THEME B

Formulator: COYNE ET BELLIER
Prediction of upstream face deflection of a CFRD during its first impounding

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Prediction of the upstream face deflection of a CFRD during its first impounding with two numerical models

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Sixth International Benchmark Workshop On Numerical Analysis Of Dams

THEME B

PREDICTION OF THE UPSTREAM FACE DEFLECTION OF A CFRD DURING ITS FIRST IMPOUNDING

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1.OBJECTIVE OF THIS BENCHMARK WORKSHOP THEME

For this new benchmark workshop, theme B is moving into the Concrete Faced Rockfill Dam (CFRD) world, focusing on prediction of movements of their upstream slabs during impounding.

Numerical models with realistic constitutive laws have been scarcely used in the past for such dams, experience and empirical design being the main way to design and to interpret their behaviour.

Participants are invited to provide the slab deflection of a typical CFRD, during its first impounding, based on settlement movements during construction of the fill. They can use any 2-D or 3-D numerical models of their choice. Enough information is also provided to determine full 3-D displacements of the slabs to allow estimation of movements along the slab joints.

Results should be provided and explained in a paper with a maximum of 15 pages, including figures, tables and a summary in English and French. The paper should describe the method, constitutive laws, assumptions used and give all pertinent information and comments on the results obtained. In addition, a set of compulsory results should be given for comparison purpose, in result sheets under Excel format.

2.GENERAL DESCRIPTION OF THE THEME

The subject of theme B has been prepared on the basis of the observed behaviours of several recent CFRD. The following description corresponds to one of these structures, which will be used to some extent as a reference for results of the analyses.

2.1 The dam

Main features:

maximum height: 125 metres

crest length: 860 meters

volume: 8 Mm^3 approximately

rockfill type: various quality and size basaltic materials obtained from quarries.

upstream slope: 1.3h : 1v

downstream slope: 1.2h : 1v (+10 m berm).

2.1.1 Dam geometry and materials

The zoning of the dam has been designed to take best advantages of the rockfill properties. Table 1 gives a description of the main zones, while location and extension are shown on Figures 1 and 2.

Table 1. Materials used for the construction.

Materials	Zone	Classification	Compaction methods
Rockfill	E0	Rockfill with a minimum of 70% of dense basalt, ∅<400 mm	Vibratory roller, 90 kN, 4 passes, compacted in 0.40 layers
	E1	Rockfill with a min. of 70% of dense basalt, ∅<800 mm	Vibratory roller, 90kN, 6 passes, compacted in 1.80 layers
	E2	Rockfill with a min. of 70% of dense basalt, ∅<1600 mm	Vibratory roller, 90 kN, 4 passes, compacted in 1.60 m layers
	E3	Rockfill composed of breccias, dense and vesicular basalt in any proportion, ∅<1600 mm	Vibratory roller, 90 kN, 4 passes, compacted in 1.60 m layers
	E1A/ E2A	Rockfill with a min. of 70% of dense basalt, ∅<1600 mm	Dumped in the river bed up to water level (maximum water depth of 10 m in the upstream zone)
Transition	T1	Transition of dense basalt, ∅<100 mm	Vibratory roller, 90 kN, 4 passes, compacted in 0.40 m layers and moisted at the rate of 100 l water/sec U/S face : vibratory roller, 60 kN, 10 passes moving upwards
Soil	SL	dumped earthfill	

For the whole set of materials, an equivalent volumetric weight of γ =22.15 kN/m^3 could be considered.

2.1.2 Foundations

The dam was mostly build on basaltic bedrock. Fully exposed rock was located along 25 meters downstream of the plinth. However, some weathered rock was left in place further downstream in order to limit the excavation volume.

<u>For the study, the foundation must be presumed impervious and rigid.</u>

2.1.3 Concrete face

It was built in one step up to the top of the dam, in adjacent vertical slabs. The slab thickness follows the established rule: e = 0.30 + 0.002 H, with H=371-Elev. Thus, the face varies from 0.30 to 0.54 meters thick.

There are only vertical joints between adjacent slabs, every 16 metres in the bank to bank direction.

The concrete strength was specified as fck = 21 MPa at 90 days. The reinforcing is distributed along the neutral line of the slab section following the criterion St = 0.004 Sc for the vertical direction and St = 0.003 to 0.004 Sc for the horizontal direction (St being the steel section and Sc being the concrete section)

2.1.4 Construction schedule

The construction started on March 1st, 1996.

Because of safety considerations, the priority section, i.e. the maximum possible volume of rockfill before the river diversion, represented almost one third of the total volume.

Most of the rockfill construction had been completed in November 1998.

The impoundment of the reservoir started from elevation 296 in mid December 1999 to elev. 356 in mid March 2000 (≈0.7 m/d) and elev. 356 in mid March 2000 to elev. 366 by the end of April 2000 (≈0.2 m/d).

The file BW6B_mon.xls/reservoir contains the reservoir level data since the beginning of the impoundment.

Figure 3 shows an upstream view of the dam construction stages.

2.2 Dam monitoring

The dam was particularly well instrumented, numerous and different type of equipment gave data during the construction and the reservoir impounding steps.

<u>Upstream concrete face</u>

- Slab deflection

Two slabs have been equipped with respectively 10 and 15 deflection tracers to measure displacements perpendicular to the concrete slab.

- Joint opening

Some joint meters were installed at plinth-slab joints, slab-slab joints and slab-parapet joints.

<u>Dam body</u>

- Settlements

Settlements are measured in two dam sections, corresponding to the upstream face slabs n° 29 and 45. Instruments are identified as CT for 'Cellule de Tassement'. There are 16 instruments in section 29 (CT-01 to CT-16) at four different levels and 10 instruments in section 45 (CT-17 to CT-26) at three different levels.

At each level in each different section the first instrument is located close to the upstream face allowing thus comparison with data from previous instruments.

