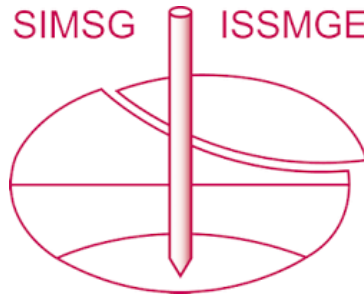


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Predicted and monitored settlements of the power house of a hydroelectric project in Pakistan

Prédictibilité et surveillance contrôlée du tassement de terrain d'une centrale hydroélectrique en projet au Pakistan

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ABSTRACT: Monitoring of foundation movements through instrumentation is recognized as a very useful tool for controlling the behaviour of structures. Besides, the difference between the predicted settlement at the design stage and the actual settlement measured during and on completion of loading, provides a measure to analyze the gaps that exist between theory and reality, so that better settlement prediction methods may be identified/evolved.

The powerhouse of 184 MW generation capacity Chashuna Hydropower Project in Pakistan, is sited 24 meters below the Indus river bed on its right bank, over a thick cushion of generally dense fine sand. During the design phase in 1994-95, the settlement of 61m x 202m powerhouse was predicted to range between 95 mm and 298 mm, by various approaches including NGI methods for rigid/flexible base, analysis of Menard pressuremeter data and elastic methods for flexible base condition.

Ten magnetic probe extensometers were installed during the excavation and construction in 1995-96, at various points at powerhouse location. Each extensometer has ten embedded spider magnets along its depth. By February 2000, the extensometers have shown settlements ranging from 63 to 142 mm, under about 99% of the average static design load. A number of surface settlement markers show the settlement of powerhouse to range from 61 to 79mm. These evaluations suggest that the settlements predicted with pressuremeter data as well as with the NGI (Flexible Base) approach fall close to the measured settlements.

This paper provides an interesting comparison between the predicted and the monitored settlements of the powerhouse of Chashuna Hydropower Project.

1 INTRODUCTION

A variety of instrumentation of a structure is possible to monitor its movement during construction and post construction. The commonly used instruments for this purpose include surface settlement markers, magnetic probe extensometers and triaxial joint meters.

The powerhouse of the Chashuna Hydropower Project is located on thick deposits of sand, on the right bank of river Indus (near Chashuna Barrage) in Pakistan. It mainly consists of six thick R.C. blocks, placed at 24 meters depth below the river bed level. The four central blocks (called unit blocks) measure 34 m x 61 m, each and support two turbo generators of 23 MW generation capacity, each. The edge block on each side (called a service block) measures 33 m x 61 m and is meant to provide the service requirements to the turbo generators, during maintenance/fault etc. The powerhouse also supports the Chashuna Right Bank Canal, which is ducted over the downstream face of the powerhouse (WAPDA/CGC 1994). This paper primarily compares the settlements monitored using magnetic probe extensometers with those predicted during the design of the project, employing a number of procedures.

A total number of ten magnetic probe extensometers were installed in the ground at the location of the powerhouse (Fig. 1), in two stages. Extensometers 1 to 6 were installed during excavation of the powerhouse in September 1995. Extensometers 7 to 10 were installed during construction of the powerhouse in September

1996. Each extensometer has ten embedded spider magnets, numbered as 1 to 10, installed at various elevations below the powerhouse blocks (base elevation 168 m), ranging from El. 177 m down to El. 83 m.

The first stage excavations (elevation 192 m to 181 m) preceded the installation of extensometers 1 to 6. The second stage excavations (elevation 181 to 168 m) continued from September 1995 to December 1995. The concreting of powerhouse floor was started in right service block (RSB) and unit block No. 4 (UB # 4) in January 1996. The concreting of the remaining blocks was started progressively between April 1996 and October 1996. The elevations of all the spider magnets in the extensometers are being recorded regularly on a weekly basis. Upto the end of February 2000, about 99% of the average static design load of 360 kPa had been applied on the foundation soil.

