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Advances in the Aplication of Electrolevels as a Monitoring System for Concrete Faced Rockfill Dams

Abstract: A considerable number of very large concrete faced rockfill dams have been constructed involving millions of cubic meters of compacted rockfill. This has stimulated the need to understand the deformation characteristic of the compacted rockfill and the prediction of the deflection of the concrete face. Difficulties/ limitation in using normal laboratory equipment to study the stress-strain-time behaviour of compacted rockfill, have led to the development of several/types of field instrumentation. The most effective procedure of understanding and of predicting the behaviour of these dams is the development of numerical methods based on the back analysis of the results obtained from field observation. In this paper it is commented on the potential use of the electrolevel for monitoring the face deflection, during construction period; reservoir impounding and long term operation.

Key words: Electrolevel, rockfill dam, field instrumentation, curve fitting, slab deflection.

1. Introduction

As pointed out by Cooke (2000), there are some inherent safety features in engineered Concrete Face Rockfill Dams (CFRD), favouring its choice, as compared to other types of dams. These inherent safety features are: the dam body is comprised of zoned rockfill providing high shear strength against static and seismic loading; the concrete face acts as the impervious membrane preventing water seepage into the dam body and, in conjunction with the nature of rockfill, controlling pore pressure build-up and subsurface erosion. In addition, the position of the impervious element outside the dam body makes the construction sequence and related foundation treatment less dependent of the rockfill placement, providing construction advantages concerning time schedule and cost.

However, the high hydraulic gradients, acting under the plinth, along the perimeter joints and on the concrete face may cause leakage specifically if defects exist, and this can be seen as a less favourable feature. The compacted rockfill is the structural element of the dam and must be designed to withstand the reservoir water loading with minimum settlements, so as to avoid crack development in the concrete face, which acts as the impervious element.

Despite these challenges, a considerable number of very large CFRD have been constructed in the past three decades involving hundreds of millions of cubic meters of compacted rockfill. This has stimulated the need to understand the deformation characteristics of compacted rockfill (structural element) and to predict displacements of the concrete face (impervious element).

He large grain sizes of the rockfill present a challenge to undertaking laboratory testing to determine its properties and behavior under field conditions. Instead, information about full scale behaviour can be obtained from carefully planned instrumentation. Measurements obtained from field instrumentation programs allow suitable models to be developed to monitor the behaviour of the dam.

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One of the most important instrumentation aspects in CFRDs is related to the measurements of the face deflection during construction and reservoir impounding.

2. Measurements of Face Slab Deflection.

Several attempts have been made to measuring the face deflection. Most of these measurements are based on the use of adapted inclinometers passing through access tube attached, either to the surface or to the back of the face membrane by steel brackets (see Figure 1). Such method involves the combined measurement of vertical and horizontal movements at discrete point on the rockfill, just behind the face membrane. Penmam and Rocha Filho (2000) have provided a comprehensive review about such procedure. Some examples are given as follows.

Fig. 1 Inclinometer tube fixed on the concrete slab.

The deflection of the reinforced concrete membrane of the 110 m high Cethana dam in Australia was measured by two methods, described by Fitzpatrick et al (1973). In one, measurements were made on floats connected to 23 points on the membrane by stainless steel wires. Complete calm was required when readings were taken by topographical surveying.

The second method used a specially made inclinometer of 1.525 m gauge length, which was passed through access tubes of 76 mm diameter attached to the surface of the membrane by steel brackets. It was claimed that after making corrections for the effects of temperature and tension of the vertical wires, there was excellent agreement between the two methods for measuring membrane deflection.

Deflections of the steel membrane of the 45 m high rockfill Aguada Blanca dam in Peru were measured by a deflectometer passed through tubes placed on the 1 Vertical on 1.7 Horizontal slopes, immediately below the steel membrane. The deflectometer consisted of a 0.5 m long body, guided in the tubes by two sets of three ball bearings; with a 0.5 m long flexible arm extending forwards to measure bend of the tubes. Movement of the arm relative to the body of the apparatus was measured by couples of vibrating wires.

The 53 m high Winscar dam was one of the first British dams to have an upstream membrane of asphaltic concrete. The rockfill dam with an upstream slope of 1 on 1.7 was fitted with horizontal plate gauges at three levels on the main section during construction. As well as giving information about movements within the body of the rockfill, these gauges, which were carried through the rockfill to special terminal units placed immediately below but in contact with the asphalt membrane, enabled membrane deflections to be measured at three approximately equally spaced heights.

An inclinometer system similar to that used at Cethana dam was planned for use at the 131 m high Khao Laem rockfill dam in Thailand. Four lines of 56 mm diameter aluminium tubes were attached to the 1 on 1.4 sloping concrete membrane with brackets to ensure that four grooves in the tube lie on planes parallel and normal to the membrane. The inclinometer had a gauge length of 0.5 m, advanced through the tubes in 0.5 m increments, a reading being taken at each increment. The longest tube requires 430 readings. This information was given by Ohdedar and Dawes (1983).

The 72 m high Marchlyn dam, in north Wales, had the waterproof element consisting of a grout curtain placed below a concrete inspection gallery structure at the upstream toe, with an asphaltic concrete membrane attached to it and covering the upstream slope.

To measure the amount of damaging differential settlement that would occur adjacent to the rigid concrete gallery structure, and to measure the deflected shape of the membrane while the reservoir was full, together with overall movements of the dam, a system of precise topographical surveying was established using stable reference pillars founded in bedrock away from the loading influence of the dam, and a face measuring system based on an inclinometer, mounted on a four-wheeled trolley so as to put the inclinometer in a vertical position (see Figure 2).

A standard commercially available inclinometer of 0.5 m gauge length that gave digital readout offset from vertical in increments of 0.1 mm was chosen

The trolley (shown in Figure 2) was lowered by a waterproof-screened cable from a winch in a vehicle on the dam crest. There were 69 reading positions and the lowest one, when the front wheels were on the rail bolted to the concrete gallery structure, was taken as the reference point for the deflection measurements. At the highest position, when the trolley was close to the dam crest, its position was measured with the system of precise surveying. Because the effective length of the trolley was four times that of the inclinometer gauge length, a complete set of reading was obtained from four trolley traverses, turning the inclinometer to face the other way between each traverse to eliminate any zero error.

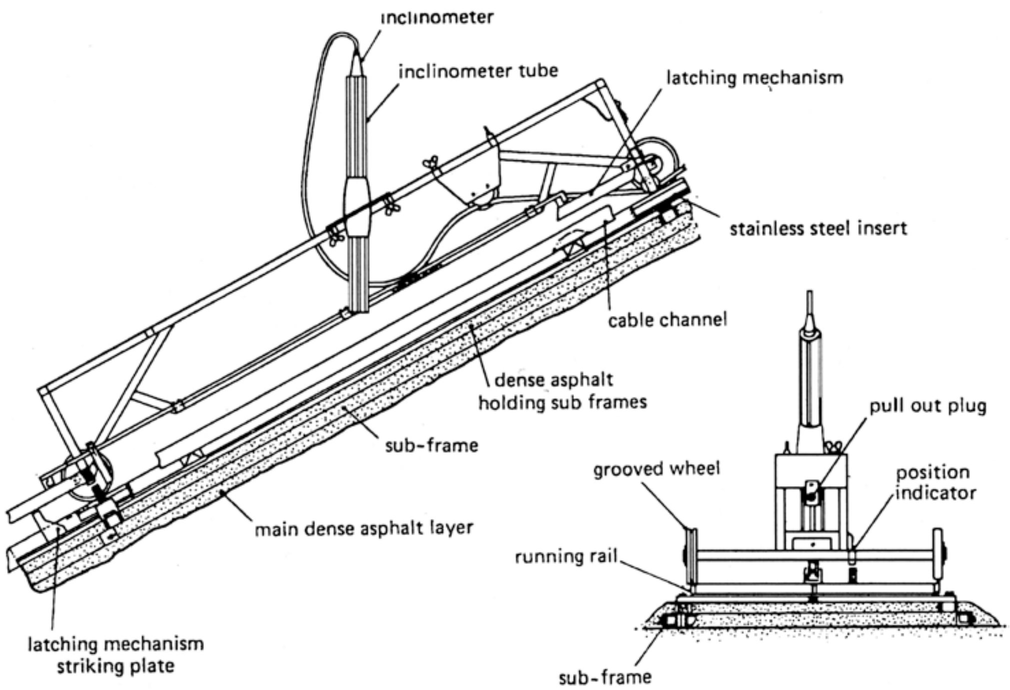


Figure 2 – Diagram of the trolley.