- Horizontal displacements

Same locations than the CTs. Instruments are identified as CL-h for 'Cabine de Lecture-horizontal' and range from CL1 h1 to CL7 h7, according to sections and levels.

- Piezometers

They are installed between elevations 295 and 365.

Downstream face

Geodetic survey gauges have been installed on a grid defined by 5 elevations and 7 vertical sections.

Figure 4 shows monitoring instruments implemented on cross section B.

2.2.1 Description of the dam construction and impounding file (monitoring.xls)

This file describes the monitoring of particular instruments, during construction. This is the only data available for the participants.

The next table gives information on instruments used for the monitoring.

Table 2. Instrument descriptions.

Measurement cabin	Instruments	Initial level (m)	Section	Maximum Rockfill height	Foundation level
CL 1	survey station	354.289	B		
	CT 15 / h1	352.991	B	358.54	251.50
	CT 16 / h2	352.994	B	375.50	251.50
CL 2	survey station	327.460	B		
	CT 12 / h1	327.488	B	333.38	252
	CT 13 / h2	327.060	B	358.85	252
	CT 14 / h3	328.309	B	375.50	252
CL 3	survey station	302.275	B		
	CT 10 / h4	302.095	B	375.50	251
	CT 08 / h2	301.699	B	333.846	252.5
	CT 07 / h1	301.331	B	307.308	252
CL 4	survey station	275.818	B		
	CT 01/ h1	273.680	B	279.615	254
	CT 02 / h2	273.495	B	306.808	254
	CT 03 / h3	273.446	B	333.308	254
	CT 05 / h5	273.961	B	375.5	252
	CT 06 / h6	273.785	B	352.731	252

All the displacement values listed in the file are expressed in absolute value, i.e. horizontal displacement or settlement from the initial level of the instrument.

These measurements in cross section B are considered enough representative of the whole behaviour of the dam.

2.3 Material properties

Materials of the various zones are described in figures 1 and 2. For modelling purpose, simplification of the dam internal zoning should at least differentiate rigid transition and rockfill close to the upstream face and a less resistant rockfill body downstream.

Based on dam monitoring, participants are free to select essential features representing the material behaviours. The dam is supposed to be free of seepage and wet densities don't have to be considered.

Any kind of model can be used. For those requiring internal shear resistance, it is suggested to use values from Table 3 below.

Table 3. Resistance parameters.

	Resistance			
	ϕ (degrees)		c (kPa)	
	(1)	(2)	(1)	(2)
Transition	43	41	0	0
Rockfill	45	41	0	0

(1) Construction phase
(2) Reservoir impoundment phase

The Young deformation modulus of the concrete face shall be considered as E = 30000 MPa.

2.4 Numerical analysis

Answering this proposal can be done using different numerical means. However, two and three dimension analysis are required. The next sections give information on the modelling and finite element meshes, useful for participants who chose this method.

2.4.1 Two dimensional analysis

2.4.1.1 Finite element mesh

A basic mesh is provided to the participants. They are free to use it or to change it.

As an example, the following figure shows one of the meshes used for the major cross section B.

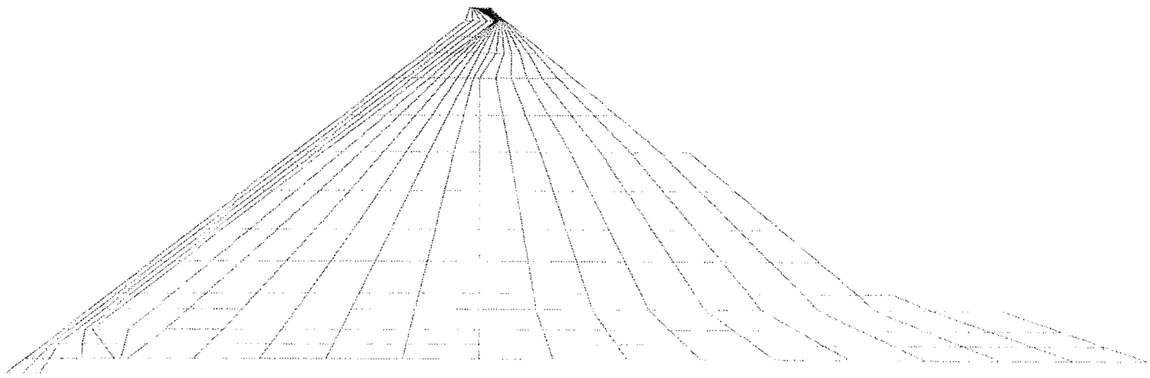


Figure 6. Finite element mesh at section B created with Mailef v 4.05.

The file BW6B_MSH.txt gives cross section B geometry and all the points used in the mesh.

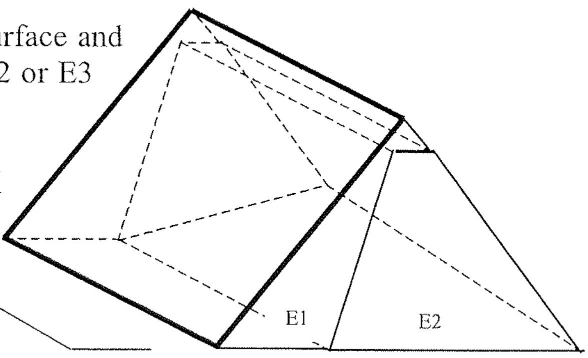
2.4.2 Three dimensional analysis

2.4.2.1 Finite element mesh

Unlike two dimensional analysis, no mesh is provided to the participants. However, the model can easily be built with the different views included in the paper (upstream , downstream views and cross-sections) and with the following assumption:

- Between two successive cross sections, the first upstream part of the foundation must be presumed plane and horizontal.
- The link between the horizontal upstream surface and the downstream surface is made at the E1/E2 or E3 material transition.

