Measurement of Displacements in Beas Dam Embankment

by
N. K. Berry*

Introduction

The Beas Dam is located in Kangra District of the Himachal Pradesh at a distance of about 38 kms by road from Mukerian Railway Station on Jullundur-Pathankot section of the Northern Railway. The dam is an earth-fill structure on the Beas river and is 132.6 m high from the deepest foundation level. The width at the base is 609.6 m and that at the top 13.72 m. The Beas Dam involves 36 million cubic metres of fill placement inclusive of 11.5 million cubic metres of impervious fill placement in the core section. It was necessary to provide for a sufficient width of the impervious zone (182.88 m at the base), as the Beas Dam location falls in a semi-arid region at the Punjab border in an area which can expect the highest ground vibration intensity in India due to earthquakes. The highest horizontal seismic coefficient required for designs in this area as recommended in the National Building Code of India 1970 published by the Indian Standards Institute is 0.08.

Figure. 1 shows the plan and cross-sections of the dam alongwith the L-section through the centre line of the key trench. The geology of the dam foundations and the soils instrumentation to monitor the Beas Dam embankment movements as provided, have been indicated in the plan and cross-sections. The L-section through the centre line of the key trench further shows the construction sequence adopted on the dam.

A considerable area in the vicinity of the dam site was explored for assessing the availability of suitable construction materials for the dam embankment, and laboratory tests performed on representative samples to determine the index and engineering properties. After a thorough and detailed exploration and investigation of impervious materials from various prospective borrow areas in the vicinity of the dam, it was finally decided to use mixed sandrock and clayshale material available from structural excavations at the Project in the core section of the dam. A few samples mixed in different proportions of claystone and sandrock were tested and the results indicated the following average values of soil properties :

Max. Dry Density	1.89 gms/cc
Optimum Moisture Content	12.5%
Permeability Range	1.2×10^{-8} to 5.9×10^{-6} cm/sec
Angle of Internal Friction	30°
Cohesion	0.32 kg/cm^2

*Director, Beas Dam Project, 732, Sector XI B, Chandigarh, India.
The paper is open for discussion till the end of January 1976.

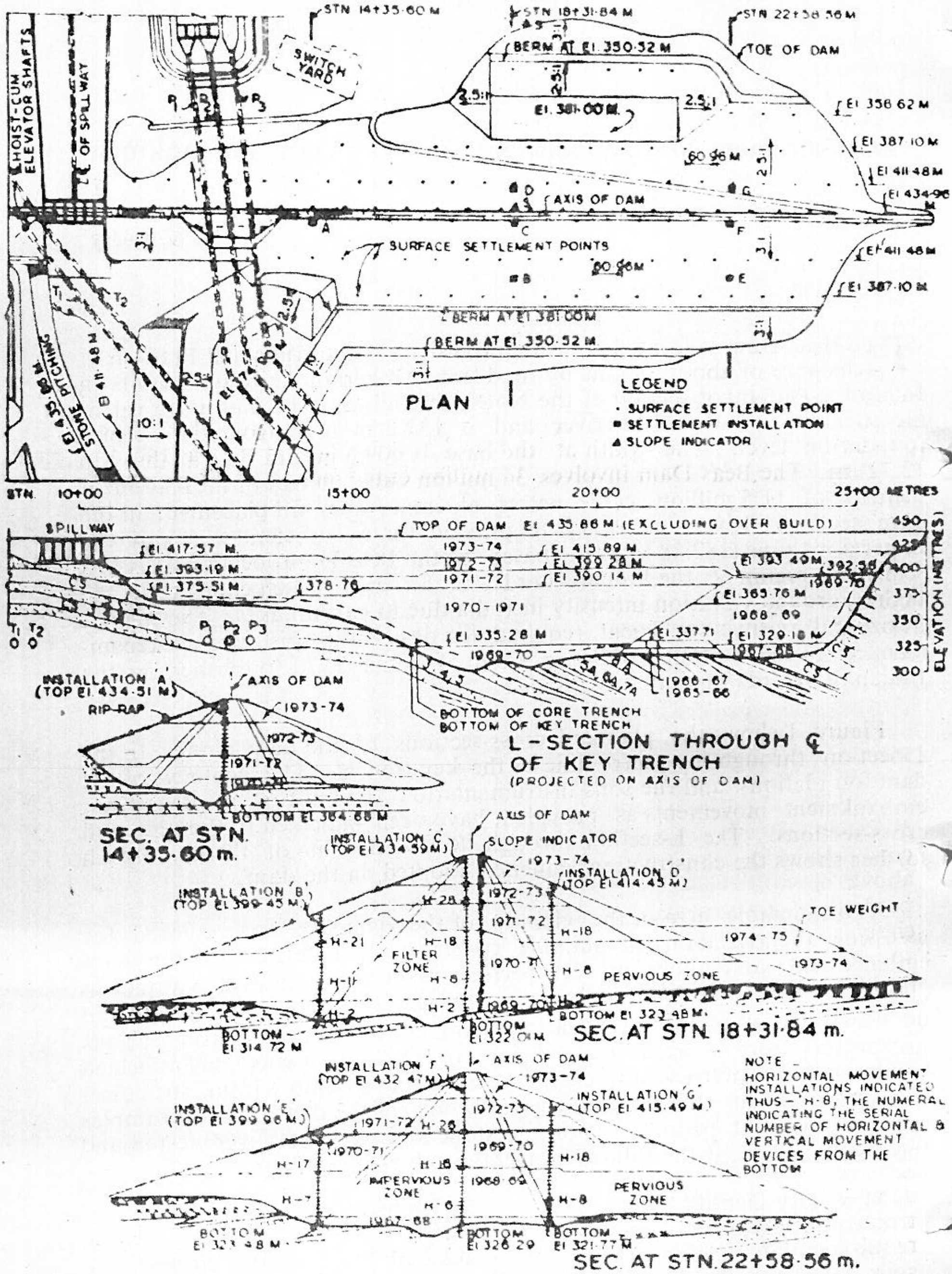


FIGURE. 1: Soil instrumentation for the Beas Dam embankment movements.

As a result of the investigations after detailed explorations and testing of samples from various prospective borrow areas for the filter zone and pervious zone of the Beas Dam located within a lead of 3.2 to 5.6 Kms suitable borrow areas commensurate with total material requirements were selected. The materials selected were predominently hard rounded quartzite free from organic impurities. The results of grading analysis indicated percentage of sand varying from 22 to 31 and that of gravel from 40 to 77. In-situ density tests during quality control gave an average range of 1.97 gms/cc to 2.00 gms/cc. One large scale direct shear test was performed on a sample from one of the borrow areas. The sample had fractions below 0.475 cm as 19% and unit weight of all-in aggregate as 2.16 gms/cc, with the angle of repose for this material varying between 37° to 38.5°. An angle of internal friction of 41° was obtained from the test. This test was conducted on the same lines as those for south Holston Dam (USA). Direct and Triaxial shear tests were also performed on these materials which indicated that an angle of internal friction of 38° would be reasonable.