3. Electrolevels

Deflection of the concrete face during: construction period; reservoir filling and long term behaviour can also be measured by a monitoring system that uses electro-levels (EL). The electro-level is a gravity sensing transducer, consisting of a small glass sealed tube partially filled with an electrolyte (iodine and ketone), which has a high electrical resistance. Three or four electrodes fitted in the longitudinal axis of the tube are used to measure the electrical resistance of the liquid between two consecutives pairs of electrodes, forming a half Wheatstone bridge, as indicated in Figure 3. As the tube is tilted, the amount of liquid between the electrodes changes, altering the electrical balance in each leg of the Wheatstone bridge. The relation between rotation and output voltage response can be obtained by a calibration procedure. Electrolevels can be calibrated automatically, by mounting a set of them on a beam, say 3.0m long, drilled to suit the dowels and with clamping bars to hold each cylinder tightly to the side of the beam or alternatively, using a rotating wheel, where the angular changes of the wheel are controlled by a stepper-motor. In both calibration procedures, either the amount of raising and lowering the beam or the actual rotations of the wheel can be checked against a standard EL or a calibrated displacement transducer.

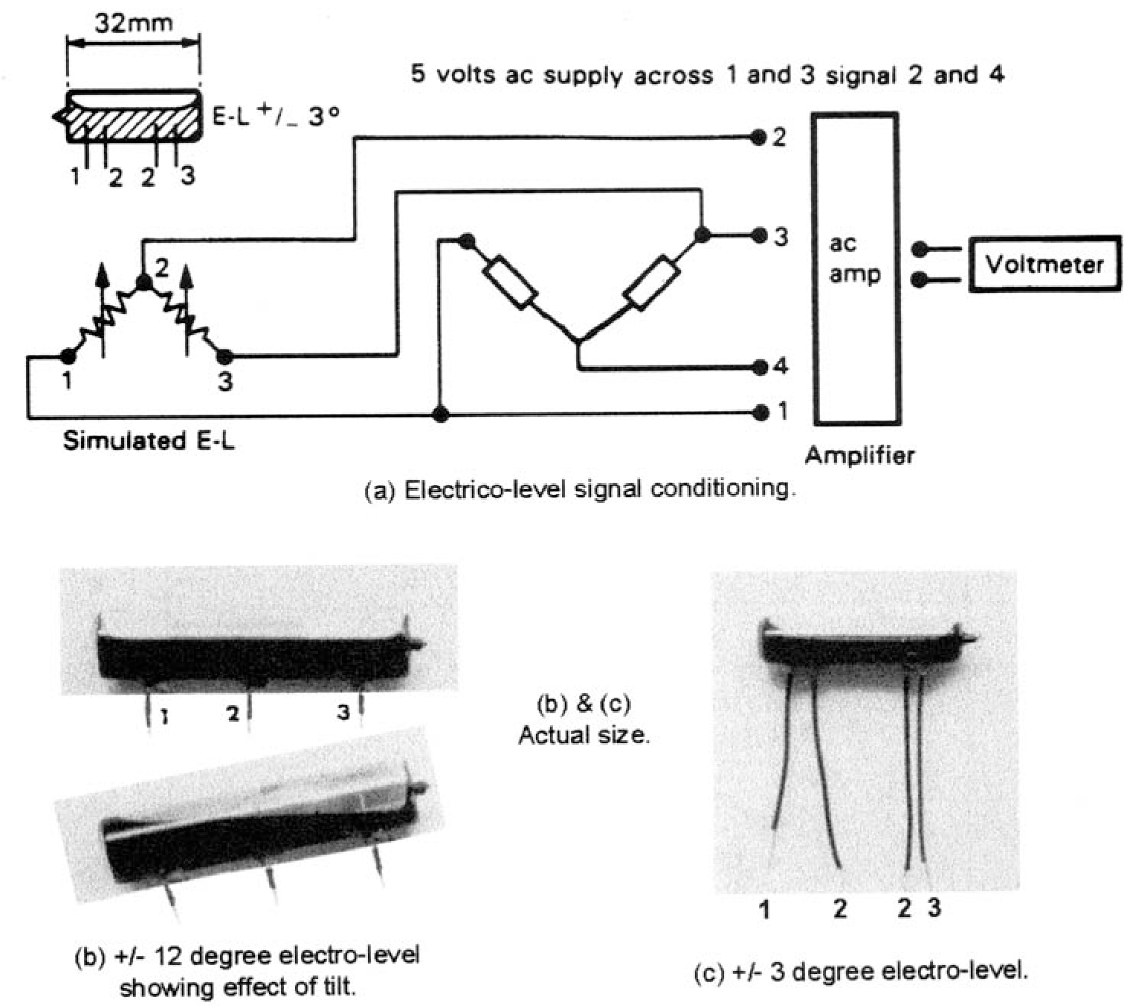


Figure 3 – Electro-level and Wheatstone's bridge.

In order to provide mechanical protection, the electro-level is housed in a short brass cylinder, positioned so that the glass tube lies across a diameter of the brass cylinder.

The connecting cable is brought out through a watertight gland at the side of the cylinder, which is closed by screwed lid bedding on-to an O-ring. The back of the brass cylinder is machined as to leave a central dowel projecting from its centre.

The linearity and sensitivity of the electrolevel depends on the type and amount of electrolyte and also on the axial curvature of the tube. To achieve the full potential use of electrolevels in civil engineering, specialised electronics have been developed over the last few years. These developments relate to control activation; current consumption; temperature effect and also loggers and software to record and process data from a system so that it can be used for real time control or long term automatic monitoring. Electro-levels are now manufactured in many shapes and sizes, with rotational ranges from ± 1 arc degree upwards. Those typically used for measurements of slab deflection for rockfill dams are about 30mm long and 6mm diameter, with a range of approximately ±3 arc degrees giving an overall level change of ±52mm over a metre length, which can be resolved as 0.01mm, for a resolution of ±1 arc second.

Each of these, like a spirit level, consists of a cylindrical glass capsule, containing an electrolytic fluid, which moves along the capsule by gravity when it is tilted. Three electrodes, at the ends and centre, are used to measure the relative resistance of the two parts of the electrolyte, thereby indicating the tilt of the capsule.

These two halves are used as two of the arms of a Wheatstone’s bridge, connected by a three- conductor cable to a remote read-out station. The resistance of the electrolyte in the capsule is very high as compared with that of the cable, so that small variations of cable resistance caused by temperature changes or some stretching have little effect on the accuracy of the instruments.

The advantage of using electro-levels to assess engineering performance is in the simplicity of fixing the devices to the material or structure being monitored, provided certain fundamental principles are adhered to. When inserted in soils subjected to a known load they can be used to determine in-situ shear modulus, when clamped on to beam elements they can be used to calculate deflections, bending moments, shear and loads on those elements.

4. Field Installation Procedure.

For the installation of electro-levels bolted to the concrete slab of the dam, it can be used aluminium angle of 63.5mm side and 6.35mm thick, cut to lengths of 120mm, for mounting the units on to the concrete slab. The vertical side was drilled to suit the dowel and also drilled and tapped to take bolts for the clamping bar, as indicated in Figure 4 (a to d). The horizontal side, cut to a width of 32mm, is drilled for two holding down bolts that are screwed into prepared holes in the concrete slab.

The units are attached at specified intervals and each unit is adjusted to bring the glass capsule approximately level, and then clamped firmly to the angle bracket. The cables are carried up the slope, terminating in a connecting box secured in a safe position on the crest wave wall. A metal cover attached independently to the concrete protected each unit. It iwas open at the ends to give free access to the rising water and finally, each unit is embedded in a concrete block for protection. The electrical connecting cables for a given section can be carried up the slope to the instrumentation house on the dam crest, either, in a shallow trench, made during the slab slip-forming work and backfilled with concrete, or laid down on the surface of the concrete slab and then protected by using half section of galvanized pipe (see Figure 4d)

a) Installing and levelling the EL b) EL mounting bolted to the concrete slab

c) Metal cover protection. d) Concrete block and cable protections.