Therefore, using only the upstream and downstream views of the dam is enough to determine all the foundation geometry of the model.



2.4.3 Concrete slab

Participants are let free to represent or not the upstream concrete slab with any appropriate method.

3. STATEMENT OF THE PROBLEM

Participants have to give their predictions of the upstream face deflection of the dam. They are welcome to give information on the following points.

3.1 Construction phase

To understand future discrepancies between participant results, initial parameters have a particular signification. Should be indicated:

- the internal zoning adopted in the dam model, described in the principal cross section,
- the different material model parameters, described in a table.

Please put tables with comments in the file BW6B_RES.xls/construcion.

3.2 Reservoir impoundment phase

3.2.1 Two dimensional analysis

- Table describing settlements and horizontal displacements at (CR-01, CR-05, CL 4) ; (CR-07, CR-10, CL 3) ; (CR-12, CR-14, CL 2) ; (CR-15, CR-16, CL 1) and at the crest,
- Normal and tangential displacement values of upstream slab located at CR-01,CR-07, CR-12, CR-15,
- Normal force, shear force and moment (N,T,M) values applied on the concrete face located at CR-01,CR-07, CR-12, CR-15,

Please fill the answer sheet in the file results.xls/2D.
Besides information asked in the answer sheet, it could be interesting to produce displacement profiles at levels 272 m, 299.50 m, 326 m, 351.50 m.

3.2.2 Three dimensional analysis

The study is concentrated on the upstream face deflection and tangential displacements.

For different levels at several cross sections, should be given:

- Normal and tangential displacement values located at levels 299.50 m, 326.00 m, 351.50 m and at the crest, for cross sections located at PM 200, 350, 484, 624 and 750,
- M,N,T values applied on the concrete face located at the previous points.

Please fill the answer sheet in the file BW6B_RES.xls/3D.
Again, it could be also interesting to give displacement field plotted along the slab and normal to the slab.

4. CONTENT OF THEME B PACKAGE

• BW6B_INF.doc	proposal of theme B
• BW6B_MON.xls	monitoring data
• BW6B_MSH.txt	definition of cross section B mesh
• BW6B_RES.xls	templates for results (construction assumptions, 2-D and 3-D analyses)
• BM6-layout.pdf	suggested layout for contribution
• BW6B_a.dwg to BW6B_d.dwg	AutoCAD files representing figures of the paper

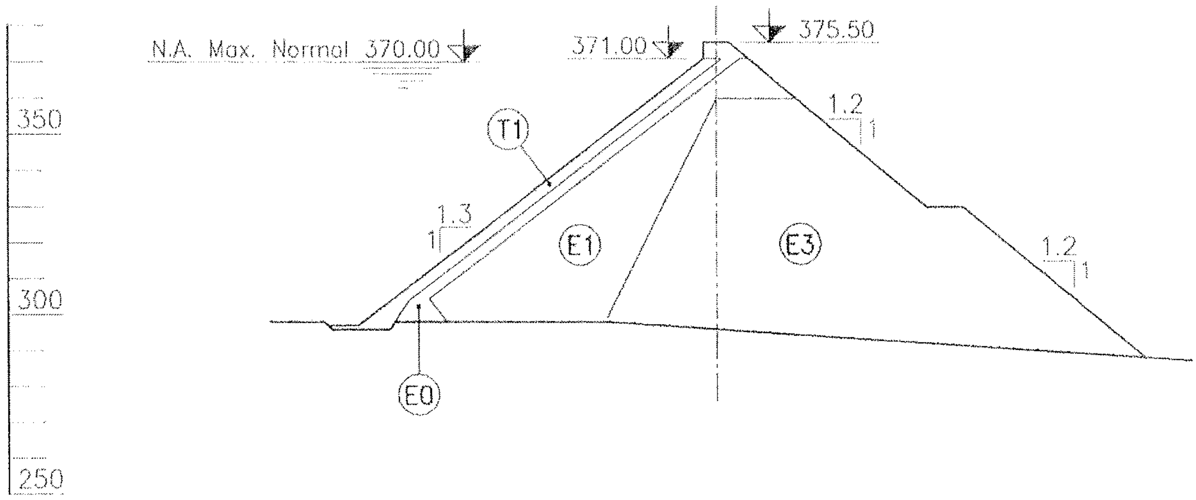
5. RESULTS TO BE PROVIDED

Participants should send their results in two files:

- BW6B_RESxxx.xls containing compulsory results,
- BW6B_PAPxxx.doc containing their paper,

where xxx is a shortcut for their acronyms.