The paper describes the 'Soils Instrumentation' provided for measurements of settlement of the dam foundations, settlements within the dam embankment, upstream-downstream horizontal strain within the dam, horizontal displacements and surface movements from surface reference points. The measurements of foundation settlement, embankment compression or consolidation, the change in elevations with time of the fill at various elevations, upstream-downstream horizontal strain in the embankment, horizontal displacements of Beas Dam embankment, and surface movements of the dam during the construction and post-construction period, have been plotted and critically analyzed.

Provision for Post-Construction Consolidation

Designs provided for an over-build along the length of the dam from 30 cm at abutments to 213 mm at the deepest section viz. El. 1437 against crest elevation of 1430 by steepening upstream and downstream slopes above elevation 1400. The overbuild comprised 91 cm for maximum predicted settlement of dam and foundations after completion and a circular arc camber of 122 cms along the length of dam so that it does not appear to sag. It was kept as 30 cm minimum at the abutments so that the crest level is not encroached with possible erosion or embankment and foundation settlement.

Soils Instrumentation

The Beas Dam has been divided into three parts for construction phasing, the right abutment dam section, the central dam section and the left abutment dam section. The L-section along the dam axis and the other dam sections in Figure-1 show the year-wise detailed sequence of construction operations that was enforced for adequate and safe diversion requirements. It is in view of this construction phasing that the main soils instrumentation in the Beas Dam was located in 3 sections as indicated.

Settlement of the dam foundations and consolidation or compression within the dam embankment is measured by seven USBR type telescoping cross-arm installations installed at the three sections of the dam.

The horizontal movement or strain within the earth-dam in a direction at right angles to the axis of dam is measured by USBR type internal horizontal movement devices provided at 30.5 m vertical intervals in six out of the seven number USBR type telescopic cross-arm installations. The horizontal movement is transformed into vertical movement by a linkage system and is obtained by using the measuring torpedo and reading scale.

The inclinometer installation has been made in the deepest section of the dam through the impervious core and consists of 69 casings of 1.5 m length each. The 75 mm inside diameter aluminium casings have four continuous longitudinal grooves. The 1.5 m casing sections are connected by telescoping couplings which allow 150 mm of vertical movement at each coupling. The longitudinal grooves are aligned parallel and perpendicular to the axis of the dam. The torpedo which runs on guide wheels inside the grooved thin aluminium casing 75 mm in diameter contains heavy pendulum which works over a wire-wound resistance so that its inclination to the torpedo axis may be detected with an electrical circuit. The torpedo is used to determine inclinations in two vertical planes at right angles for each 1.5 m length of the aluminium casing along the two pairs of grooves in the casing. The horizontal movement or displacement is measured by plotting the deflections and depths. Settlement or vertical movement is also measured by this instrument with a special settlement torpedo having collapsible fins which is lowered through the telescoping sections of the casing on a steel surveyor's tape. Spring loaded arms catch the bottom of each 1.5 m section and the depth to the bottom of each section is recorded. The elevation of the top of the slope indicator casing is then used to convert the depth readings to elevation.

Surface monuments to measure both settlement and horizontal alignment involving the installation of 1.5 m long steel rods of 25 mm dia. embedded in concrete have been provided on the upstream and downstream faces of the dam as also on the upstream parapet wall at the top of the dam as indicated.

Tests of Embankment Materials at Settlement Instrument Installations

Samples of the soil at the location of each cross-arm were taken for record field and laboratory tests and were tested for Mechanical Analysis, Specific Gravity, Atterberg's limits, Field Moisture Content, Optimum Moisture Content, Field Density and Maximum Laboratory Dry Density.

Embankment Movements

Embankment Compression or Consolidation

(*a*) The vertical consolidation of the individual 305 cms deep layers between the adjacent cross-arms in the telescoping USBR type cross-arm installations was observed and plotted against time for each of the seven such installations and also for the 335 cm layers around two casing section numbers each of 152 cm for the Slope Indicator installation. The vertical consolidation was generally found to vary considerably. It is believed to be due to normal variation in moisture content, degree of compaction and the character & hetrogeneity of the soil, rather than to any fault in the measurements or apparatus. No significant relationship between the consolidation of the 305 cm or 335 cm layers with the loading conditions could thus be found. It was however seen that in general the lower layers with greater fill loads had shown more consolidation than the top layers with lesser loads. All the 305 cm or 335 cm layers of impervious material followed more or less the same pattern of consolidation *i.e.* consolidating at a fast rate immediately after placement during the construction period, than slowing down during the shut-down period and again consolidating at a fast rate during the following construction period. Consolidation was faster during the early construction period and slowed down with the passage of time. It was seen that 15 to 30 percent (with an average of 22.5 percent) of the total consolidation of any 305 Cm or 335 Cm layer took place during the first one month after placement. During the large shut-down of practically around 3 years on the right abutment dam section, consolidation of lower elevation layers continued for about 5-6 months at the reduced rate whereafter there was nominal consolidation only till the end of shutdown period. Rate of consolidation again increased suddenly on the start of construction operations.

Consolidation of individual layers of the impervious core continued uninterrupted at a rate varying from 0.22 mm/day to 0.00 mm/day as time passed after completion of the dam structure in June 1974 despite filling of the reservoir from July 1974 onwards. Almost the same rates of consolid-ation were observed during previous shutdown periods thus indicating that the reservoir storage from June 1974 onwards did not have any significant effect on consolidation of the impervious core. Over-all consolidation of the embankment after completion of the dam has however varied from 0.22 mm/day to 0.03 mm/day.

Negative consolidation was observed in a few layers in the pervious fill zone due to increase in the distance between consecutive cross-arms as a result of greater settlement of lower cross-arms as compared to the upper cross-arms. This unequal settlement of the cross-arms could obviously be due to unequal degree of compactness of the different fill placement layers. Most of the pervious layers (67% to 75% of the total number of layers) consolidated upto 1% of the original distance. Almost all the layers showed nominal consolidation during shut-down periods. Some consolida-tion was noticed in the pervious fill due to submergence under water with 49 mm and 76 mm consolidation in a total height of pervious fill of 77 m and 86.5 m at installation B and F.

(*b*) The embankment compression during and after construction for 15.25 m layers at the location of USBR type cross-arm installation and for 16.77 m layers at the location of Slope Indicator installation was also plotted by adding the observed compression between five consecutive USBR cross-arm/10 consecutive Slope Indicator pipe sections, alongwith the fill placement elevations and reservoir elevations against time (Figure 2). Notice the very little embankment compression that took place during the 3 years shut-down period for installation 'F'. The compression in the dam embankment can also be seen to have increased only slightly in the first year after construction.