Fig. 4 The drilled vertical side and the bolts for the clamping bar.

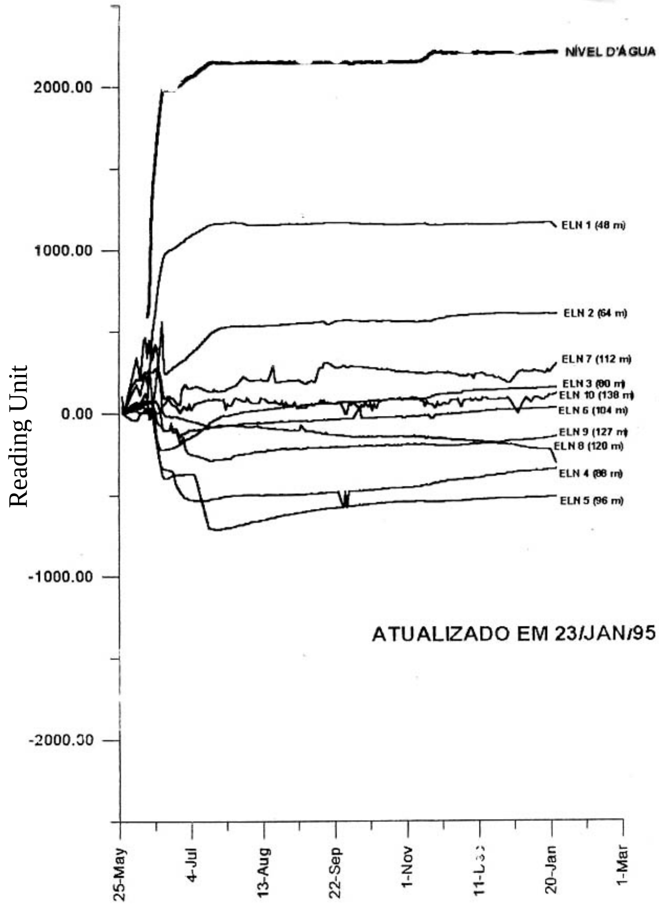
5. Interpretation of Results

Deflection of the concrete face during construction period and reservoir impounding causes small rotation of the electro-levels, and as a first and practical engineering approach it is most useful to plot readings (the results directly from the acquisition system) versus time, indicating all events that take place such as: placement of rockfill during construction, level of the reservoir, etc. Some examples are presented in Figure 5.

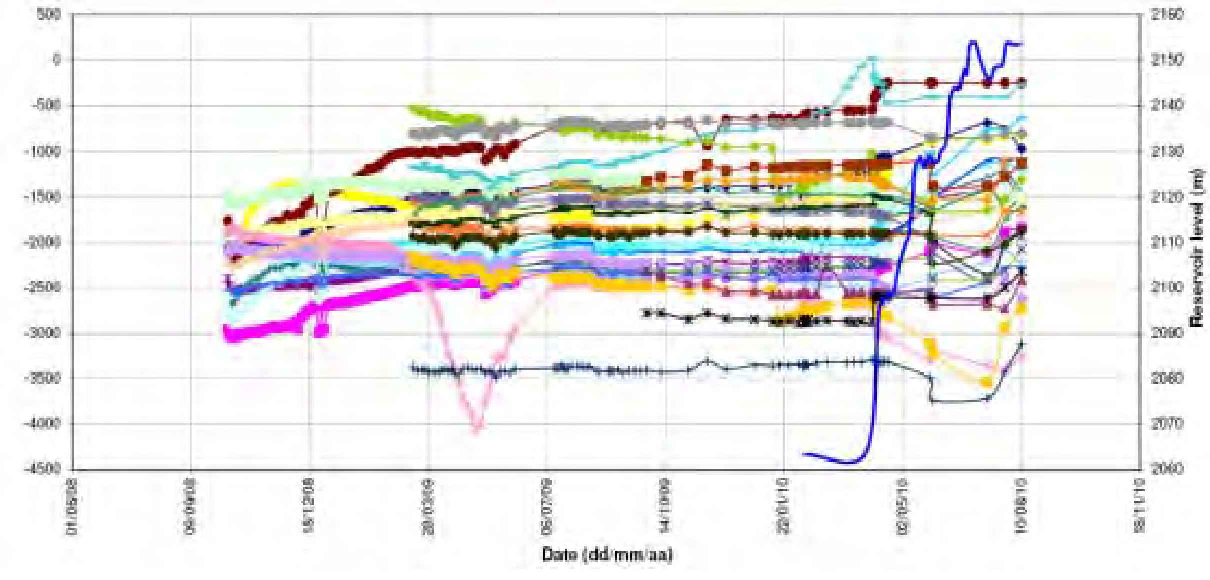
In order to calculate the deflection of the concrete face, the incremental variation of angle, in radians, of each individual electrolevel, for a given time, is plotted against distance up the sloping concrete.

Two different interpretations procedure can be used. The first is to consider the concrete slab between two consecutive electrolevels to act as a rigid bean, and deflections can be incrementally indicated by (see Figure 6):

$$d_k = \sum_{i=1}^{k-1} \tan \phi_i \cdot s_i \qquad (1)$$



(a)



b) Mazar Dam.

Figure 5 – Variations of measured readings with time. Details need to be clearer; Y axis should show units.

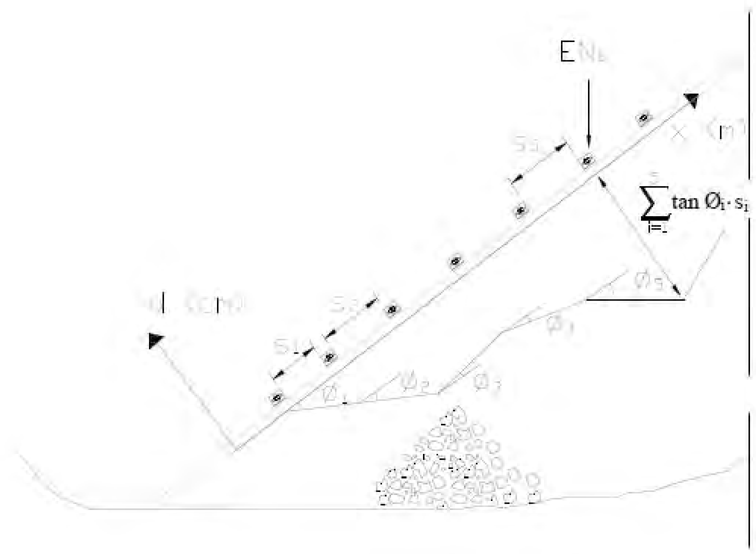


Figure 6- Deflection obtained by incremental procedure.

This very simple interpretation procedure has the advantage to eliminate any mathematical complexity involved in the calculation of the slab deflection. In the order instance, it does not provide any information that may occur between two consecutives electrolevels. So this calculated deflection can be taken as reference value establishing limits values of the slab deflection for a given set of data.

The measured changes from the initial condition of the angles of rotation (θ radians) of each of the ten electro-level gauges were plotted with distance (z) along the sloping slab. This set of measured values was used to establish a polynomial curve in a curve fitting exercise, as indicated in Figure 7 and mathematically expressed as:

$$[\ \theta = F(z) = Az^{n} + Bz^{n-1} + Cz^{n-2} + ... + Gz^{n-n}\ ] \quad (2)$$

where:

θ is the measured angles of rotation (in radians), z is the distance, from the base plinth up to the dam crest, along the sloping slab,

A to G are coefficients of the fitted polynomial,

n indicates the order of the polynomial curve.

Generally, a fifth to sixth order polynomial curve (n=5 or 6) is found to be the one that gave satisfactory fits. This satisfactory polynomial curve is then integrated to produce a continuous deflection curve (Equation 3), or by a mathematical derivative process, to indicate the bending moment acting on the concrete slab (Equation 4).

$$[y = \int F(z) + K] \text{ – Continuous deflection curve.} \quad (3)$$

The constant of integration (K) can be determined, either by assuming no displacement of the plinth (y=0 for z=0) or by considering values of deflection measured either by the use of three-dimensional perimeter joint metre (y= measured for z=0) or by surveying at the dam crest (y=measured for z=L, being L the total length of the slab at the corresponding instrumented section).

