDDC-1977). In Oct. 1976, when the work of removal of this slip was in progress

ing Feb. 1975, some water was observed oozi out from left cut clope at El. 324.0 follow by a major slide in March, 1975 (CCC-1976). manifestation of the slips were in the form the upheaval of the excavated channel bed b about a meter between km. 11.287 and 11.383 with subsidence accompanied by extensive cr cking of the ground 50 m.away on the left s of the centre line of the power channel.Lat the side slopes were regraded to a slope of so that the slope may be stabilized. The wo was started from top El. 352.0 m. downwards with a 2.0 m. wide berms at every 5.0 m. de The excavation reached to El. 322.0 m. by 21.7.77; but during rains the slip again oc rred on the slope with subsidence and crack extending upto the bed of channel. In April 1978 when excavations (CCC-1978) of the rig bank near the inner toe between km. 11.30 t 11.375 was in progress, some lateral moveme accompanied by upheaval took place in the b of the channel on the right side. This resu ted into development of cracks and ultimate in failure of the right bank. These cracks ter increased in width and depth from km. 11.300 to 11.400 and the centre line pillar in the bed near km. 11.325 heaved up and mo by about 1.6m. towards left on right bank.This failure was contrary to any anticipation and were attributed to the presence of thin inverted subsidiary seams running from right to left on right bank located subsequently. The profiles of slips in different dates are shown in Fig. 3.

During filling, the power channel in reach km. 7.5 to 11.5 exhibited serious seepage troubles on the right bank where it is in heavy filling with natural drainages, depressions, slushy regions with high spring level on area away from the toe of the bank. A regular seepage drain connecting these natural drainages by cross drainage was constructed to carry seepage water from rock toe and natural drainages to River Ganges. The power channel is lined by tile lining and vertical pressure release valves have been provided. The circular valves are made of plastic having internal diameter of lid as 75 mm. Before filling of channel the discharge in the seepage drain was of the order of 0.370 cumec. With the first trial filling of the channel on March 21,1980, the discharge increased to 0.59 cumec. On 28.3.80, when water level in the channel reached El. 325.80m. followed by dangerous boiling conditions with movement of fine particles in depressions. In addition, heavy seepage was also observed at km. 14.0 where the power channel is in cutting on its right bank through a natural ridge. These conditions endangered the safety and stability of banks of power channel and other structures. Further filling of the power channel was stopped and the remedial measures adopted.

REMEDIAL MEASURES

Soil samples both disturbed and undisturbed were collected from various points along the intake slope and power channel and were tested for various engineering properties. To find out sliding friction between clay seams and clay band material, direct shear tests were carried out (IRI-1977) in 6 cm. size box by keeping the contact planes between the two materials coinciding with the shear plane of the shear box. As the clay at power channel site was fissured, block shear tests (IRI-1977) were also conducted at site on 60x60 cm.blocks under drained conditions. The test results are tabulated in Table I.

Measures Adopted to Check Sliding

Since the clay seam material had very low shear strength,when saturated, the stability (TDDC-1977) of penstock slopes and intake structure was evaluated by assuming clay seams daylighting at El. 300.4, 296.5 and 286.6 m. as slip planes. Pore pressure was assumed to vary linearly (TDDC-1977) from El.321.0m. upstream of intake block (water level during monsoon period) to zero where clay seam was daylighting on the downstream side, thereby presupposing 100 per cent drainage. The following remedial measures were adopted to check further sliding (TDCC-1977):

(i) The intake slope was eased to 2:1 from 1/2 :1 and the nonflow portion was loaded by a minimum of 2m.of concrete. Elaborate drainage was provided in the form of vertical sand drain of 12.5 cm. dia.located 1.5 m. C/c to 3m. C/c deep enough to pierce the two clay seams and penetrate well into the pervious strata and connected with horizontal drains and filter.

(ii) The material upstream of intake block upto second clay seam was excavated and a RCC retaining wall constructed to avoid transfer of stresses of the backfill to the intake structure and to hold it back by means of steel anchors against sliding along clay seams(Fig.4).

At the slip zone site of the power channel,the steps taken to check sliding ware as under (CCC-1978).