CROSS SECTION A _ A PM 200



CROSS SECTION C _ C PM630

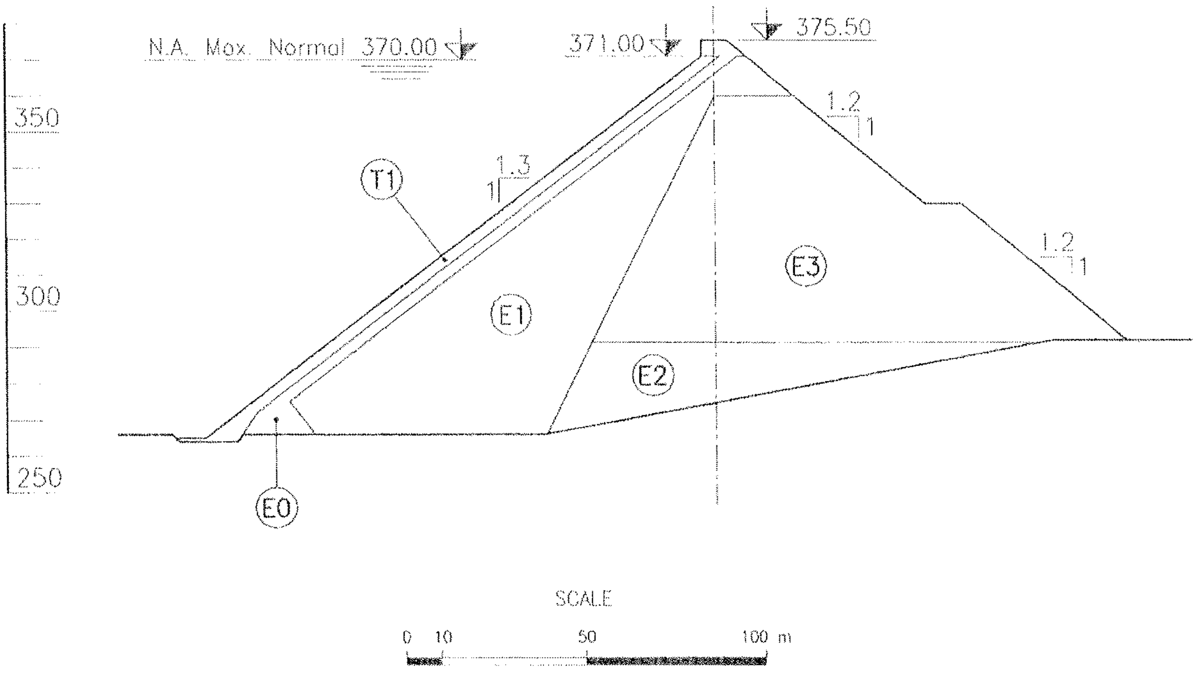


Figure 1.Cross sections A and C.

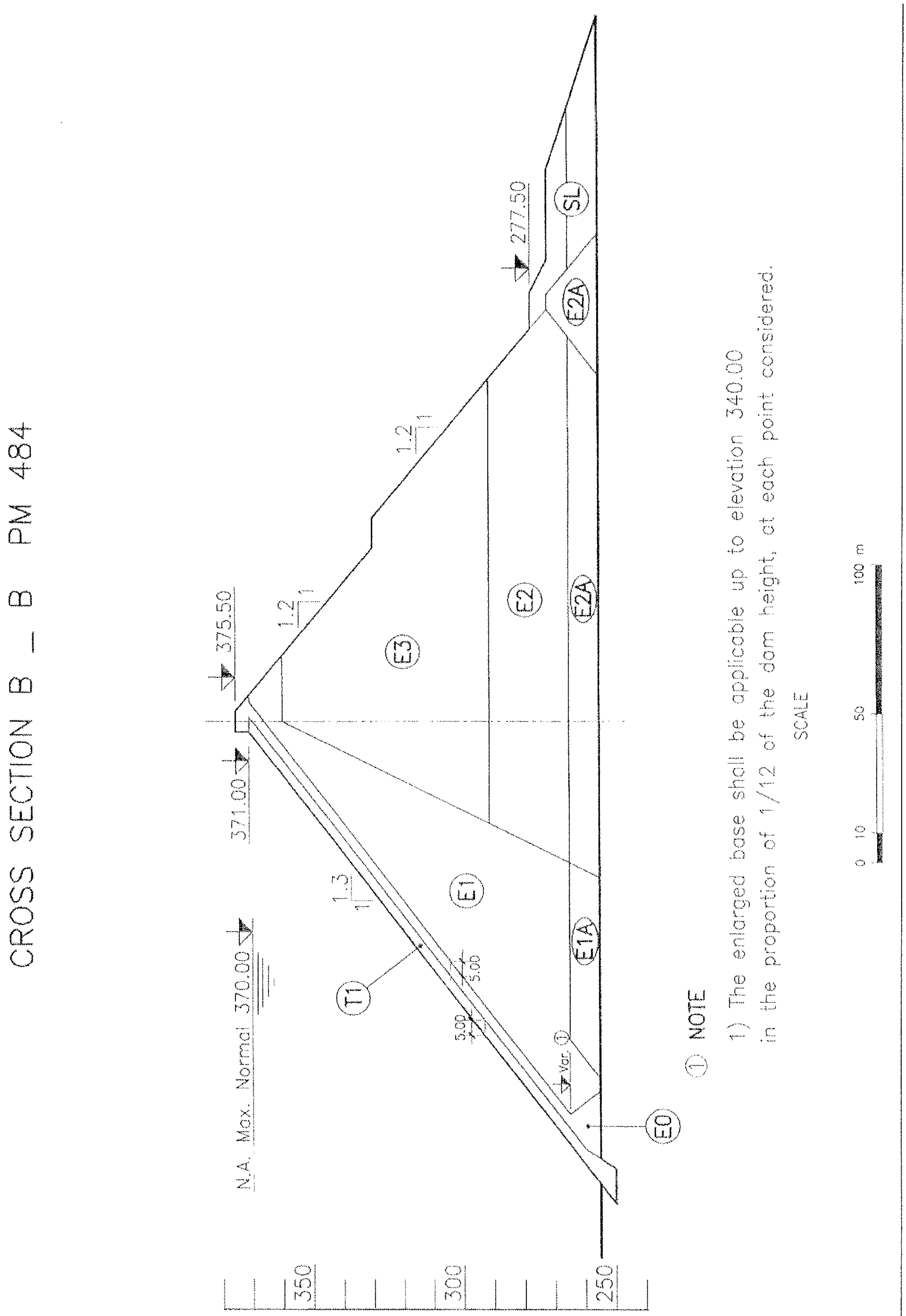


Figure 2. Cross section B.