(*c*) The change in elevations with time of the various cross-arms in the USBR type vertical and horizontal movement installations from the install-ed elevations depends apart from the placement moisture content and soil characteristics, on the speed of construction, relative durations of the shut-down periods in the past, the amount of progressive foundation settlement,

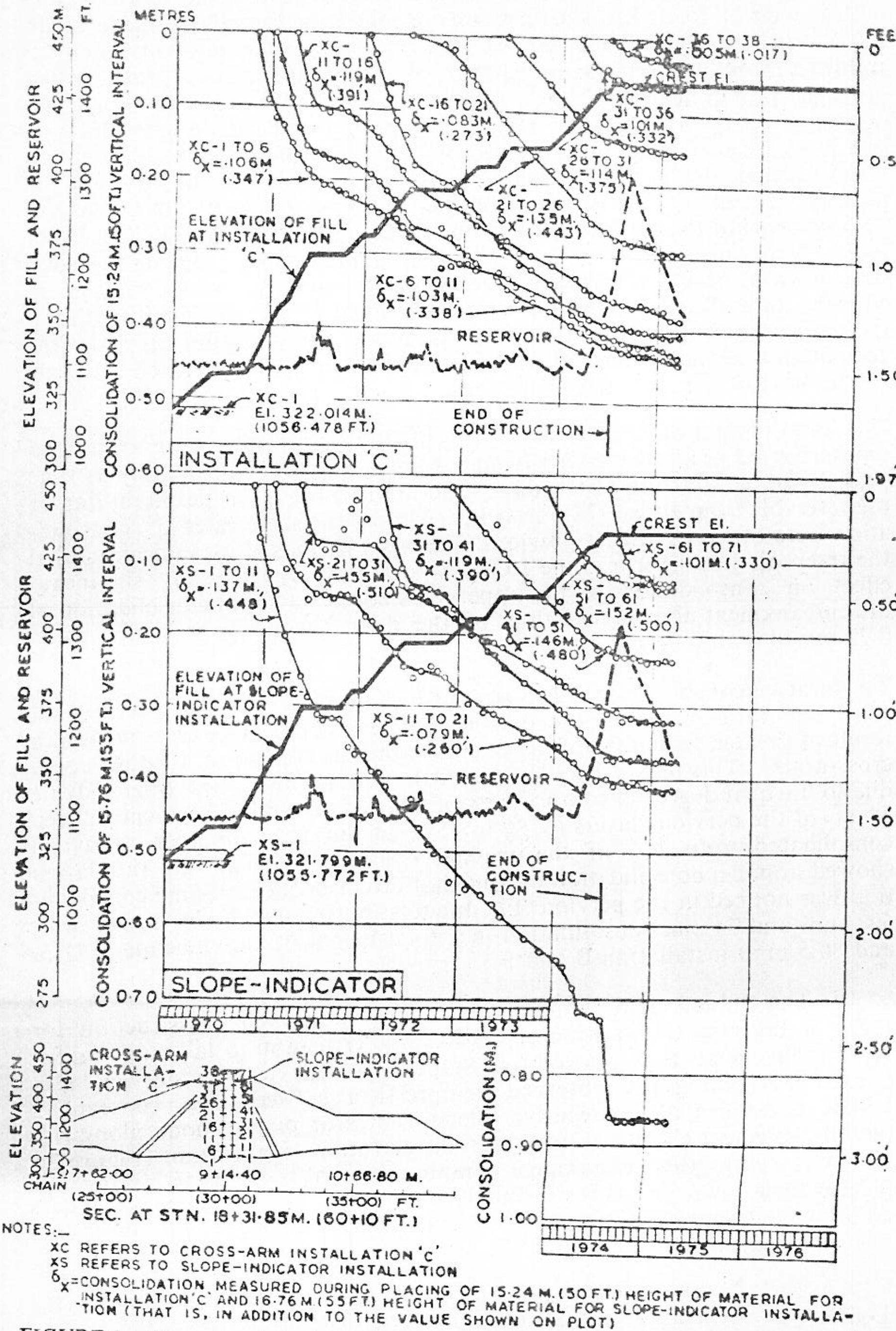


FIGURE 2. Summary plot of measurements of embankment compression during and after construction