(i) The ridge slope from road level of 332.0 to El.354.0 m. to 5:1 was almost the same as bedding plane mapped by geologist and line of disturbance seen in the field. There after the slope upto road level of 332.0 m.was regraded

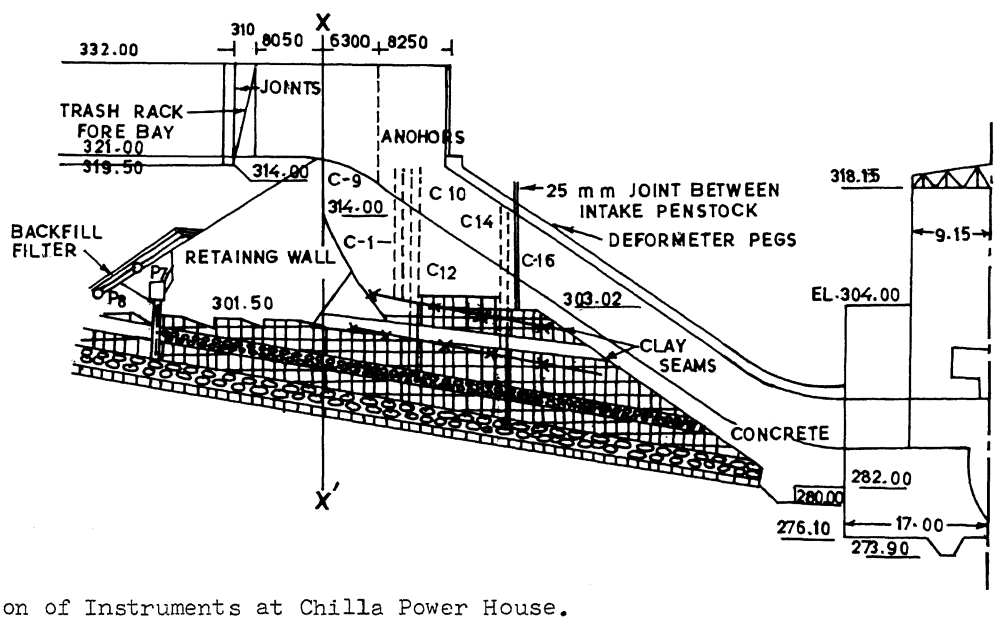

Fig.4: Location of Instruments at Chilla Power House.

to 3:1 (ii) Filter toes were provided at the road level 332.0m.and at El.344.0m. with adequate drainage arrangements. (iii) The seam was completely excavated from the portion

Table I - Summary of Test Results

Location	IS Classification	Clay %	PI %	Sr %	Undrained Strength Peak C kg/cm^2	Undrained Strength Peak Ø deg.	Undrained Strength Residual C kg/cm^2	Undrained Strength Residual Ø deg	Drained Strength Peak C kg/cm^2	Drained Strength Peak Ø deg.	Drained Strength Residual C kg/cm^2	Drained Strength Residual Ø deg.
Penstock Slopes												
1. Clay Band (El.296-300)	CL-CI	20-30	10-23	53-81	0.5	16	1.5	15	2.7	25	1.0	18
2. Penstock seam (El.300.8m)	Cl	40	18	100	0.95	8.5	0.1	7	1.0	14	0.0	12
3. Bye-pass seam (El.305.0m)	Cl	53	18	46	0.5	17.5	0.35	14	0.5	23	0.3	19
4. Sliding Friction Test (El. 301.0 m)	CL-Cl	-	-	-	-	-	-	-	0.7	17	0.28	13
Power Channel at km.11.3												
5. Clay(centre line of power channel)	CI	11-20	13-19	100	-	-	-	-	0.45-0.65	17-19	0.35-0.50	16-18
6. Clay (El.325-330)	CL	16-19	11-53	53-90	-	-	-	-	6.8	33-34	2.0	30-3
7. Block shear test(El.335.0m)	CL	14	10	-	-	-	-	-	1.5	29	0.6	27
8. Open pit	CL	17-33	16-28	-	-	-	-	-	0.6-1.3	19.5	0.4-1.0	15. 17.
9. Seam (El.327.0m)	Cl-CH	41	28	40	-	-	-	-	1.7	14.5	-	-

where the overburden material above it was 2.5 m. or less measured from the final excavated level (iv) on the right bank all material was excavated upto 1/2 m. below the seam in slope of 1:1 filled on it by pervious material.