To calculating the bending moment acting on the concrete slab the fitted polynomial curve is derived and then multiplied by the mechanical properties (Young's modulus) and by the geometrical characteristic (moment of inertia of the cross section) of the concrete slab, to give:

$$M = (E \cdot I)\frac{d\theta}{dz} \quad (4)$$

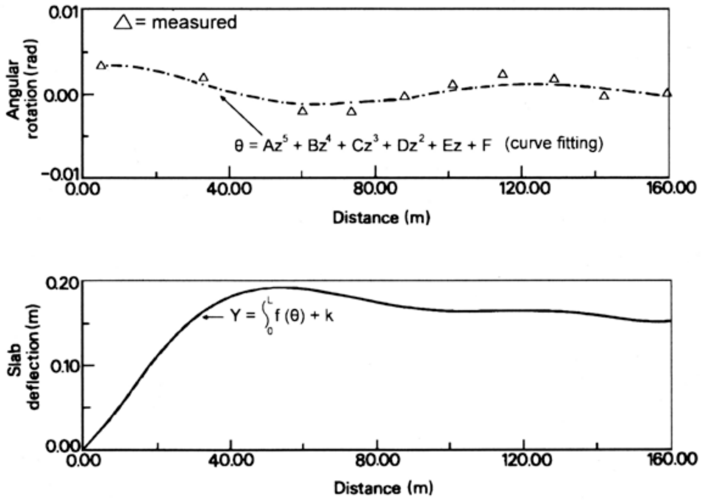


Figure 7 – Rotations of ELs plotted against distance up the sloping slab.

Figure 8 presents a comparison between these two distinct interpretation procedures.

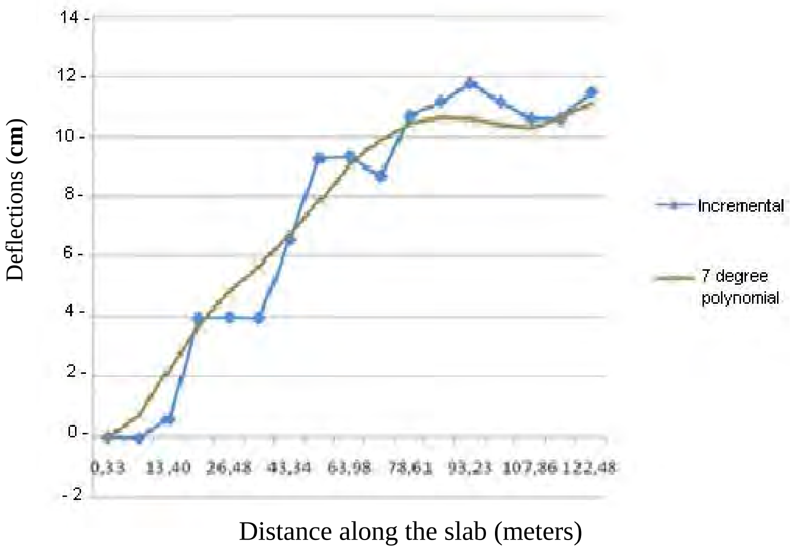


Figure 8 – Comparing incremental and polynomial procedures; Distance at Electrolevels need to be shown clearer; deflection is better shown in normal format not Scientific.

where:

E is the Young's Modulus and I is the moment of inertia.

Results obtained in three large dams: Xingo Dam (150m high) in Brazil, Tianshenqiao Dam (176m high) in China and Mazar Dam (166m high), in Ecuador, are presented.

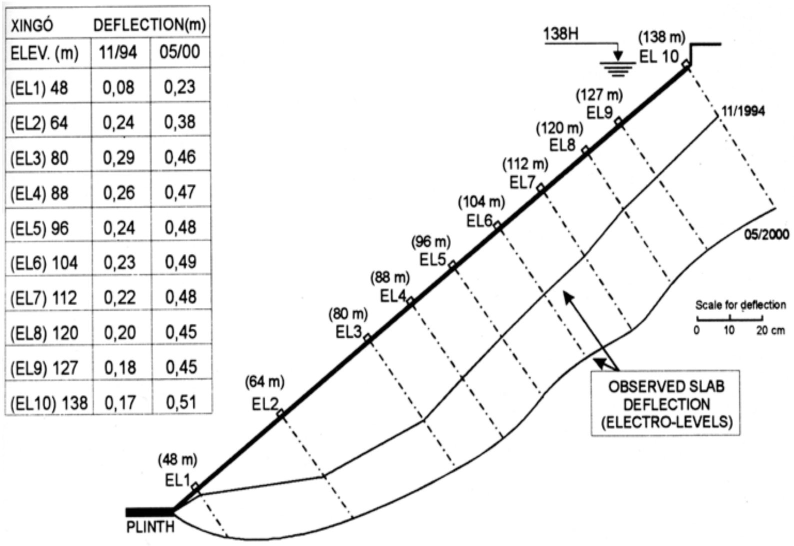
5.1 Xingo Dam.

The Xingo Dam Project is located on the San Francisco River, in the northeast of Brazil. It has a maximum height of 150m, involving 12.6 millions of cubic meters of a gneiss rockfill.

A set of ten units was installed up the sloping 160 m long face slab at dam section C62+00. Figure 9, presents the calculated slab deflection just after the reservoir first filling and for long term condition. Fuller details of this instrumentation have been given by Rocha Filho (1995).

5.2 TSQ1 Dam.

This dam-was constructed in the southeast of China, to a maximum height of 178 m, involving 17.7 millions of cubic meter of compacted limestone fill, was instrumented with 64 electro-levels on three sections. Measurements of rotations have been obtained for several stages of construction; see Figures 10 to 13. The slab deflections as indicated by the full set of electrolevels, for the three instrumented sections, have been presented by Wu et al (2000), including observed values for two reference dates: just after completion of the final stage of the slab construction and with the reservoir water at the maximum level (see Figure 14).



XINGÓ	DEFLECTION(m)	
ELEV. (m)	11/94	05/00
(EL1) 48	0,08	0,23
(EL2) 64	0,24	0,38
(EL3) 80	0,29	0,46
(EL4) 88	0,26	0,47
(EL5) 96	0,24	0,48
(EL6) 104	0,23	0,49
(EL7) 112	0,22	0,48
(EL8) 120	0,20	0,45
(EL9) 127	0,18	0,45
(EL10) 138	0,17	0,51

Figure 9 – Calculated slab deflection. Short and long term behaviour – Xingo Dam

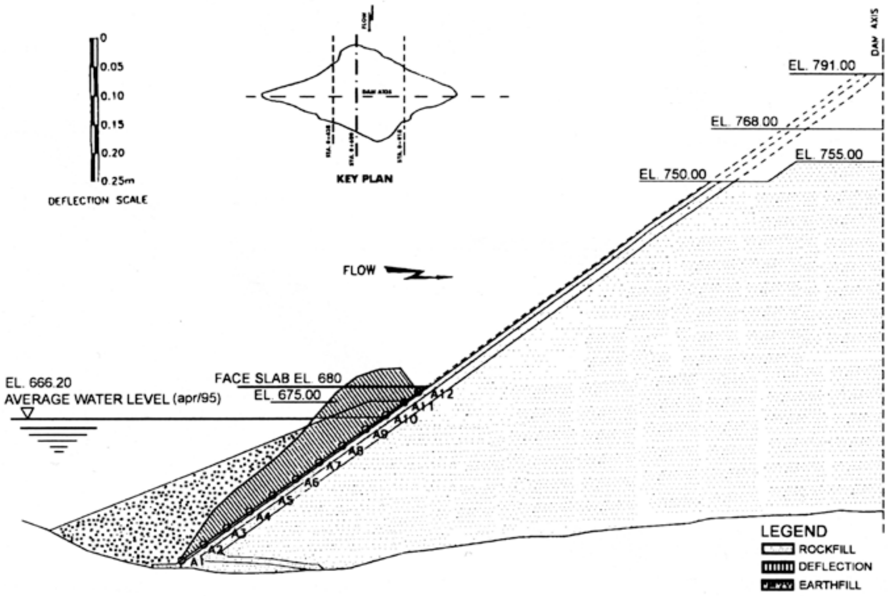


Figure 10 – Tianshengqiao dam. Deflection of 1st stage slab on major section.