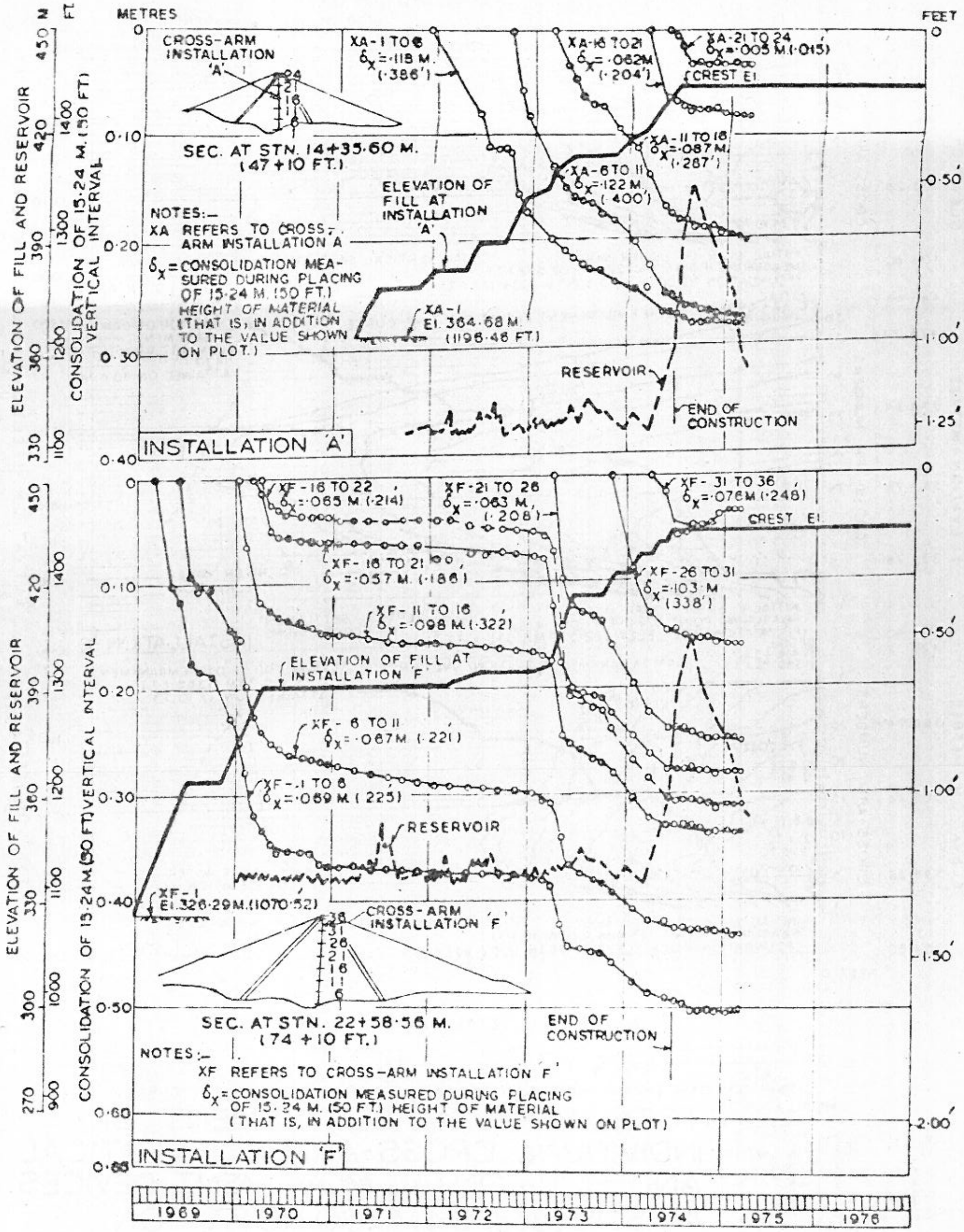
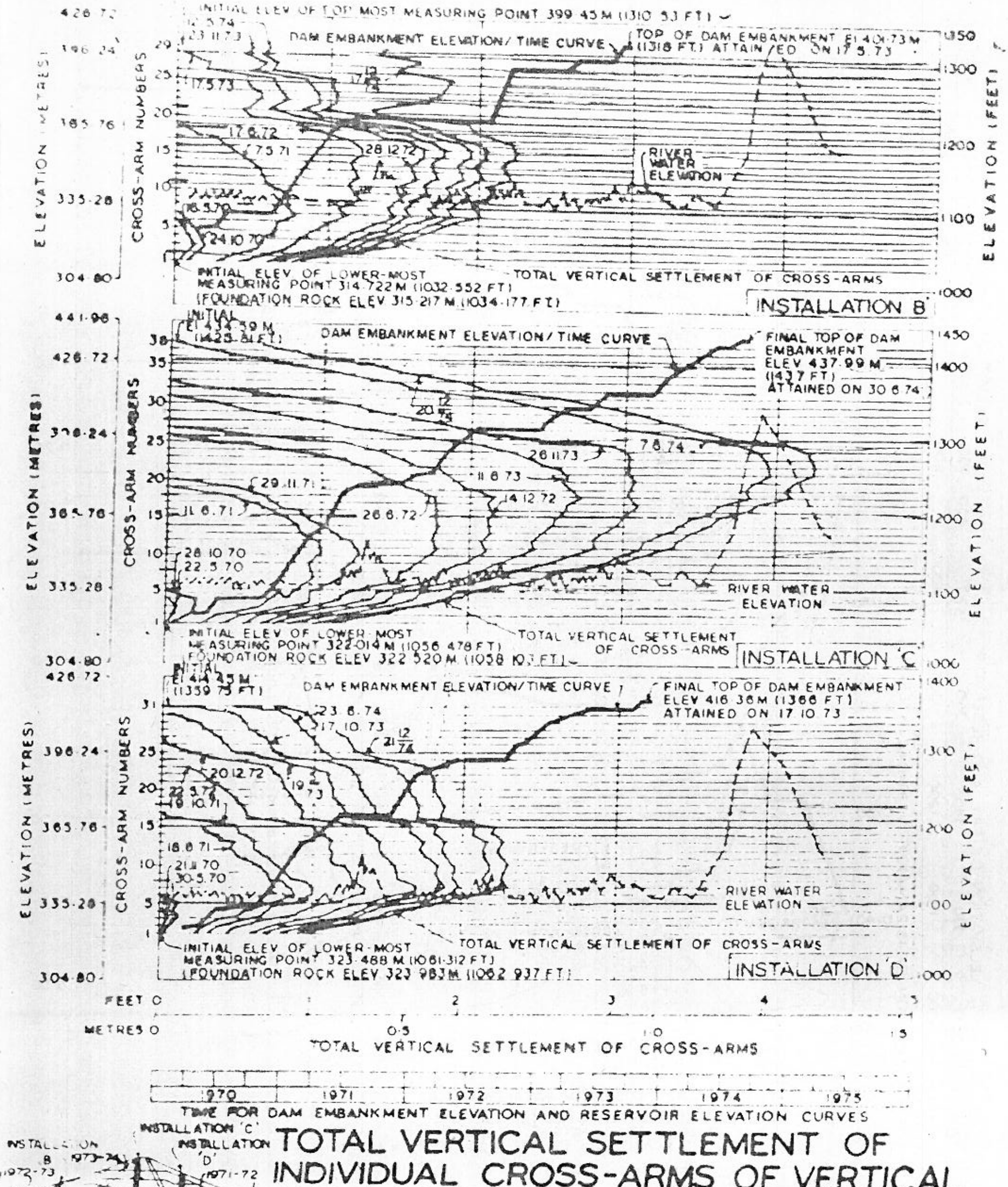
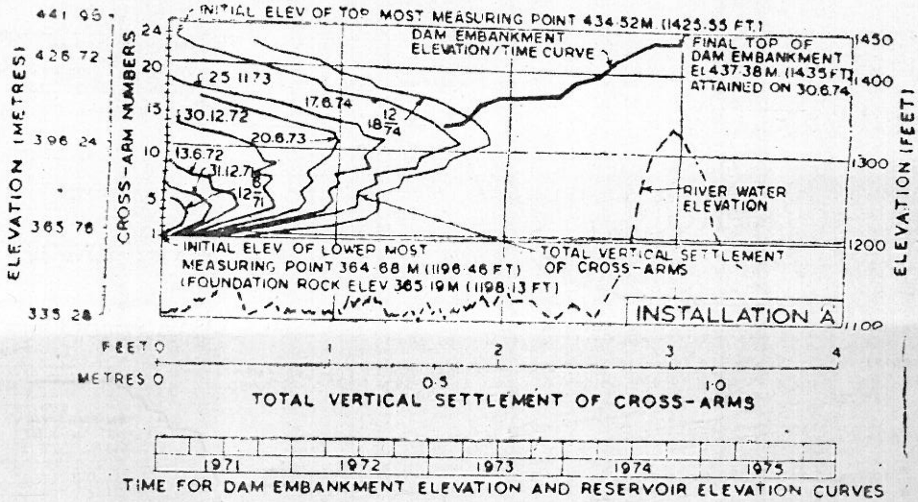


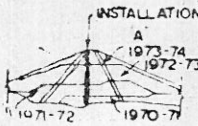
FIGURE 2. (Contd.) Summary plot of measurements of embankment compression during and after construction



TOTAL VERTICAL SETTLEMENT OF INDIVIDUAL CROSS-ARMS OF VERTICAL AND HORIZONTAL MOVEMENT DEVICES AT STN. 18+31·84 M. (60+10 FT.) ON SPECIFIED DATES

FIGURE 3 :