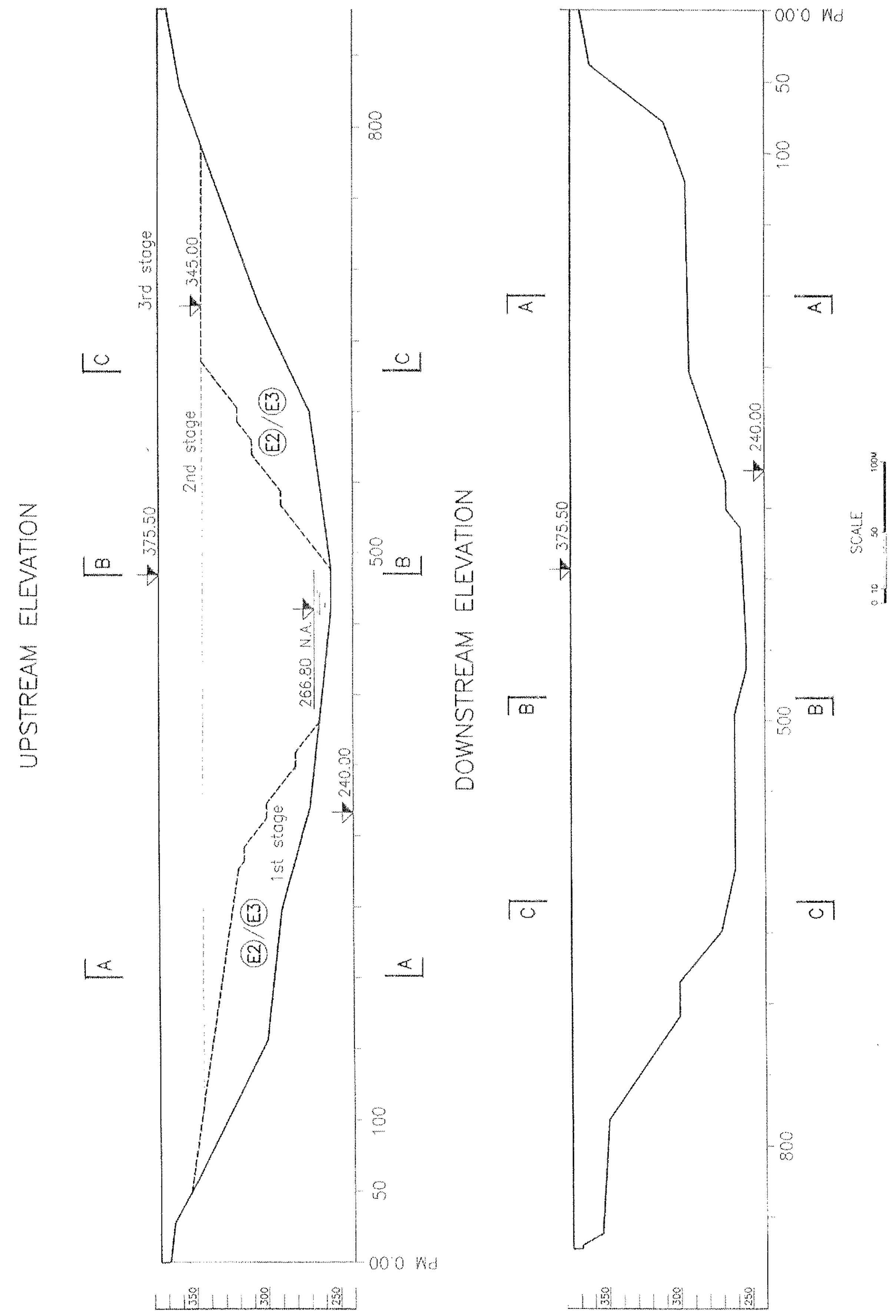


Figure 3. Upstream and downstream views of the dam.