2 PREDICTED SETTLEMENTS

The expected settlements under powerhouse blocks were estimated at the design stage by various methods. For this purpose, the SPT, CPT and pressuremeter test results were employed. The total static settlements were predicted for individual blocks in isolation as well as considering the effect of loading on the adjacent blocks under net applied pressures. These settlement predictions are for long-term condition; under the cumulative effects of design loads (WAPDA/CGC 1996). The settlements predicted during the design stage are given below in Table 1.

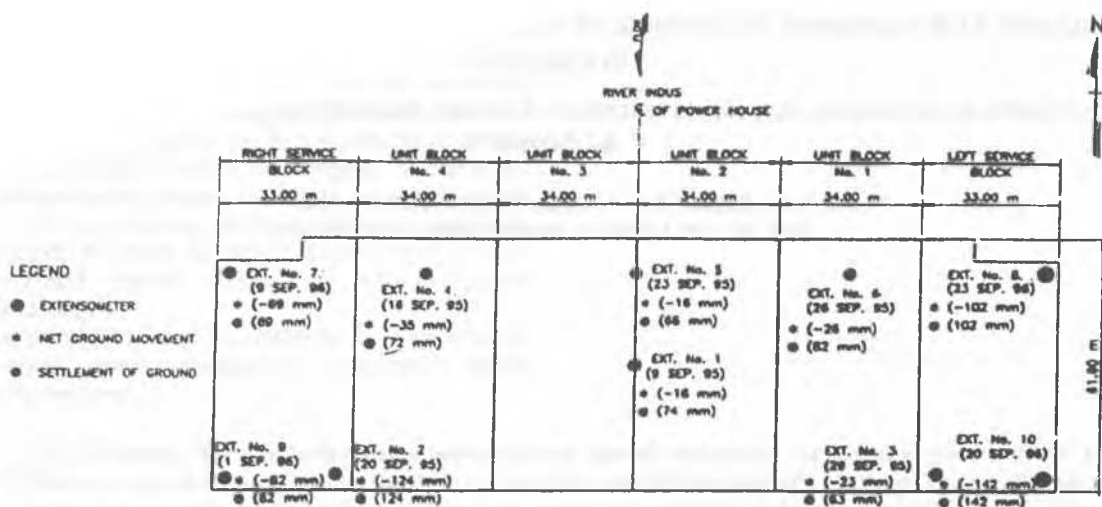


Figure 1. Location of magnetic probe extensometers and ground movements

Table 1. Settlements predicted during the design stage

Basis of Analysis Procedure	Total Static Settlement of Individual Block (mm)	
	In Isolation	Considering Loading Effect of Adjacent Blocks
(a) NGL assuming Rigid Base	139	298
(b) NGL assuming Flexible Base	60	123
(c) Pressuremeter Data	65	95
(d) Elastic solution for Flexible Base	68	222

The procedure (a) is based upon Whitman & Richart and Lee relationships for a rigid rectangular base. The required elastic moduli were estimated by NGL's Lunne and Christoffersen (1983) method (Skempton 1986), (Lunne & Christoffersen 1983) on the basis of CPT data and SPT data converted to CPT cone tip resistance.

The settlement (ρ_z) of a rigid rectangular base at any depth is given by:

$$\rho_z = (P(1 - \nu^2)) / (\beta_z \sqrt{BL} E_c) \quad (1)$$

Where: P= Total Load = $p_{ave} \times B \times L$
 B,L= Foundation breadth and length
 ν = Poisson's ratio of sand
 β_z = A factor depending on L/B
 E_c = Equivalent elastic modulus

The sand deposit between elevation El. 168 and El. 80 was divided into five layers of 8 m, 10 m, 20 m, 20 m, and 30 m thickness, respectively.