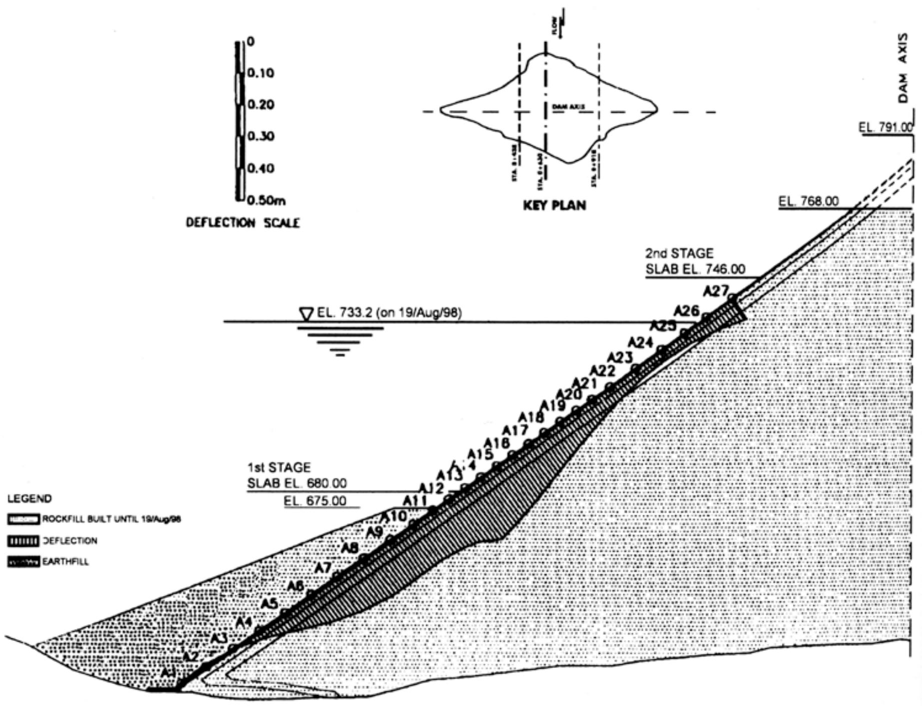


Figure 11 – Tianshengqiao dam. Deflection of 1st & 2nd stage on major section.

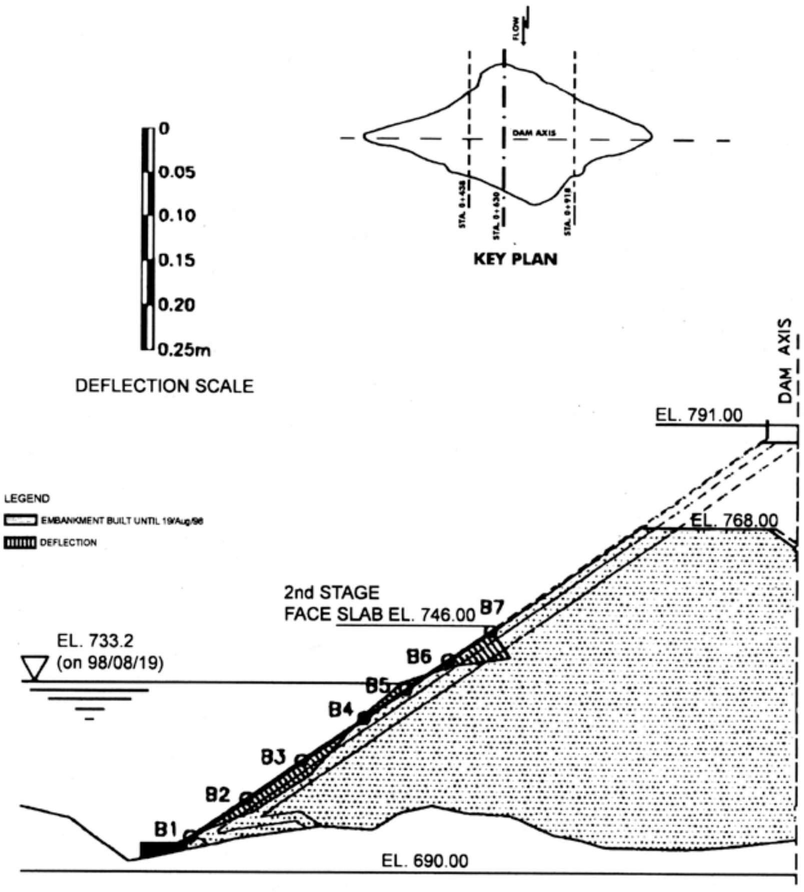


Figure 12 – Tianshengqiao dam. Deflection of 2nd stage slab on section B.

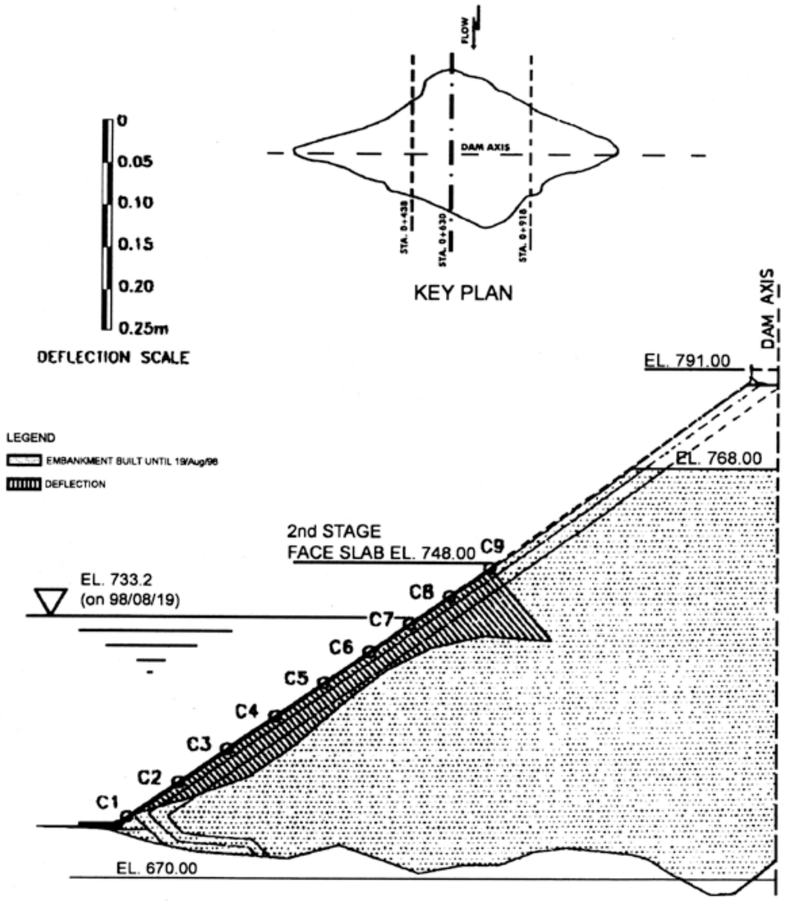


Figure 13 – Tianshengqiao dam. Deflection of 2nd stage slab on section C.