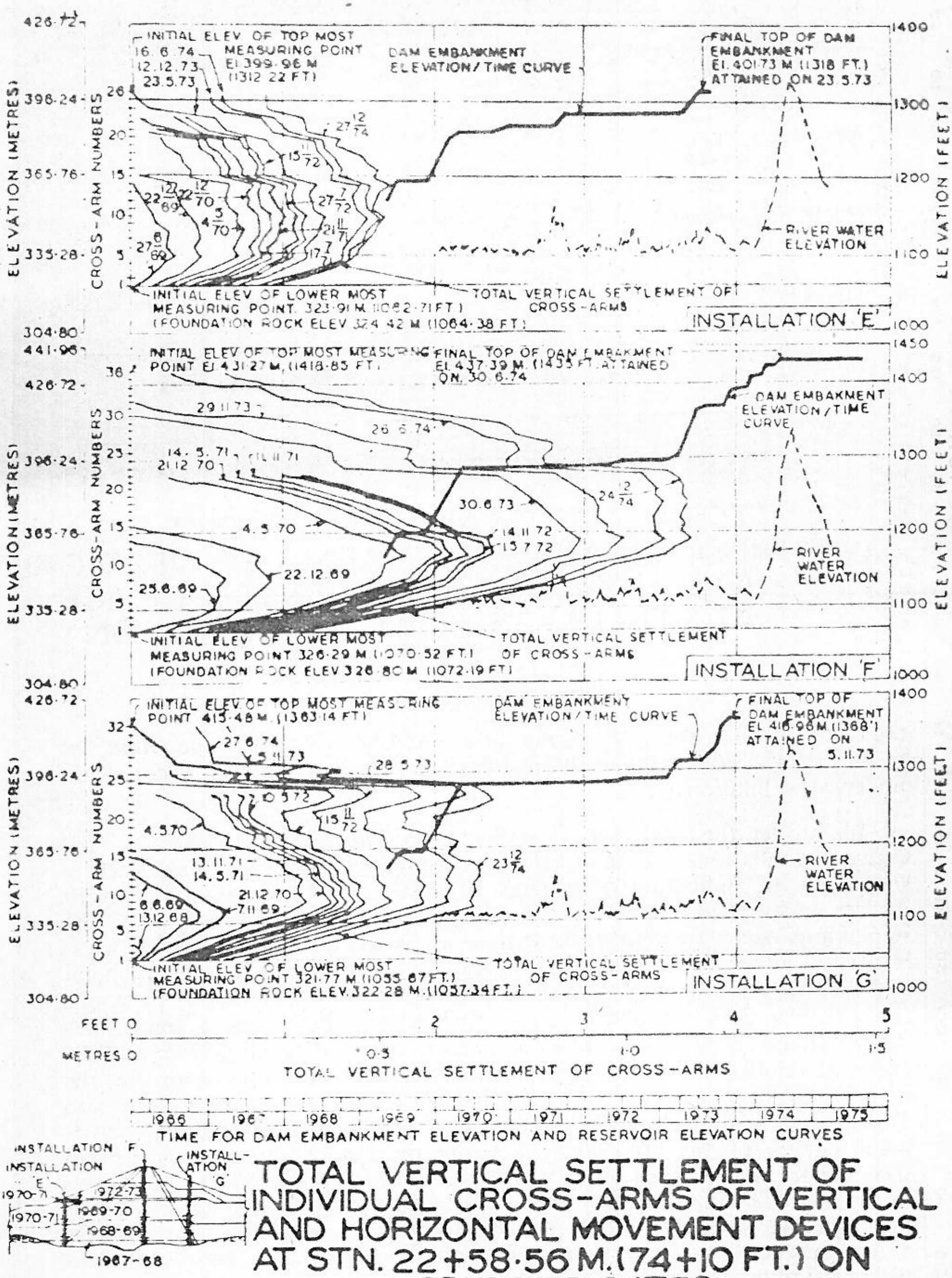
TOTAL VERTICAL SETTLEMENT OF INDIVIDUAL CROSS-ARMS OF VERTICAL AND HORIZONTAL MOVEMENT DEVICE AT STN. 14+35.60 M. (47+10 FT.) ON SPECIFIED DATES

FIGURE 3 (Contd.)

the depth of fill between the foundation and the cross-arm, the time lag of laying the lower layers with reference to the laying of the cross-arm under consolidation etc.

We plotted the total vertical settlement of individual cross-arms for the various installations. Figure 3 shows a typical plot of the total vertical settlement of individual cross-arms of vertical and horizontal movement devices on specified dates. A study of these plots indicates that for the impervious zone the maximum settlement occurs at the cross-arm which is located at about the mid-height or slightly above the mid-height point and the shape of the height versus settlement curves approximates parabolic curves. For right abutment dam section where there was a long shut-down period of around 3 years, a distortion of the parabolic shape of the curve was noticed. Maximum settlement of individual cross-arms in the impervious zone as observed was found to be more than 1.23 m upto 31.12.74, while in the pervious zone it was 0.69 m. Total vertical settlement curves of the cross-arms are smooth for the impervious zone and irregular for the pervious zone which is due to the much greater variations in the degree of compactness in the case of pervious material. No significant change was noticed due to impounding of water in the reservoir except that further compaction after completion of the dam has been found to have significantly reduced.

(*d*) The cumulative embankment consolidation measurements during construction were analyzed and it was concluded that the compression in the central portions of the dam in the impervious fill where the measuring



TOTAL VERTICAL SETTLEMENT OF INDIVIDUAL CROSS-ARMS OF VERTICAL AND HORIZONTAL MOVEMENT DEVICES AT STN. 22+58·56 M. (74+10 FT.) ON SPECIFIED DATES

FIGURE 3. (Contd.)