Measures Adopted to check Seepage Troubles

The reasons for an abnormal seepage in km.7.5 to 11.0 of the power channel were attributed to (i)presence of heavy depressions in the area away from the toe of the right bank.(ii)dislodging of the maxphalt filled in the joints of lining and (iii)malfunctioning of the pressure release valves in the form of opening of the flap by the flow of water in the channel, thereby providing recess for direct flow of water in to the bank.

The remedial measures adopted to counteract this trouble included (i)loading of the effected area in a length of about 2.5 km by filling GP-GW material giving cross slope of 15cm. towards seepage drain underlain by a layer of fine filter of 300mm. average thickness to arrest the movement of particles (ii)providing 15cm.diameter relief wells at a distance of 20.0m. centre to centre to a depth of 4.0m. approximately below the bed of the long drains (iii)refilling of joints a proper mixture of 80/25 and maxphalt and (iv) closing and repairing of the valves to proper adjustments.

PERFORMANCE OBSERVATIONS

In order to verify the design assumptions and the efficacy of the remedial measures adopted, instrumentation was carried out at the power house and at the channel (Km 7.5-10.0) site. Casagrande and USBR piezometers for obs rving pore pressures on the penstock slopes,h draulic earth pressure cells behind retaining wall to observe the earth pressures develope deformeter pegs on either side of the joint between intake and penstock slopes for findi out the movement, if any, of the penstocks w installed. Observation pipes were installed the seepage affected area along the power ch nnel to obtain the residual pressures. The l cation of instruments installed at power hou site is shown in Fig. 4.

Pore Pressure Measurements

Four embankment type USBR piezometers were i stalled at points where clay seams were dayl hting in different units of penstock slopes ring its construction in 1977. The observati carried out on these piezometers indicated t maximum residual pore pressures varied betwe 0.34 to 1.03 m.which subsequently reduced to neglegible amount thereby confirming the des assumption of zero pore pressure at the exit point of the seams. Thirteen Casagrande piez meters (Arora et al-1981) were installed in nonflow section between units 1,2 and 3,4 on penstock slopes with few tips resting in gra lly band, some in clay seam and some in clay band. Regular observations recorded are plot in Fig.5 for selected tips and the maximum p pressure values are tabulated in Table II.Th observations indicate that (i) in the initia stages, the pore pressure were slightly erra but after attaining maximum values they decr sed. After filling of the channel, pore pres re in tips resting in clayey strata remained almost unaffected, possibly due to low perme bility of the strata but pore pressure in ti resting in gravelly bands increased. (ii) th values of pore pressure coefficient 'r_u' com ted from the maximum pore pressures recorded before and after filling of channel(Table II varied from 0.25 to 0.30 for clay seam mater and from 0.14 to 0.24 for sandy gravelly mat ial which were almost the same as obtained e lier (IRI-1978). After running of the channel, the pore pressure ratio r_u did not change substantially in clayey strata but increased for sandy strata (iii) The observed pore pressures were invariably less than the design values (Table II) both during construction and after filling of the channel.

Earth Pressure Measurements

Six hydraulic earth pressure cells were installed on the back of retaining wall (Fig.4). The maximum earth pressure values as indicated by these cells before and after running of the channel are given in Table III (IRI-1982) which indicates that the earth pressure values had either remained constant or increased slightly with time. Since the retaining wall has not shown any movement so as to change the at rest