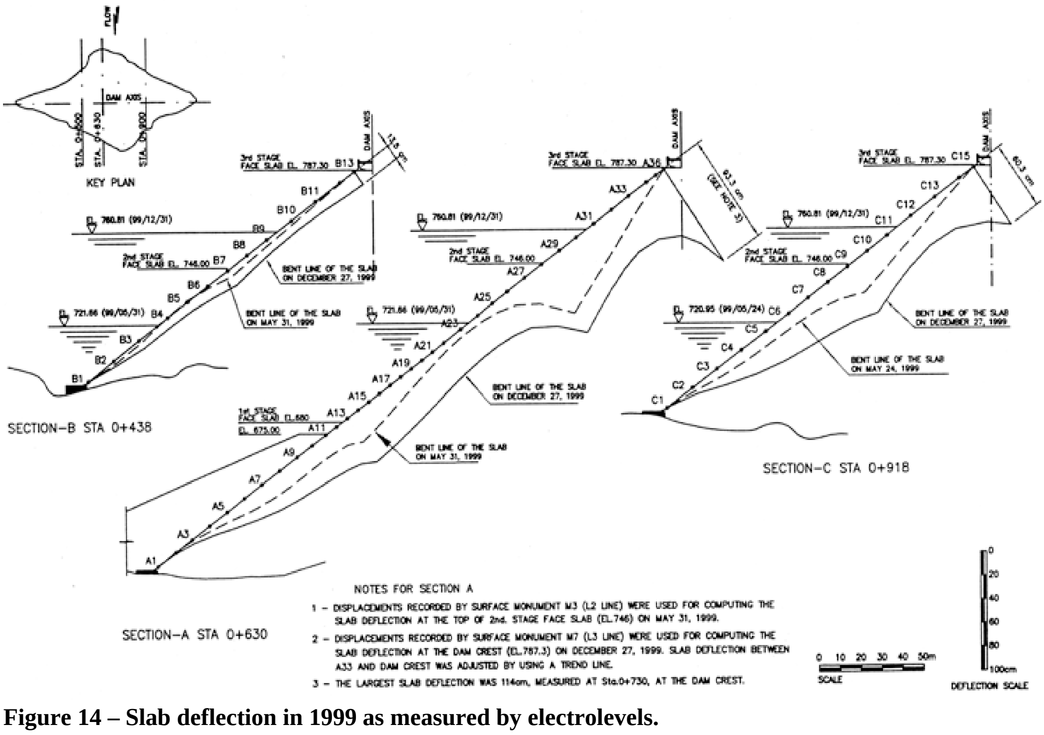
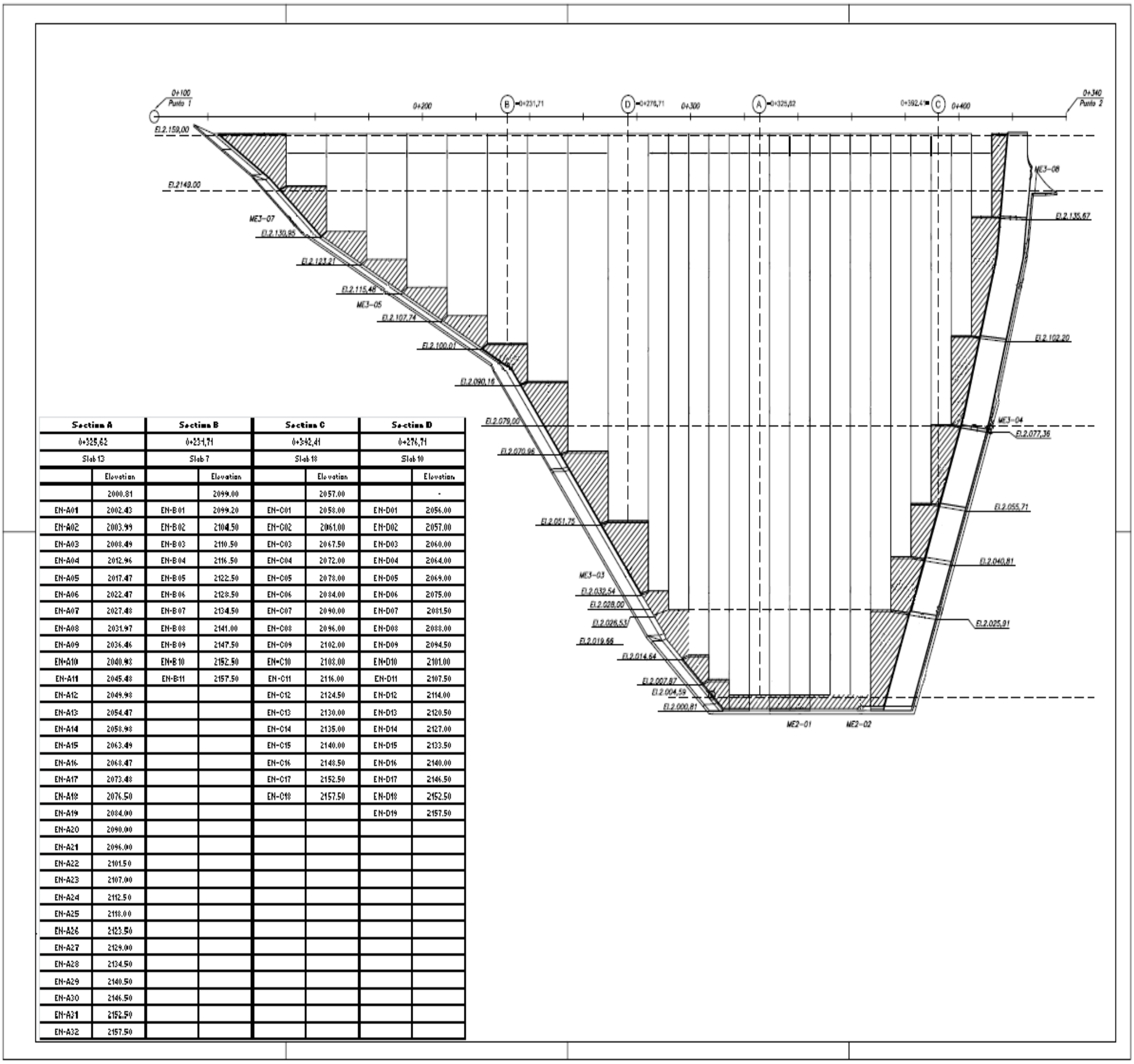


Figure 14 – Slab deflection in 1999 as measured by electrolevels.

5.3 Mazar Dam

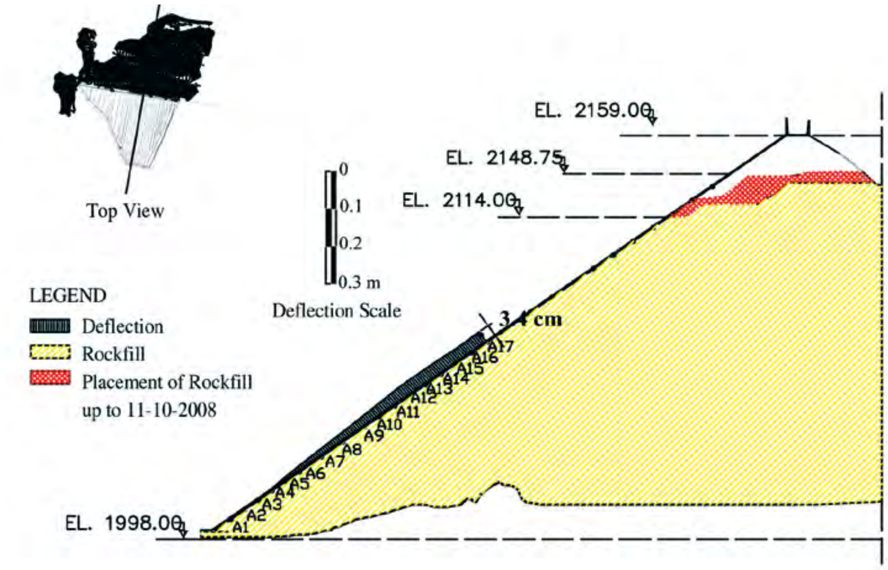
The Mazar dam was constructed in the southeast part of Equador, with maximum height of 166m, involving a 5.5 millions of cubic meters of compacted schist rocks. For the Mazar dam it was used eighty electrolevels, in four sections: section A (32 Els), section B (11 Els), section C (18 Els) and section D (19 Els), as shown in Figure 15.

Figures 16 a) to 16 g), present results of the deflections obtained for different stages of construction for Mazar Dam, along Section A. The calculated deflections due to reservoir impounding, along section A, are shown in Figure 17.