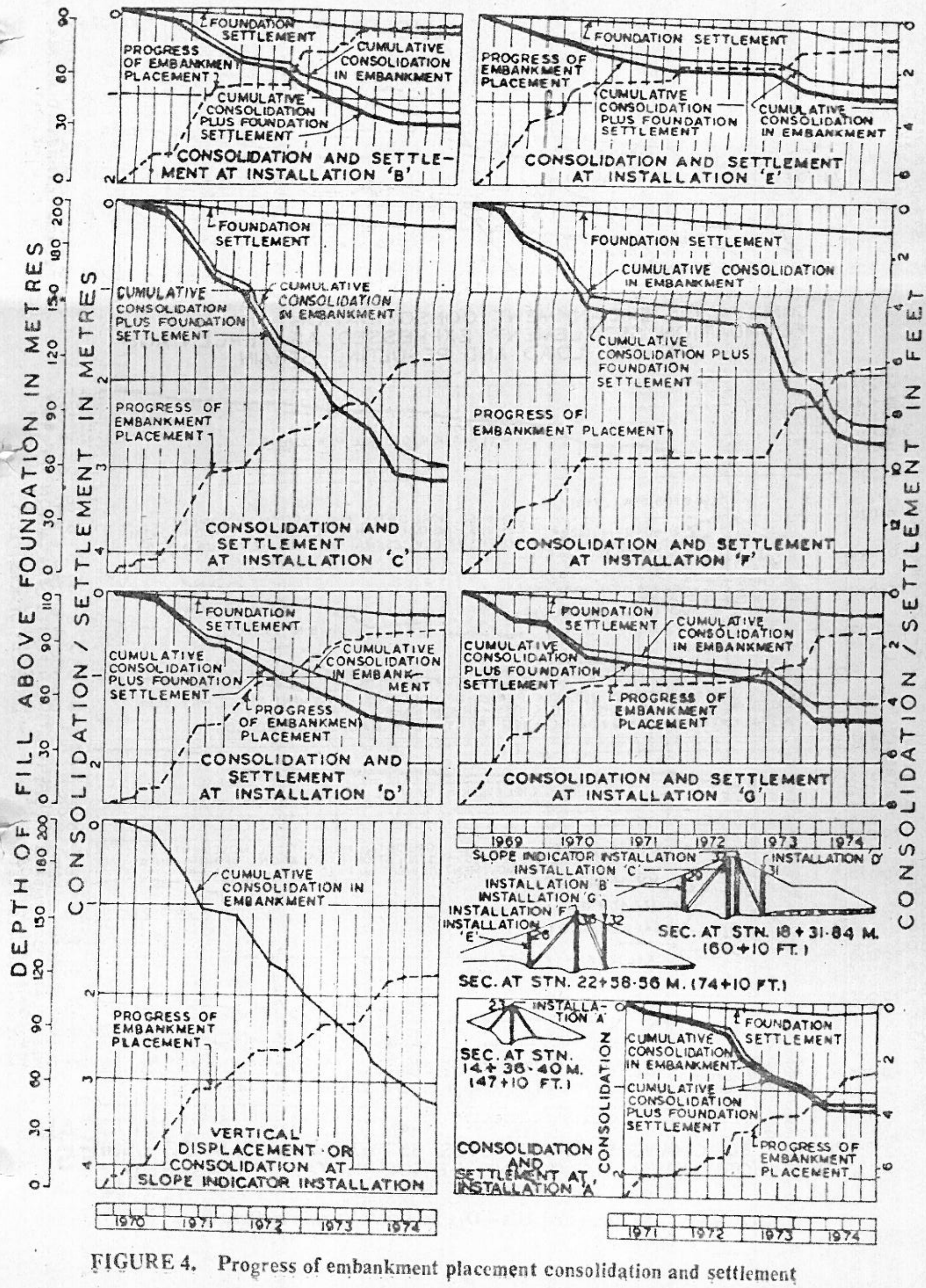


FIGURE 4. Progress of embankment placement consolidation and settlement

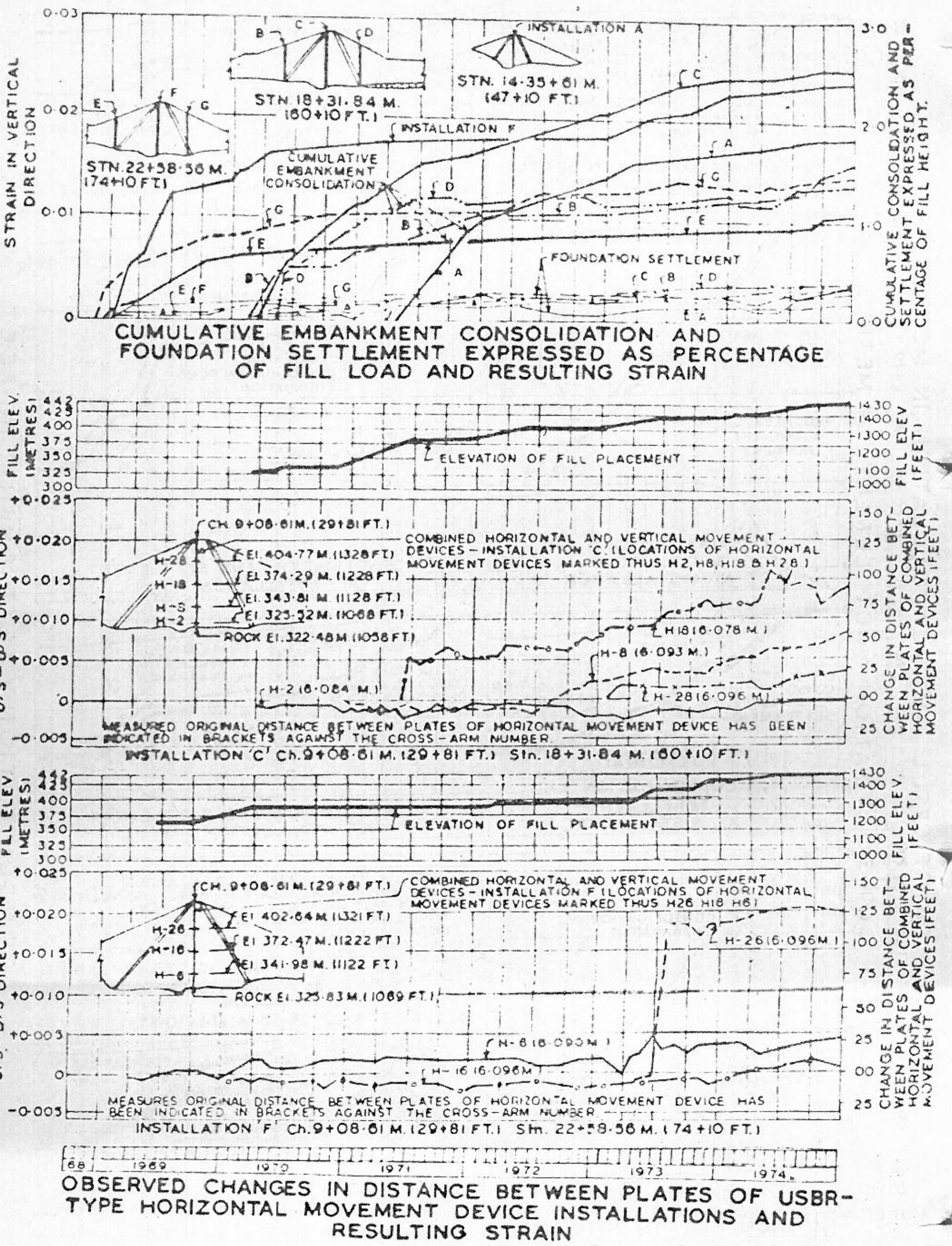


FIGURE 5. Vertical and U/s—D/s horizontal strain in Beas Dam

devices were installed, took place essentially in a vertical direction as though the material had lateral restraint except in central river-bed section of the dam where a truncated dam section with higher embankment elevations on the upstream side had to be adopted for adequate diversion requirements in the year 1970-71.

Figure 4 shows a plot of the progress of Embankment placement, Foundation Settlement, Cumulative Consolidation in Embankment and Cumulative Consolidation in Embankment plus Foundation Settlement against time for the various installations made in the Beas Dam.

Settlement of the foundation continued to increase with the increase in the load over the foundation. The foundation settlement took place at a nominal average rate of 1.22 mm for every 3.05 m rise in the fill load upto a fill height of nearly 30.48 m. Above 30.48 m height of fill, foundation settlement continued gradually at rates varying from 0.3 cm to 1.8 cm for every 3.05 m rise in fill height.

It was observed that the foundation settlement continued throughout the prolonged shut-down period of nearly 3 years in the right abutment dam section where the measured settlement increased from 8 cm to 18 cm in 1023 days at installation F with 64 m fill height.

The foundation settlement and maximum cumulative embankment consolidation expressed as a percentage of fill load or in other words the strain in the vertical direction for the various installations is shown plotted against time in Figure 5.

We also plotted the average measured compression as a function of the weight of an overlying column of embankment material. After subtracting the pore pressures measured in nearby piezometers, we obtained curves relating the vertical compression or strain and the vertical effective stress. A plot of the average consolidation or average vertical strain and vertical effective stress for the various installations in the impervious core of the Beas Dam is shown plotted on a log-log scale in Figure 6. The stress-strain curves approximate straight lines on log-log plots and can be defined by any two points, such as the compression at any two vertical pressures say 0.7 kg/sq. cm. and 7 kg/sq. cm.

Under 7 kg/sq. cm. the measured vertical compression for all the USBR dams has generally ranged approximately between 1.0 and 4.0% while in our case it ranges between 1.1 to 1.3% as per measurements made so far.

Factors Influencing Compression

It is well known that the compressibility of the embankment depends primarily on the compaction water content and the soil properties, and the absolute value of the dry density (or void ratio) of the embankment material has little or no influence.