The equivalent elastic modulus of two consecutive layers was evaluated by the procedure given by Thenn de Barros (Poulos & Davis 1973) as below:

$$E_e = (((h_1 \times 3 \sqrt{E_1}) + (h_2 \times 3 \sqrt{E_2})) / (h_1 + h_2))^3 \quad (2)$$

Where: h_1, h_2 =Thicknesses of consecutive layers
 E_1, E_2 = Elastic moduli of consecutive layers

The elastic modulus for each layer in the calculation model was derived as;

$$E = M(1 - (2\nu^2 / (1 - \nu))) \quad (3)$$

$$\text{and } M = M_0 \sqrt{\sigma' v'_0 + (\Delta \sigma' v' / 2) / \sigma' v'_0} \quad (4)$$

Where, M = Constrained modulus
 M_0 = Initial constrained modulus
 $\sigma' v'_0$ = Initial effective vertical overburden stress
 $\Delta \sigma' v$ = Increase in stress due to construction

The procedure (b) is based upon evaluation of elastic moduli as for procedure (a) and subsequently evaluation of settlement under flexible area using elastic theory, ultimately adjusted by the least square method.

The procedure (c) is based upon the evaluation of Menard pressuremeter data (Anar 1991) as follows:

$$w = (2q B_0 / 9 E_d) (\lambda_d B / B_0)^\alpha + (\lambda_c q B / 9 E_c) \alpha \quad (5)$$

Where:

w = Settlement of a rigid shallow foundation

q = Average contact pressure

B_0 = Reference width (60 cm)

B = Foundation width

λ_c, λ_d = Shape factors depending on L/B

E_c, E_d = Moduli measured in the zones dominated by isotropic and deviatoric stress components, respectively.

α = Rheological factor depending upon soil type and pressuremeter modulus to limit pressure ratio.

The procedure (d) is based upon evaluation of elastic settlements ($w = (\sigma/E) h$) by adopting an 84 rectangular elements based, 8 - layer model. The stress in a layer (of thickness h) has been evaluated as under, as per Holl (Poulos & Davis 1973):

$$\sigma = p/2\pi (ATN((lb/zR_3) + lbz/R_3(1/R_1^2 + 1/R_2^2))) \quad (6)$$

Where:

p = Contact pressure

l,b = Foundation length and breadth

z = Depth between application of pressure and measurement of stress

$R_1 = \sqrt{l^2 + z^2}$

$R_2 = \sqrt{b^2 + z^2}$

$R_3 = \sqrt{l^2 + b^2 + z^2}$

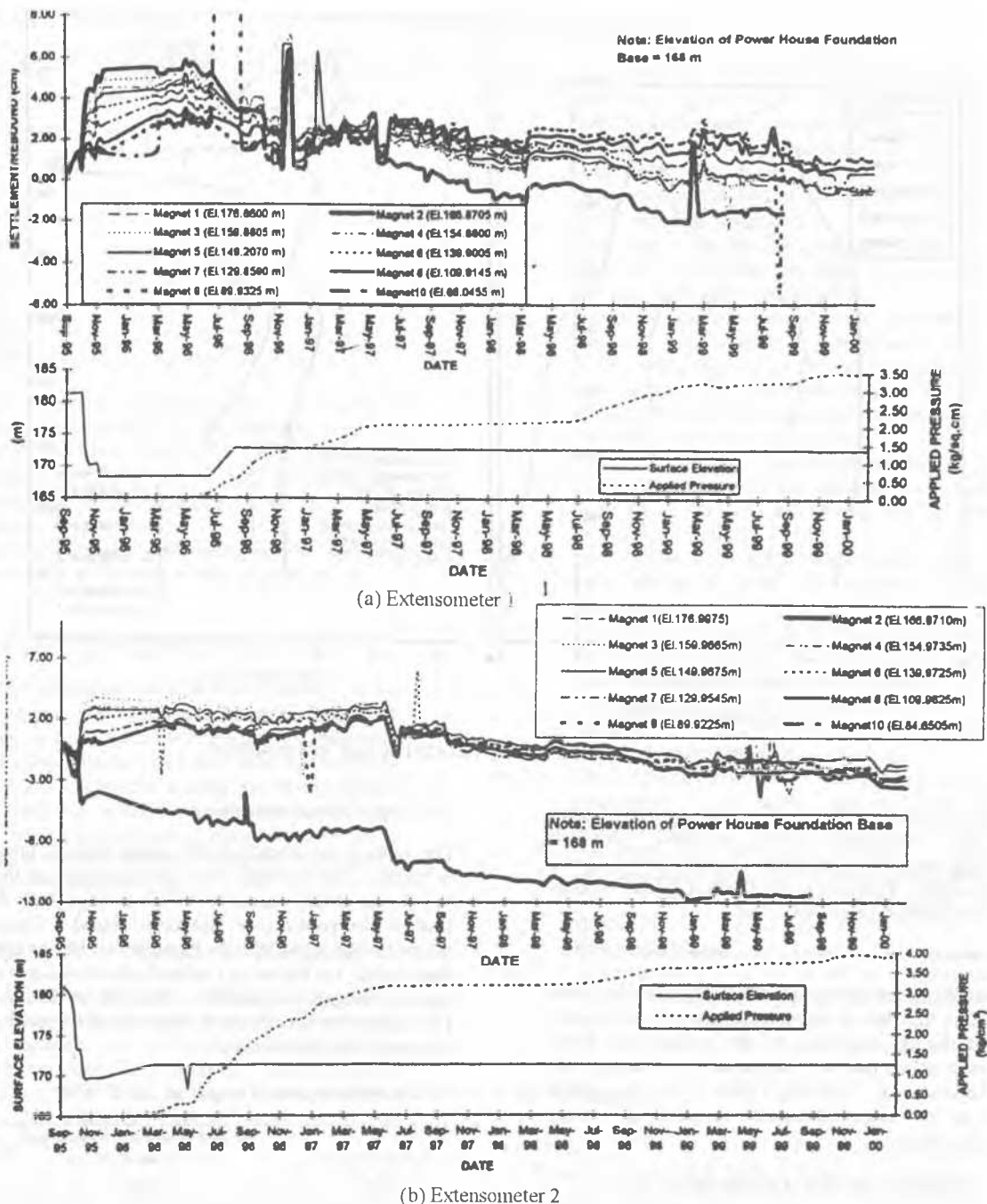


Figure 2. Settlement-time trend of extensometers 1 and 2

3 VERTICAL MOVEMENTS OF SPIDER MAGNETS

3.1 Settlement-Time Trend of Extensometers 1 to 6

Vertical movements (rebound or settlement) of the foundation soils at different elevations (magnet locations) have been measured. The surface elevation due to excavation and the applied pressure due to concreting of the powerhouse blocks have also been included. The applied pressures have been taken as the average pressures over the entire powerhouse block containing the extensometer. Where an extensometer lies between two blocks, the applied pressure has been taken as the average of the average pressures of the two blocks. The following information (Table 2) has been inferred from the vertical movement data (WAPDA/CGC):

Table 2. Settlement-time trend of extensometers 1 to 6

Extensometer No.	General Range of	
	Rebound (mm)	Max. Settlement Following Rebound (mm)
1	25 – 55	0 – 16
2	0 – 30	10 – 40
3	0 – 36	5 – 20
4	6 – 13	25 – 35
5	10 – 32	5 – 15
6	18 – 42	10 – 30

It is to be noted that the higher limit of rebound or settlement generally corresponds to the upper most magnet of an extensometer. Some unusual peaks and depressions have also been noted, which may be attributed to some human error. Besides, an unusual behaviour of experiencing another rebound (starting