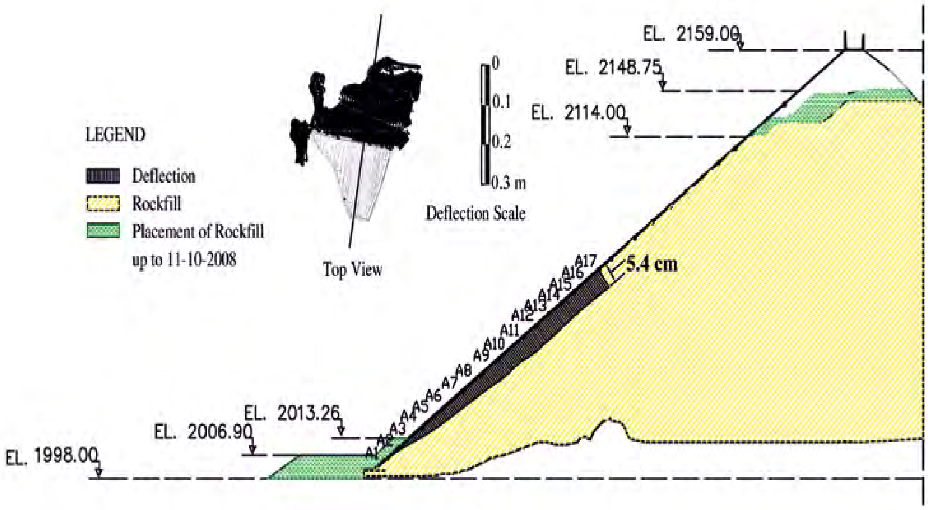


Section A		Section B		Section C		Section D	
0+325,62		0+231,71		0+342,41		0+276,71	
Slab 13		Slab 7		Slab 18		Slab 10	
	Elevation		Elevation		Elevation		Elevation
	2000.81		2099.00		2057.00		-
EN-A01	2002.43	EN-B01	2099.20	EN-C01	2058.00	EN-D01	2056.00
EN-A02	2003.99	EN-B02	2104.50	EN-C02	2061.00	EN-D02	2057.00
EN-A03	2008.49	EN-B03	2110.50	EN-C03	2067.50	EN-D03	2060.00
EN-A04	2012.96	EN-B04	2116.50	EN-C04	2072.00	EN-D04	2064.00
EN-A05	2017.47	EN-B05	2122.50	EN-C05	2078.00	EN-D05	2069.00
EN-A06	2022.47	EN-B06	2128.50	EN-C06	2084.00	EN-D06	2075.00
EN-A07	2027.48	EN-B07	2134.50	EN-C07	2090.00	EN-D07	2081.50
EN-A08	2031.97	EN-B08	2141.00	EN-C08	2096.00	EN-D08	2088.00
EN-A09	2036.46	EN-B09	2147.50	EN-C09	2102.00	EN-D09	2094.50
EN-A10	2040.98	EN-B10	2152.50	EN-C10	2108.00	EN-D10	2101.00
EN-A11	2045.48	EN-B11	2157.50	EN-C11	2116.00	EN-D11	2107.50
EN-A12	2049.98			EN-C12	2124.50	EN-D12	2114.00
EN-A13	2054.47			EN-C13	2130.00	EN-D13	2120.50
EN-A14	2058.98			EN-C14	2135.00	EN-D14	2127.00
EN-A15	2063.49			EN-C15	2140.00	EN-D15	2133.50
EN-A16	2068.47			EN-C16	2148.50	EN-D16	2140.00
EN-A17	2073.48			EN-C17	2152.50	EN-D17	2146.50
EN-A18	2076.50			EN-C18	2157.50	EN-D18	2152.50
EN-A19	2084.00					EN-D19	2157.50
EN-A20	2090.00						
EN-A21	2096.00						
EN-A22	2101.50						
EN-A23	2107.00						
EN-A24	2112.50						
EN-A25	2118.00						
EN-A26	2123.50						
EN-A27	2129.00						
EN-A28	2134.50						
EN-A29	2140.50						
EN-A30	2146.50						
EN-A31	2152.50						
EN-A32	2157.50						

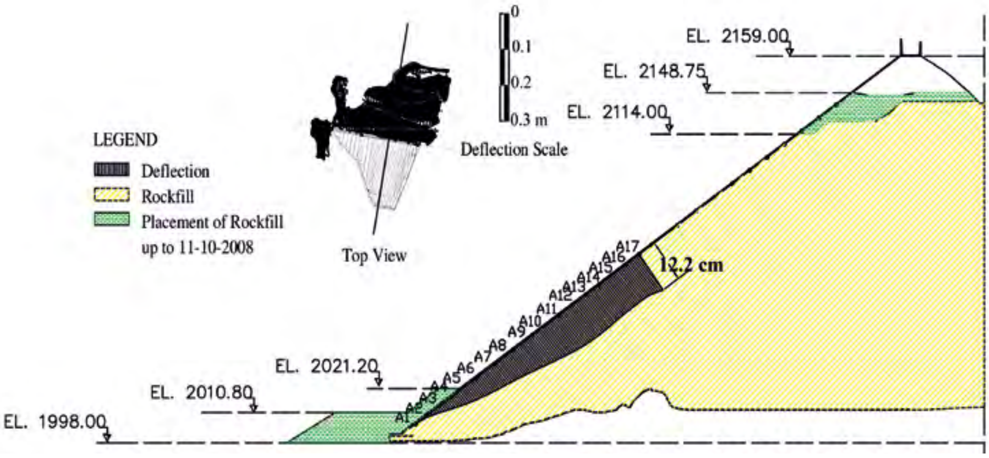
Figure 15 – Frontal view of the position of the ELs in the four sections-Mazar Dam unclear



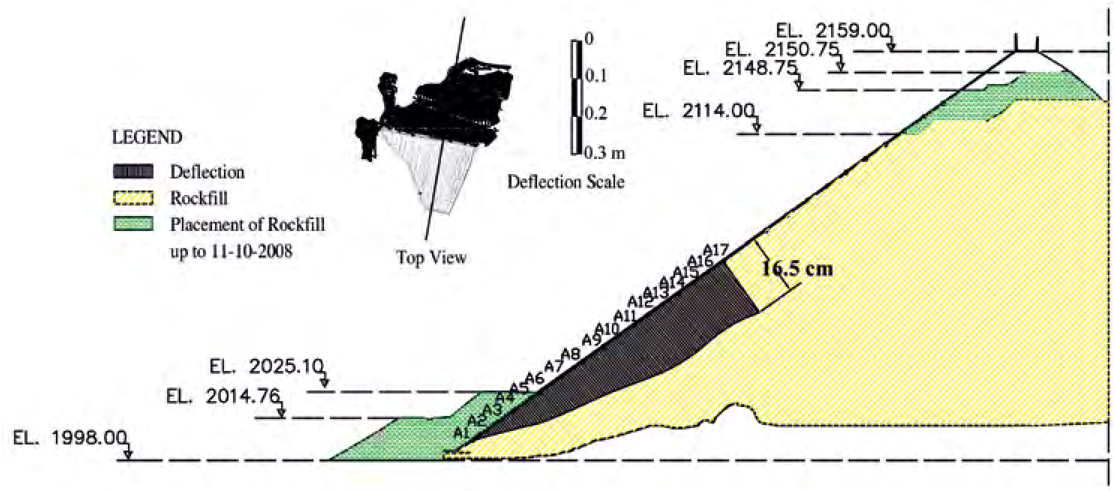
a) Deflections along the section A (11-10-2008)



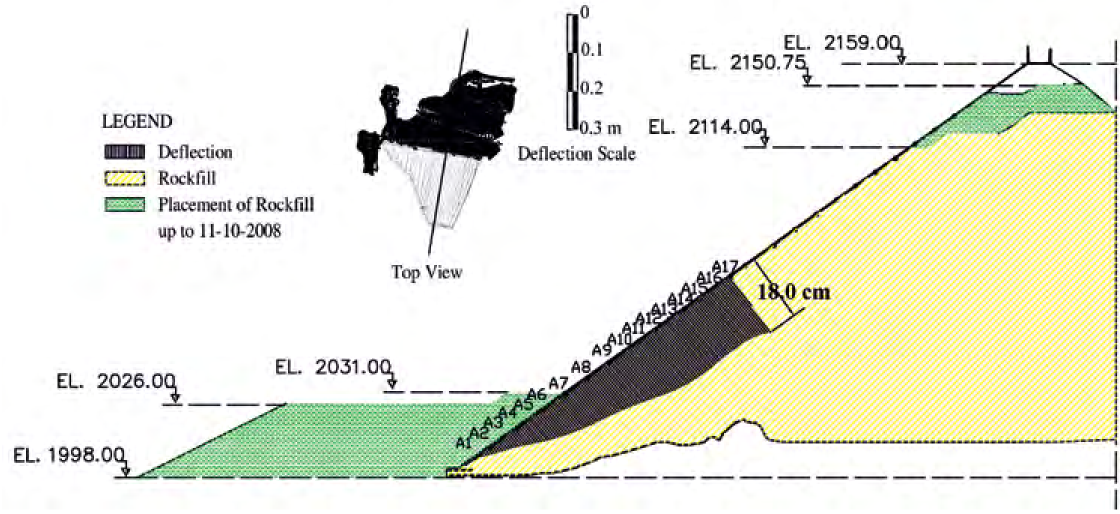
b) Deflections along the section A (31-10-2008)