Gould used USBR measurements on more than 20 large dams and concluded that in the range of stress below a value of approximately 7 kg/sq. cm., the compaction water content had the most important influence on compressibility. The compression curves of embankments constructed with relatively low average water contents indicated low initial strain and

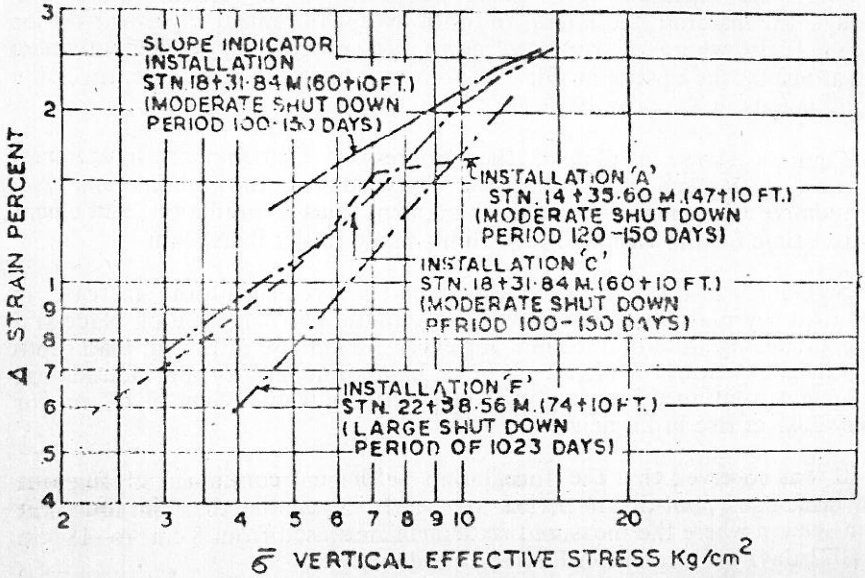


FIGURE 6. Average Consolidation versus vertical effective stress measured at various sections in the impervious core

constant or only moderately increasing compressibility under higher pressures. These curves were generally slightly concave in the upward direction on the log-log plot. The curves for embankments constructed with water content at or above Standard Proctor Optimum are characterized by high initial strains and have a slight concavity in the downward direction. Under pressures of 7 kg/sq. cm. or higher, however, compressibility was more or less independent of the compaction water content and was governed by the gradation and plasticity of the embankment soil. The plasticity of the fines was relatively more important than the coarseness and gradation. Gould determined the relationship between embankment compression and soil type as determined for measurements on USBR dams and grouped the embankments as shown in Table I roughly according to their soil properties in order of increasing compressibility.

TABLE I

	Embankment Soil Type	Apporximate Range of Measured Compression (%) at 0.70 kg/cm²	at 7.0 kg/cm²
1.	Silty gravel and coarse silty sand (GM and SM)	0.2-0.3	0.9-1.4
2.	Fine silty sand and silt of low plasticity (SM-ML)	0.2-0.5	1.3-2.1
3.	Clayey sands and gravels (GC-SC)	0.3-0.8	1.9-3.3
4.	Clay of low to medium plasticity (CL and CL-ML)	0.2-1.1	2.8-4.2

The impervious zone soil used in the Beas Dam falls in the category at S. No. 3 and S. No. 4 in the Table I. The range of measured compression at 0.7 kg/sq. cm and 7 kg/sq. cm respectively by us on the Beas Dam was 0.1% to 0.4% at 0.7 kg/sq. cm and 1.1 to 1.7% at 7 kg/sq. cm respectively.

Upstream—Downstream Horizontal Strain

Figure 5 also indicates the change in distance between plates of USBR type combined Horizontal and Vertical movement devices and the U/s—D/s Horizontal strain in the Beas Dam as measured in the combined Horizontal and Vertical Movement Device Installations C and F, plotted against time. H_2 and H_{23} at Stn. 18+31.84 m show insignificant horizontal movements while H_2 and H_{18} show some movement in the downstream direction which is apparently due to the surcharge load as a result of the truncated section adopted for dam construction during 1970-71 for adequate diversion requirements. H_{26} at Stn. 22+58.56 m also indicated some movement in the downstream direction which may also be due to the adoption of truncated dam section during the construction period above the cross-arm elevation.

Slope Indicator Installation and Horizontal Dis-placement of Beas dam Embankment

There have been no technical problems with the slope indicator installation. The data on the vertical movements as determined from inclinometer coupling surveys is in excellent agreement with the settlement pipe measurements on the horizontal and vertical movement installation C. The lateral movement data indicated in Figure 7 appears reasonable, but the following factors were thought to effect precision of the measurements. It is but natural that 110 m long inclinometer will not be installed exactly vertical. The initial eccentricity from the vertical varies and the initial position on date of installation of the various casing sections is indicated in the figure. Further, as the position of the inclinometer is computed by integration of recorded slopes from the bottom (i.e. fixed point) to the top, a very small bias in measuring system which is hard to detect by the field technician may accumulate to significant errors.

There is no relevant pattern in the horizontal movements given by the casings. Each casing has movements of different magnitude in either direction. In U/s—D/s plane, the movements of casings are usually in D/s direction whereas in right-left plane movements of lower casing are in left side while movement of middle and upper ones are in right side.

Surface Settlement

The simplest and easiest measurement to gauge the performance of an earth dam is by determination of surface movements from a series of measuring points at established locations on the final surface of the finished embankment. Periodic check of the location of these measuring points with level transit and chain will indicate total settlement and longitudinal movement. The data is not so valuable because of the lateral and impossibility of ascertainining whether movement is deep seated or superficial and because the magnitude of motion is normally within the limit of error of the measuring method.