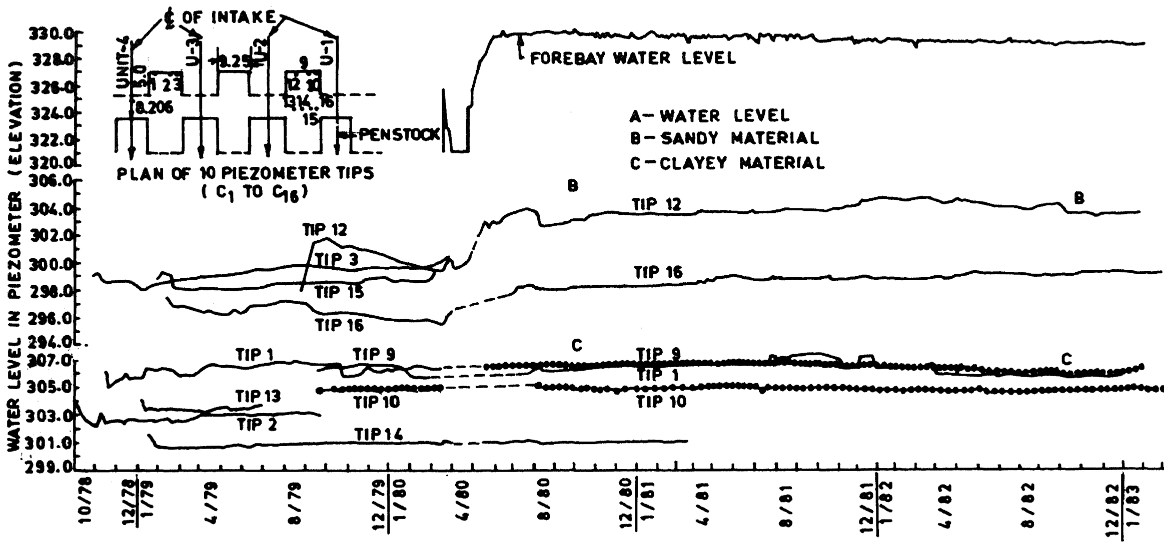

Fig.5: Pore Pressure Observations

TABLE II - Observed and Computed Pore Pressure

Tip No.	Location	Stage of filling forebay channel	Maximum pressure El.m.	Tip elevation	Pore pressure in head of water m.	Ground level m.	r_u	Pore pressure assumed in design (El.m.)
	Between Unit 1 & 2							
9	Seam I	B	306.20	304.28	1.92	307.00	0.28	309.00
		A	306.40		2.12		0.30	
10	Seam II	B	304.60	301.46	3.14	307.70	0.25	308.40
		A	304.60		3.14		0.25	
12	Sandy Layer I	B	300.50	293.68	6.82	307.80	0.24	
		A	304.60		10.92		0.38	
14	Clay seam II	B	300.50	299.127	0.873	307.00	0.05	301.20
		A	300.50					
16	Sandy Layer II	B	298.00	290.97	7.0	307.00	0.22	
		A	299.10		8.13		0.25	
	Between Unit 3 & 4							
1	Clay seam I	B	305.4	304.32	1.08	307.00	0.20	309.00
		A	305.9		1.68		0.28	
3	Sandy Layer I	B	300.4	297.42	2.98	307.10	0.15	
		A	-		-		-	

Note :- B-Before filling of forebay channel & A - After filling of forebay channel

TABLE III - Observed & Computed Earth Pressures

Cell No.	Cell elevation m.	Pressure before running of forebay channel: Computed	Observed in kg/cm²	Observed % of computed value	Pressure after running of forebay channel: Computed	Observed in kg/cm²	Observed % of computed value
2	306.205	2.125	0.0	0.0	3.244	0.40	12.3
3	303.695	2.593	0.0	0.0	3.580	0.575	16.0
4	307.480	1.494	0.66	33.0	3.039	2.00	65.8
6.	307.445	1.900	0.84	44.0	3.044	1.950	64.6
7	303.725	2.588	0.0	0.0	3.573	0.0	0.00
8	303.640	2.602	0.20	8.0	3.583	0.65	18.1

condition to active value, the observed values have been compared with the values computed un -der K_o condition by considering $K_o = 1 - \text{Sin } \emptyset$. It is evident from Table III that cell no.2,3, 7&8 which have reflected only zero to 18 percent of the computed pressures have malfunctioned which might be attributed to local arching of soil around the cell, leakage from the cell or its leads and rusting of cell diaphragm of mild steel. The lead of cell no.2 was ac -tually seen leaking at site. The other cells 4 and 6 however yielded pressures upto 65 percent of the computed values during running of the channel.