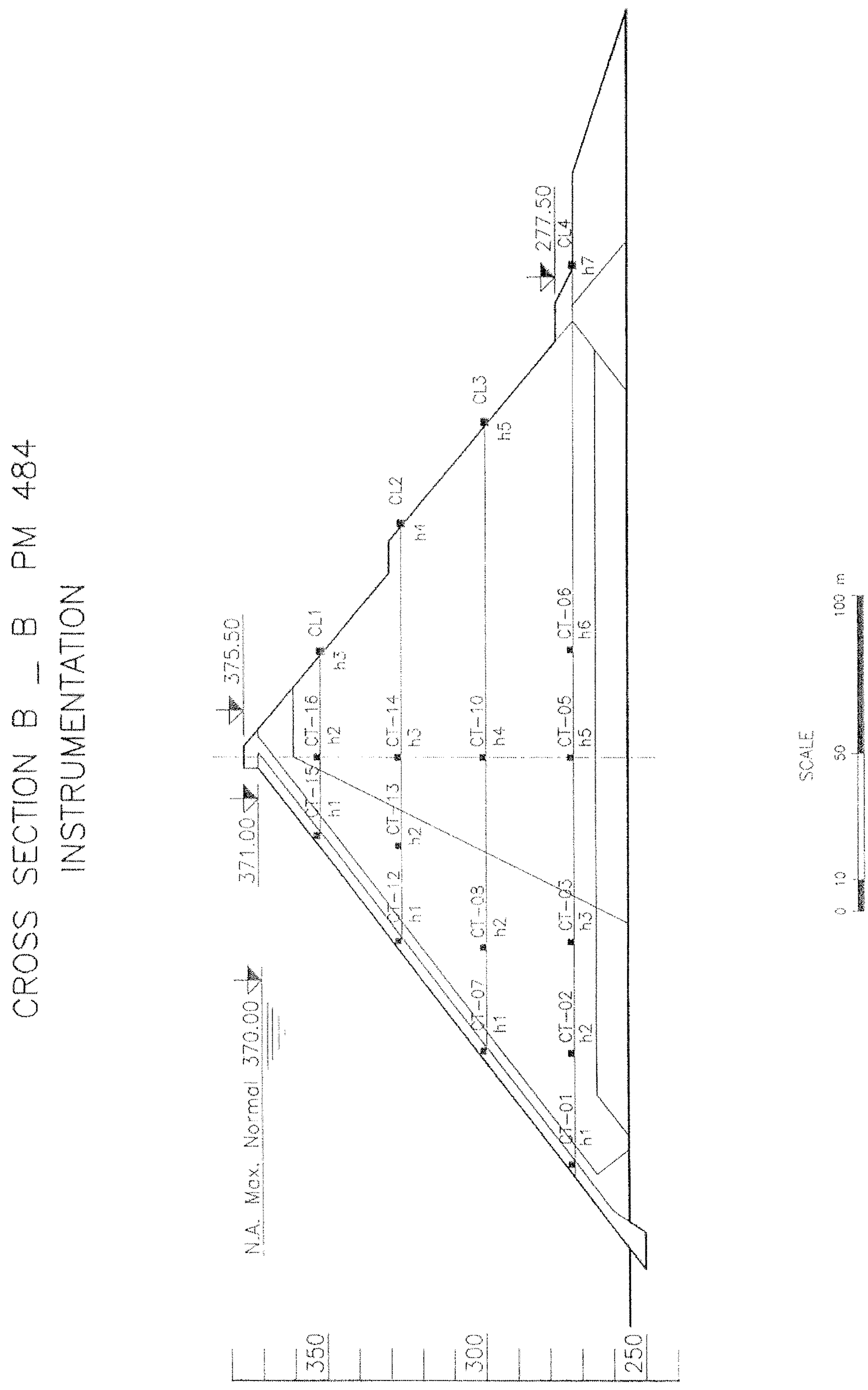


Figure 4. Cross section B, Instrumentation.

References

Coyne et Bellier, *Analysis of Dam Seepage during Impounding*, report n° ITA: ALA-00.110, July 2000

ICOLD, J. Barry Cooke Volume, *Concrete Face Rockfill Dams*, Beijing 2000

ICOLD, Proceedings, *International Symposium on Concrete Faced Rockfill Dams*, 18 September 2000, Beijing

Touileb, Bachir N., *Theme B2 Embankment dams. First Fill of a Rockfill Dam: a Case Study, LG-2 Rockfill Dam*, Fifth International Benchmark on Numerical Analysis of Dams, Denver CO, June 2-5 1999

Y. Uchita, Y. Hoshino, H. Yamaguchi, T. Ishiguro (-will not be presented by the authors)
Prediction of the upstream face deflection of a CFRD during its first impounding with two numerical models

PREDICTION OF THE UPSTREAM FACE DEFLECTION OF A CFRD DURING ITS FIRST IMPOUNDING WITH TWO NUMERICAL MODELS

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ABSTRACT: The authors have conducted analytical studies to estimate the behavior of the face slab of a CFRD during its first impounding. Two methods are applied in this study; Method-A (using elasto-plastic constitutive equations) and Method-B (using elasto/visco-plastic constitutive equations). Both the maximum settlement and maximum horizontal displacement of the face slab are detected at about half the dam height in both methods. The maximum displacement normal to the face slab is about 1.2m in Method-A and is about 1.0m in Method-B. While the distribution of the displacements exhibits an almost smooth shape pattern from the plinth to the crest in Method-A, an inflection point appears at a distance about 20m from the plinth in Method-B. This point is equivalent to the boundary between layers of rock zone.

RÉSUMÉ: Les auteurs ont effectué des études analytiques pour évaluer le comportement de la dalle de face d'un CFRD durant son premier endiguement. Deux méthodes furent appliquées pour cette étude, à savoir Méthode A (utilisant des équations constitutives élasto-plastiques) et Méthode B (utilisant des équations constitutives élasto/viscoplastiques). Le tassement maximum aussi bien que le déplacement horizontal de la dalle de face furent détectés à environ la moitié de la hauteur du barrage dans les deux méthodes. Le déplacement maximum normal au revêtement était d'environ 1,2 m dans la Méthode A et d'environ 1,0 m dans la Méthode B. Tandis que la distribution des déplacements montre un modèle de forme presque souple à partir de la plinthe à la crête dans la Méthode A, un point d'inflexion a apparu à une distance d'environ 20 m à partir de la plinthe dans la Méthode B. Ce point est équivalent à la limite entre les couches de la zone rocheuse.

Key Words: Elasto-plastic, Duncan-Chang, Elasto/visco-plastic, Sekiguchi and Ohta Rockfill dam, Settlement, Pounding, and Face slab

1. BASIC ANALYTICAL CONDITIONS

1.1 Analytical Model

1.1.1 Analytical range

A two-dimensional mesh shown in the "BW6B_MSH.txt" file is used for analyzing the cross section during this study. A beam element is added to the upstream side of the above-mentioned mesh to model a face slab. The mesh of Method-A and Method-B are common.

1.1.2 Zoning of the analytical model

Method-A : As shown in the "Construction" sheet of the "BW6_RESTEPCO.xls" file, the internal zoning of the dam is divided into 8 segments.
Method-B: As shown in the above-mentioned file, the internal zoning of the dam is divided into 10 segments.

1.2 Boundary Conditions

The displacements occurring in vertical and horizontal directions at the foundation of the dambody are fixed. The other boundaries are possible to move.

1.3 Processes

The dam construction schedule is as shown in Fig. 1 and Table 1, both based on the rockfilling altitudes showed in the "BW6_MON.xls" file. In this file, instrument CT-14 indicates a rockfilling altitude of E.L. 327 m on April 18, 1998, on the other hand instruments CT-5 and CT-10 installed under the CT-14 indicate a rockfilling altitude of E.L. 341 m on the same rockfilling date. On this account, the rockfilling process was modified so that the rockfilling altitudes in the CT-14 related zones during the period from April 18 to May 5, 1998 matched the rockfilling altitudes at CT-5 and CT-10. For the processes of the first impounding, the data shown in the "BW6_MON.xls" file are directly used for this study.