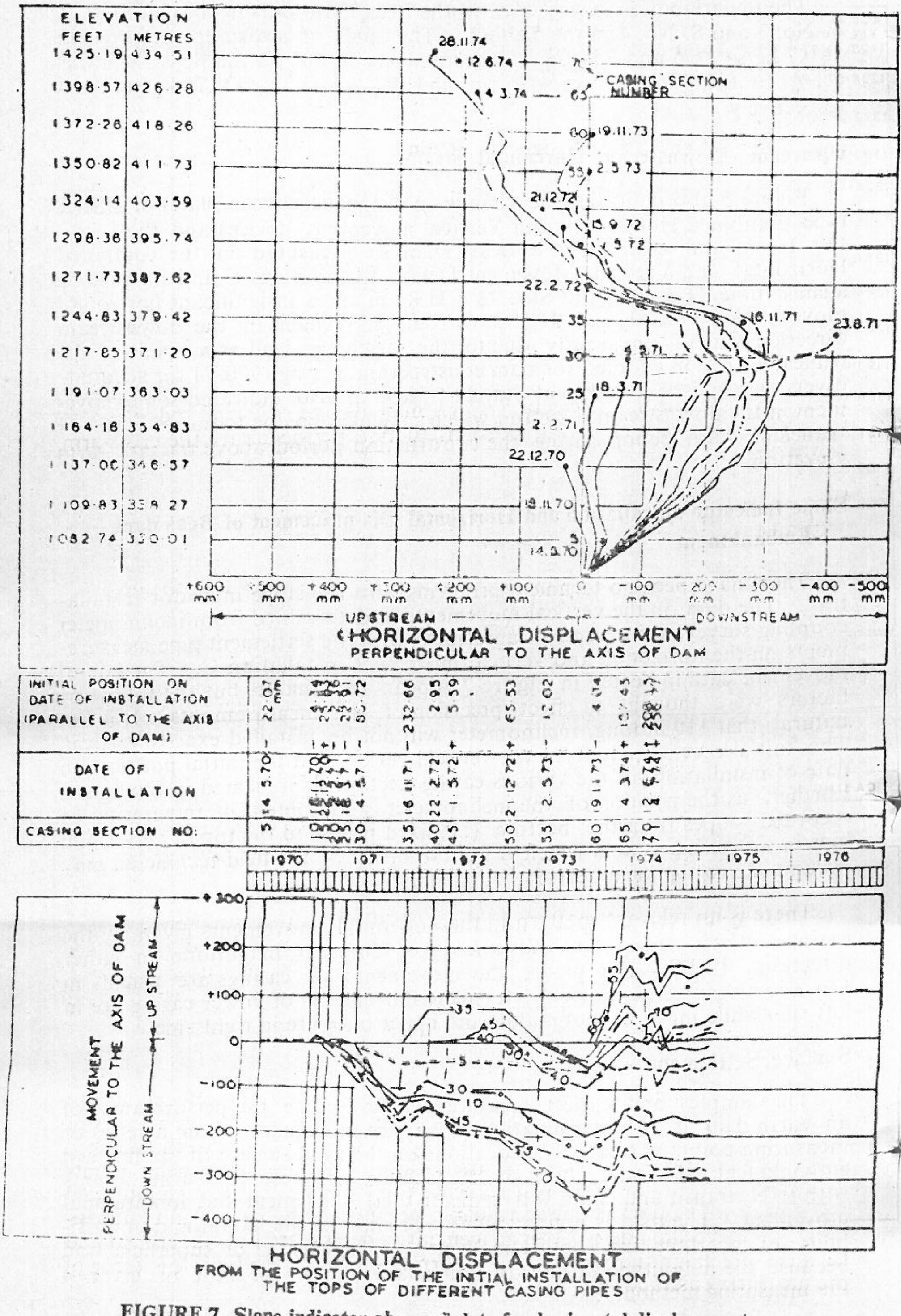


FIGURE 7. Slope-indicator observed data for horizontal displacement

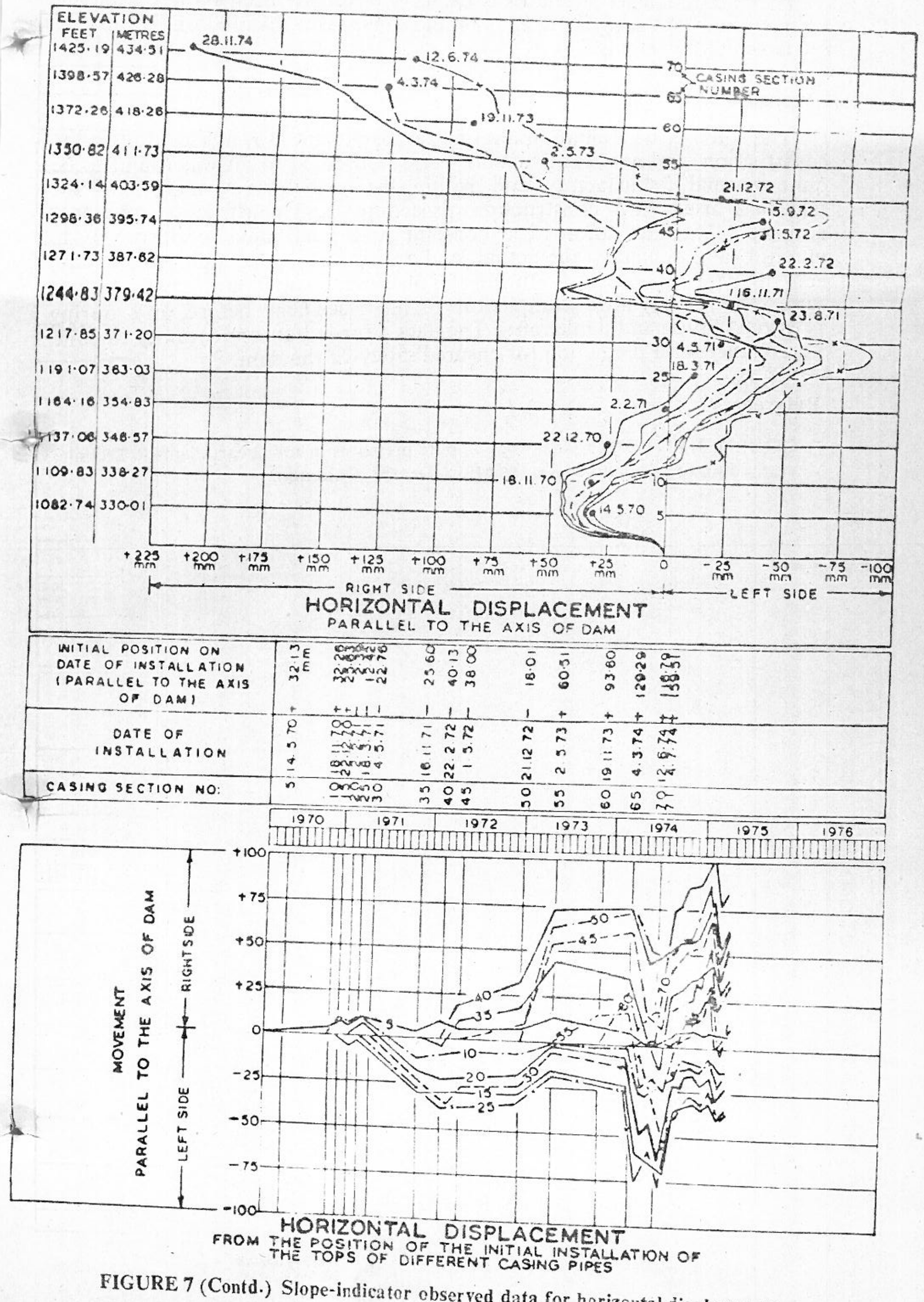


FIGURE 7 (Contd.) Slope-indicator observed data for horizontal displacement

The measurements on the Beas Dam have recently been started after the completion of the dam in June, 1974 and subsequent installation of Surfaee Settlement Movements.

Conclusion

The soils instrumentation data for embankment movements during the construction and post-construction period observed so far was found to be quite normal, satisfactory and within the anticipated values for the materials used and construction procedures and construction sequence adopted. The data afforded a constant and continuous opportunity to monitor and document the actual and anticipated behaviour of the dam embankment progressively during fill placement, after shut-down periods and subsequently after completion of fill placement before and during reservoir filling and depletion. The data afforded an opportunity to compare reality with prediction for ensured safety of the dam.

References

GOULD, J. P. (1959) : "Construction pore pressures in rolled earthdams," Technical Memorandum 650, Bureau of Reclamation, Denver, Colorado.