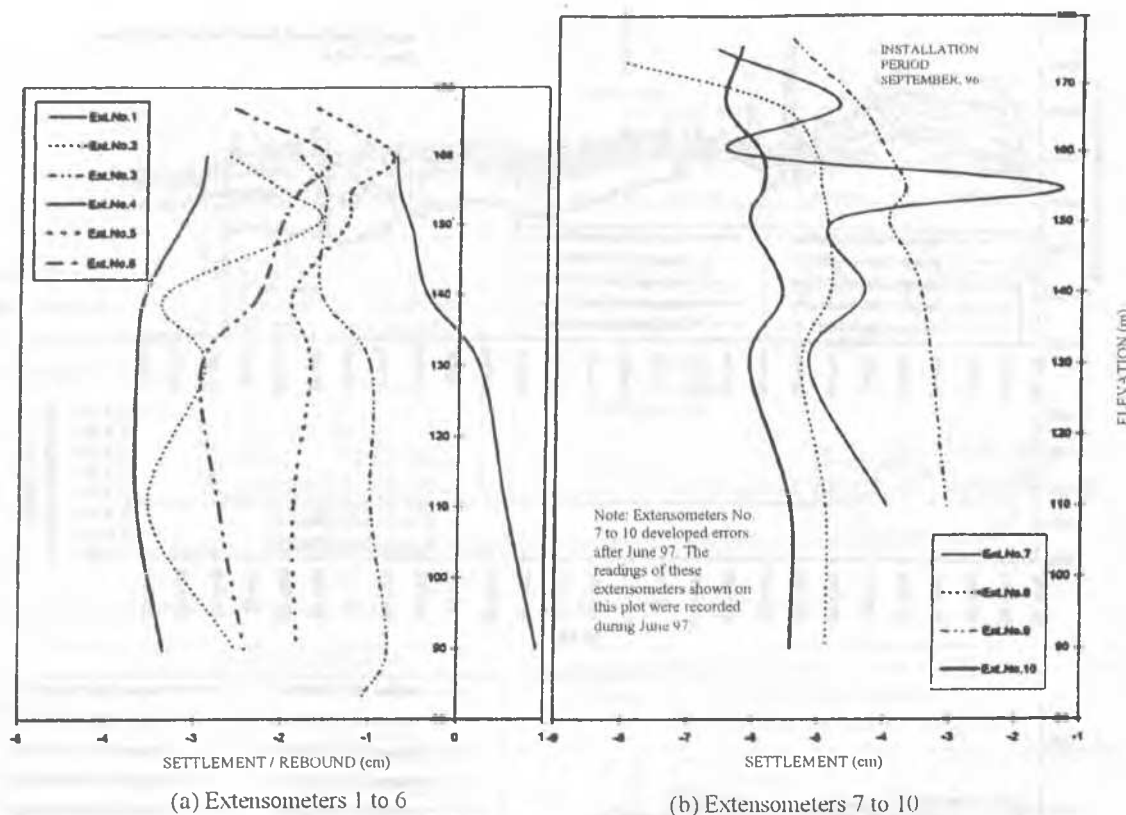


Figure 3. Vertical movement of soil recorded through extensometers

between October 1996 to December 1996) after maximum settlements were recorded, although concreting was still in progress, has also been observed in all the plots. Figures 2a and 2b show typical records of data for extensometers No. 1 and 2.

3.2 Settlement – Time Trend of Extensometers 7 to 10

The surface elevations due to excavation have not been included in this data as the extensometers 7 to 10 were installed during concreting of the powerhouse floor, when excavations had been completed and heaving had been partly closed. Following (Table 3) are the general ranges of maximum settlements recorded at various magnet locations:

Table 3. Settlement-time trend of extensometers 7-10

Extensometer No.	General Range of Maximum Settlements (mm)
7	69 – 94
8	71 – 105
9	70 – 90
10	82 – 146

Some unusual trends have also been observed in the data, which include excessive settlement along very steep slopes in the initial readings, unusual depressions/peaks and rebounds during the concrete loading.

The settlements recorded by extensometers 7 to 10 are somewhat higher compared to those recorded by extensometers 1 to 6, although the pressures have not been excessively different. This is logical as extensometers 7 to 10 started operating only when concreting was in progress and a portion of rebound had been recovered i.e. settlements followed the virgin behaviour of soil.

3.3 Final Vertical Movements

The vertical movements of the spider magnets measured in various extensometers have been evaluated at the end of February 2000, when 99% of the average static design load of the powerhouse had been placed. These are shown in Figures 3a and 3b respectively. On the basis of these trends, the following (Table 4) net movements of the top-most magnet (magnet No. 2 installed at elevation El. 167 m) have been considered as the net movements at the base of powerhouse blocks:

Table 4. Net movement of magnet no. 2 at El 167 m

Extensometers No.	Net Movement of Magnet No.2 At Elevation 167 m (mm)
1	- 16
2	- 124
3	- 23
4	- 35
5	- 16
6	- 26
7	- 69
8	- 102
9	- 82
10	- 142

The net ground movements are also shown on Figure-1. Negative sign indicates compression.