Deformation Measurements

Four pairs of deformeter pins, one in each unit were installed across the joint between intake block and the penstock (Fig.4). The distance between the pins were measured regularly by a mechanical deformeter and are plotted in Fig.6

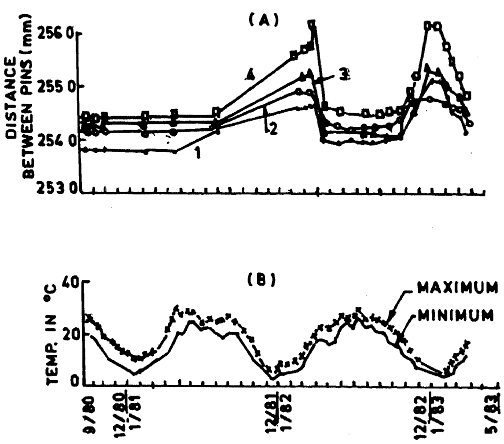

Fig.6 : Deformeter Observations

A perusal of Fig.6 indicates that after July, 1981 the distance between the pins in all units has increased regularly upto Feb. 1982 after which it decreased regularly upto June '82 to March 1983. Possible reasons for such abnormal variation were looked into and it was found that this variation is due to variation in tem -perature at site. From July to Feb., due to decrease in temperature, the contraction of concrete of intake and penstock blocks have taken place thereby widening the joint whereas in summers, the expansion of concrete has reulted into narrowing the gap and hence decresing the distance between the pegs. The maxium increase in distance between the pegs was ound to be 1.752 mm in unit 4 in Nov.1982 here-as in 1983,the maximum increase was .7636 in the same unit and occurred in the onth of November. No conclusions regarding ovement of intake or penstock blocks could, ıerefore, be drawn from these observations.

ːRFORMANCE OF SEEPAGE CONTROL MEASURES

ıe performance of seepage control measures ːre observed by regular inspection at site ıd observations made in the observation pipes ıstalled at site. It was indicated that the measures adopted were giving satisfactory p formance in the reach as the boiling was cc rolled in the affected area. The zone of bo has, however, shifted itself further downwa of the loaded area near the River Ganges. T discharge in the seepage drain has been red ced to about 0.36 cumec in March 1981 and (in March 1982.

CONCLUSIONS

The plastic clay seams present at Chilla Po House scheme caused a number of hazards dur construction stage. Since they are zones o weakness in the foundations, there is alwa danger of movement along them. A number of remedial measures have been adopted to cont act them, but these seams are still risky l ely to endanger the safety of the structure any time. Despite the instrumentation data served so far having indicated the correctn of the design assumptions for the safety o the structure. The only way out seems to c out the observations on the installed instr ments regularly and to keep watch over the -formance of the structure so that distres any, may be known at the proper time.

REFERENCES

Arora P.K. et al (1981) "Instrumentation o Chilla Power House" Proc. 49th Annual R arch and Development Session of Central Board of Irrigation and Power, Ooty (Ta Nadu) Sept. 1-5, 1981

CCC Hardwar (1977) "Landslide at Intake St ures of Power House" pp.4&6 (unpublishe

CCC Hardwar (1976) "Note on Slip zone km.1 to 11.400", pp.2 (unpublished)

CCC Hardwar (1978) "Slips in Slip Zone nea 11.3 of power channel and its remedial sures" (unpublished).

CDD IDUP(1977) "Left Bank Slide at km. 11 of Power channel, GRC Hydel Scheme" pp. 17 (unpublished)

Henkel,David,J (1967) "Local Geology and t Stability of natural Slopes" Journal S ASCE, SM4 July 1967.

IRI,UP (1976-78) "Test Report T.M.No.47(S- T.M.No.48(S-39),T.M.No.48(S-58), T.M.No (S-15), T.M.No.49 RR(S-7) (unpublished)

IRI,UP (1982) "Analysis of Instrumentation data of Chilla Power House site Hardwar T.M.No. 53 RR(S-2).

TDDC Roorkee (1977) "Stability of Intake B and Downstream Slope-Chilla Power House May 1977 pp. 5 (unpublished).