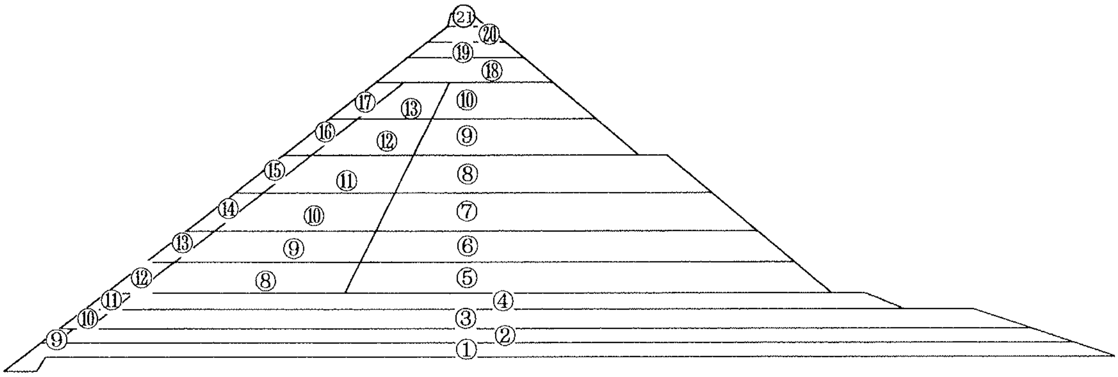

Fig.1 Dam construction schedule

Table.1 Dam construction schedule

	Schedule		Schedule
①	1997/10/14～1997/10/22	⑫	1998/4/16～1998/4/28
②	1997/10/23～1997/10/29	⑬	1998/4-29～1998/5/26
③	1997/10/30～1997/11/5	⑭	1998/5/27～1998/6/19
④	1997/11/6～1997/11/13	⑮	1998/6/20～1998/7/16
⑤	1997/11/14～1997/11/28	⑯	1998/7/17～1998/7/30
⑥	1997/11/29～1997/12/10	⑰	1998/7/31～1998/9/16
⑦	1997/12/11～1998/1/11	⑱	1998/9/17～1998/10/17
⑧	1998/1/12～1998/1/29	⑲	1998/10/18～1998/11/2
⑨	1998/1/30～1998/2/21	⑳	1998/11/3～1998/11/18
⑩	1998/2/22～1998/3/26	㉑	1999/10/26～1999/11/28
⑪	1998/3/27～1998/4/15		

2. PHYSICAL PROPERTIES DATA

2.1 Dambody

2.1.1 Method-A

1) The physical properties are as shown in the "Construction" sheet of the "BW6_RESTEPCO.xls" file. Establishment of these physical properties is based on the followings.
2) It has been known that the relationship between the stresses and strains of the rockfill material shows an elasto-plastic behavior dependent on a constraint pressure

(Matsumoto, Toyoda, and Yasuda, 1988(1), and Nakamura Toyoda, 1996(2)). Elasto-plastic constitutive equations are usually used for the deformation analyses that have been performed on a number of dams in Japan, and among these constitutive equations, Duncan-Chang's constitutive equations have been most commonly used. The authors used Duncan-Chang's constitutive equations in this analysis. Since the assigned physical properties have not provided the useful information on the non-linear physical properties or on the stress-strain relationship required for the establishment of the physical properties, the authors assumed physical properties using the procedure shown below and conducted analyses.

3) Elastic modulus of each segments of rock zone was followed by Duncan-Change's formula:

$$E_t = \left(1 - \frac{R_f * (1 - \sin\phi) * (\sigma_1 - \sigma_3)}{2 * C * \cos\phi + 2 * \sigma_3 * \sin\phi}\right)^2 * K * P_a * \left(\frac{\sigma_3}{P_a}\right)^n \quad \text{-(1)}$$

c : Cohesion

ϕ : Angle of internal friction

P_a : Atmospheric pressure

K, n, R_f : Experimental constants

4) The analyses during the rockfilling process have been performed on several cases by using "K" value of each zone as a parameter, and then a "K" value of each zone that can well account for measurement during the rockfilling process is determined. And this value is used to perform analyses during the first impounding. The parameters were decided by following conditions:

- As the initial value for the parameter analysis, analytical cases at existing sites in Japan have been referred to and their average values are taken.
- For the E1, E2, and E3 zones, the "K" values well accounting for the measurements of the instruments CT-2, 3, 5, 6, 8, 10,16 during the construction are finally adopted.
- In the E1A and E2A zones, rocks are dumped in the river bed. On this account, it is estimated that the compaction effect obtained for these zones was not as significant as that for the E1, E2, and E3 zones. Accordingly, the smaller "K" values are adopted as these zones than the E1, E2, and E3 zones.
- For the T1 and E0 zones, the “BW6B_INF.doc” file dictates that their materials are more rigid than those of other zones and that the T1 and E0 zones be sufficiently compacted for a small winding thickness. Accordingly, the lower limit values of "K" are set above the value adopted for the E1, E2, and E3 zones. Also, the upper limit values

are determined to allow for the reproducibility of measurement during the rockfilling process.
5) For the coefficient "n" and "Rf", analytical cases at existing sites in Japan have been referred to and their average values are adopted.
6) For the Poisson's ratio, although non-linear representation can be used, since this ratio does not affect analytical results as significantly as an elastic modulus, a general value of 0.33 is adopted for all the rockfill material.
7) The assigned cohesion and the assigned angle of internal friction in the "BW6B_INF.doc" file are adopted for shear strength characteristics.
8) For the SL zone, elastic condition is adopted as physical property, because this zone is located at the most downstream of the dambody so that the deformation behavior of the face slab is unlikely to be affected significantly.