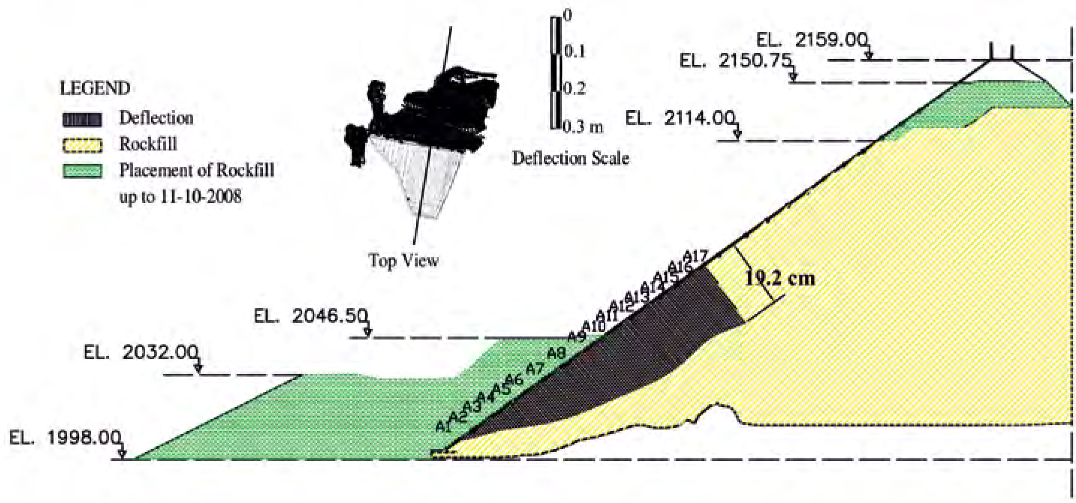
c) Deflections along the section A (29-11-2008)



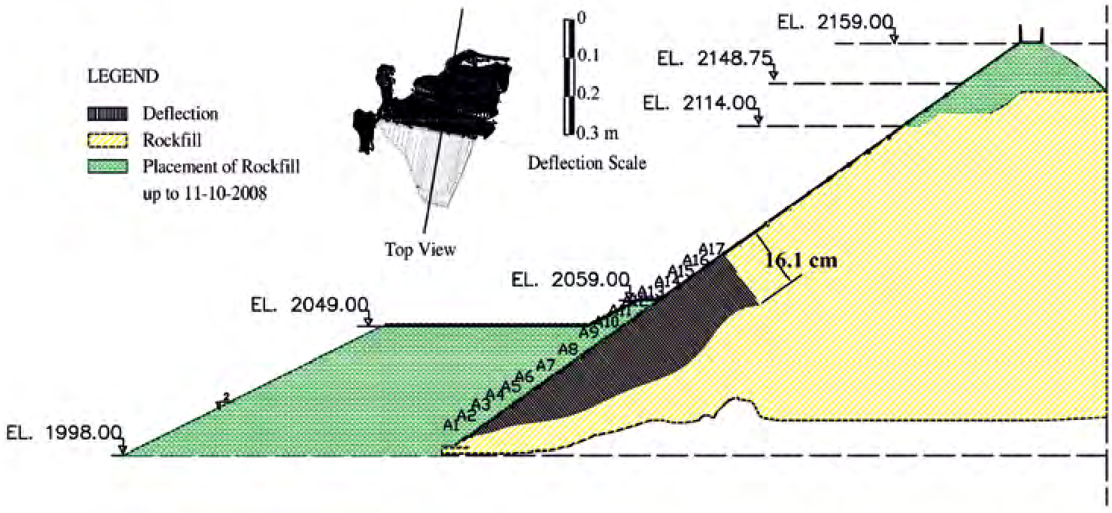
d) Deflections along the section A (30-12-2008).



e)Deflections along the section A (31-01-2009)



f) Deflections along the section A (28-02-2009)



g) Deflections along the section A (31-03-2009)

Figure 16 (a to g) – Evolution of the deflections, along section A, for different construction stages (Toledo Ramos, 2009)

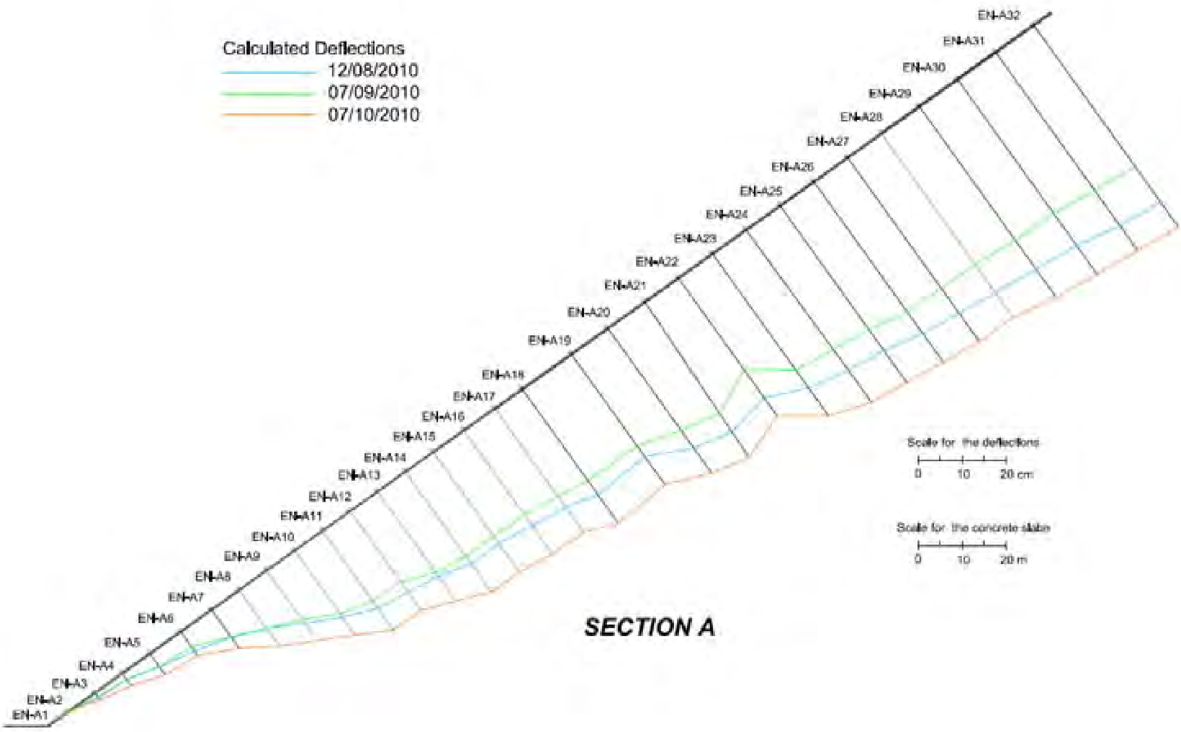


Figure 17 – Calculated deflections caused by the reservoir impounding. Mazar Dam.

6. Concluding Remarks.

In the past three decades a considerable number of very large CFRD have been constructed involving hundreds of millions of cubic meters of compacted rockfill. This has stimulated the need to understand the deformation characteristics of compacted rockfill (structural element) and to predict displacements of the concrete face (impervious element). Information about full scale behaviour can be obtained from carefully planned instrumentation. One of the most important instrumentation requirements is related to the measurements of the face deflection during construction and reservoir impounding. Various methods have been made around the world to measure the face deflection of CFRDs. Most of these measurements are based on the use of adapted inclinometers passing through access tube attached, either to the surface or to the back of the face membrane by steel brackets. These procedures, however, have associated difficulties.

In order to overcome these difficulties, the use of electrolevels to obtain the face deflection of the concrete slab has shown its potential. One of the major benefits of using electrolevels system to assess engineering performance of CFRDs is related to the simplicity of fixing the devices to the concrete slab. The viability of their incorporation into the monitoring system of CFRDs has been demonstrated in this paper.

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