4 MEASURED GROUND SETTLEMENT

The total settlements of spider magnet No. 2 installed at elevation El. 167 m have been measured following elastic rebounds, introduced by excavation. These settlements are tabulated below (Table-5) and also shown on Figure – 1.

Table 5. Measured ground settlements

Extensometer No.	Measured Ground Settlement (mm)
1	74
2	124
3	63
4	72
5	66
6	82
7	69
8	102
9	82
10	142
Average settlement	88

The average settlement of extensometers 1 to 6 is 80 mm upto February, 2000, while that for extensometers 7 to 10 is about 99 mm upto June, 1997. The Extensometers 7 to 10 stopped working after June, 1997. Therefore, the actual settlements of Extensometers 7 to 10 to date, could be higher.

The block-wise average settlement, on the basis of extensometer data varies from 70 mm to 122 mm.

5 SETTLEMENT MARKER DATA

Eight surface settlement markers were installed in the access gallery and on the upstream face wall of each unit block during April to December, 1996. The settlement readings of these markers have also been analysed upto February, 2000. The results indicate that the settlement of various markers varies from 61 mm to 79 mm. Blockwise, the average settlement ranges from 54 mm to 62 mm, which is lower than the settlement indicated by the magnetic probe extensometers.

6 TRIAXIAL JOINT METER MOVEMENTS

Ten triaxial joint meters were installed to record three-dimensional movements between the blocks. These were installed in upstream and downstream drainage galleries at the expansion joints. The relative movements recorded upto February, 2000, in all the three directions appear quite low. The maximum recorded movement in any direction is 6.97 mm.

7 COMPARISON OF PREDICTED AND MEASURED SETTLEMENTS

The analysis of extensometer data shows minor anomalies, as explained in the above sections. However, the average recorded settlement is evaluated as 88 mm at magnet No. 2 (approx. El. 167 m) under an average concrete pressure of 358 kPa (99% of the average static design load).

The settlement predictions during the design stage have yielded settlements ranging from 95 to 298 mm (using four different prediction methods) under maximum average pressure of 360 kPa. The average settlement of 88 mm recorded by magnetic probe Extensometers is very close to the lower bound of the predicted range of settlements. The recorded settlements are likely to increase slightly due to the creep effects, as indicated by the trend of time-settlement plots in Figure – 2(a) and 2(b).

8 CONCLUSIONS

- (i) The measured settlements in the foundation soils are close to the lower limit of the predicted ones under concrete loading upto end of February, 2000. An average of 88 mm settlement has been indicated by the extensometers (at 99% of average static design load) versus a prediction of 95 to 298 mm. The creep of settlement continues at a slow rate and would have exceeded the recorded average value of 88 mm, by now.
- (ii) Structural analysis of the powerhouse model is based on a static soil spring stiffness of 3000 kN/m³ which is based on 120 mm settlement of unit blocks. This settlement is more than the average recorded and expected settlement and therefore, indicates a safe design.
- (iii) The settlements and the relative movements of the unit blocks under the imposed loads, upto February 2000, are satisfactory and do not indicate any distress.
- (iv) The surface settlement markers show slightly less settlement than the magnetic probe extensometers.
- (v) The settlement measured by the extensometers fits very closely to the predictions made on the basis of the pressuremeter data (procedure (c)). The measured settlement is also fairly close to the prediction made by NCI method assuming flexible base condition (procedure (b)).
- (vi) The extensometers 7 to 10 were installed during concreting. The elastic rebounds due to excavations had been partly closed up by then. As such, the settlement records of these extensometers show higher order settlements (due to virgin compression) as compared to those shown by extensometers 1 to 6 (recompression).
- (vii) Extensometers 7 to 10 would have indicated somewhat higher settlements, had they not gone out of order after June, 1997.

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