2.1.2 Method-B

1) The physical properties conditions are as shown in the "Construction" sheet of the "BW6_RESTEPCO.xls" file. Establishment of these physical properties is based on the following:
2) The elasto/visco-plastic constitutive equations proposed by Sekiguchi and Ohta are adopted in Method-B. It has been indicated that under high-overburden pressure conditions, in-situ compacted rockfill material, as with that of a large dam, takes an elasto/visco-plastic behavior similar to that of clayey soil and that this behavior can be approximately represented by applying prior elasto/visco-plastic constitutive equations (Duncan ewt.al, 1969(3), Miura and Yasufuku, 1983(4)). Also, a method of setting parameters about the above-mentioned analysis has recently been proposed by Ohta et.al. And the applicability of that method to AFRD has also been verified (Ohta et.al, 2001(5), and by Mori, Uchita, Nakano, Ishiguro, and Ohta, 2001(6)).
3) The parameters required for the analysis relate to one-dimensional compression characteristics and shear strength characteristics. The assigned angle of internal friction is used intactly for the latter. The former characteristics not assigned as the given-condition, are estimated using the prior data shown in References (3) to (6) and the grain size correction method shown in Reference (5), since there were no laboratory one-dimensional compression test results based on dam embankment materials. In Fig. 2 based on these References, the compression index λ, and swelling index κ, of the dam embankment material are plotted with respect to its maximum grain size. Figure 2 indicates that since increases in the maximum grain size of the dam embankment material reduce the number of intergranular contact points in the rockfill material, the loads intergranularly transmitted will increase and thus the amount of compressive deformation will increase. Since the rockfill materials that are used during the analysis are specified to be a relatively rigid material in the "BW6B_INF.doc" file, the lower

limit values of prior data, in principle, are adopted as the compression index. And the appropriate compression indexes are determined according to the maximum grain size in each zone. For the E1, E11, E1A, and E2A zones, since the measurements during construction have been obtained, the respective compression indexes are modified so that their matching to those measurements are ensured in the range of the prior data. The swelling index is calculated by using the irreversibility ratio (Λ=1-κ/λ) derived from the λ and κ values of the Yashio rockfill dam in Fig. 2, and the above-mentioned λ value derived from the maximum grain size.

4) For the equivalent pre-consolidation pressure values (kinks in the consolidation curves) of E1, E11, E12, E2, and E3 zones, the vibromotive forces of the vibration rollers are shown in the "BW6_MON.xls." file. Also the authors have known the equivalent pre-consolidation pressure values obtained by compaction with a vibromotive force of 270 kN from References (5) and (6) above. Accordingly, the authors determined the equivalent pre-consolidation pressure values for the current model by using the ratio that is calculated from both vibromotive forces. The equivalent pre-consolidation pressure on the E1A and E2A zones which were dumped in the river bed are set to small values because the effectiveness of compaction is unlikely to be too significant. Also, since the T0 and E0 zones are both made of a carefully compressed material, large equivalent pre-consolidation pressure values are assigned in order to take a non-linear elastic behavior for these zones.

5) The physical properties conditions of SL zone is as same as Method-A'one.

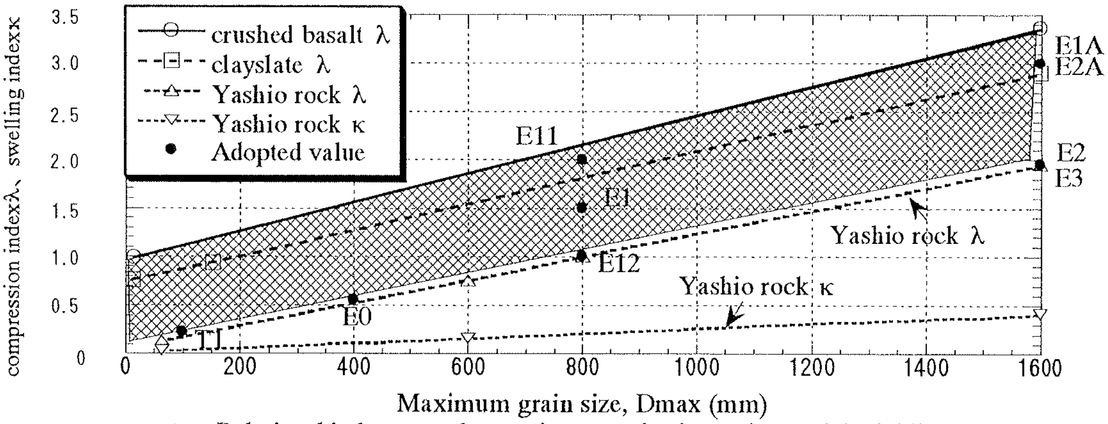

Fig.2 Relationship between the maximum grain size and materials rigidity

2.2 *Face Slab*

- Elastic condition is adopted for the face slab since it is made of a highly rigid material (30000 MPa).
- The thickness of the face slab has been set using the following expressions (pre-conditions):

$$e = 0.30 + 0.002H \quad \text{-(2)}$$

$$H = 371 - Elev \quad \text{-(3)}$$

e: thickness of the slab (m).

3. ANALYTICAL RESULTS

3.1 Settlements

- Comparisons between data that has been measured using the displacement gauges installed on and inside the dambody, and analytical results on the data, are shown in Figs. 3 and 4. In Figs. 3 and 4, measurements up to completion of the construction are plotted, and analytical results up to completion of the first impounding are plotted.
- As shown in Figs. 3 and 4, analytical results on the settlements of the dambody are likely to simulate measurements almost accurately in the Method-A and Method-B, except for CT-14.
- The settlements during the first impounding exhibit a tendency of increasing at sections closer to the upstream end of the dambody, and the settlements at sections closer to the downstream end are not so much as the upstream end are. And this tendency applies to any altitude. The settlements at the upstream surface of the dambody during the first impounding are estimated to be up to about 1.0 m. The difference of settlements in both methods is small.

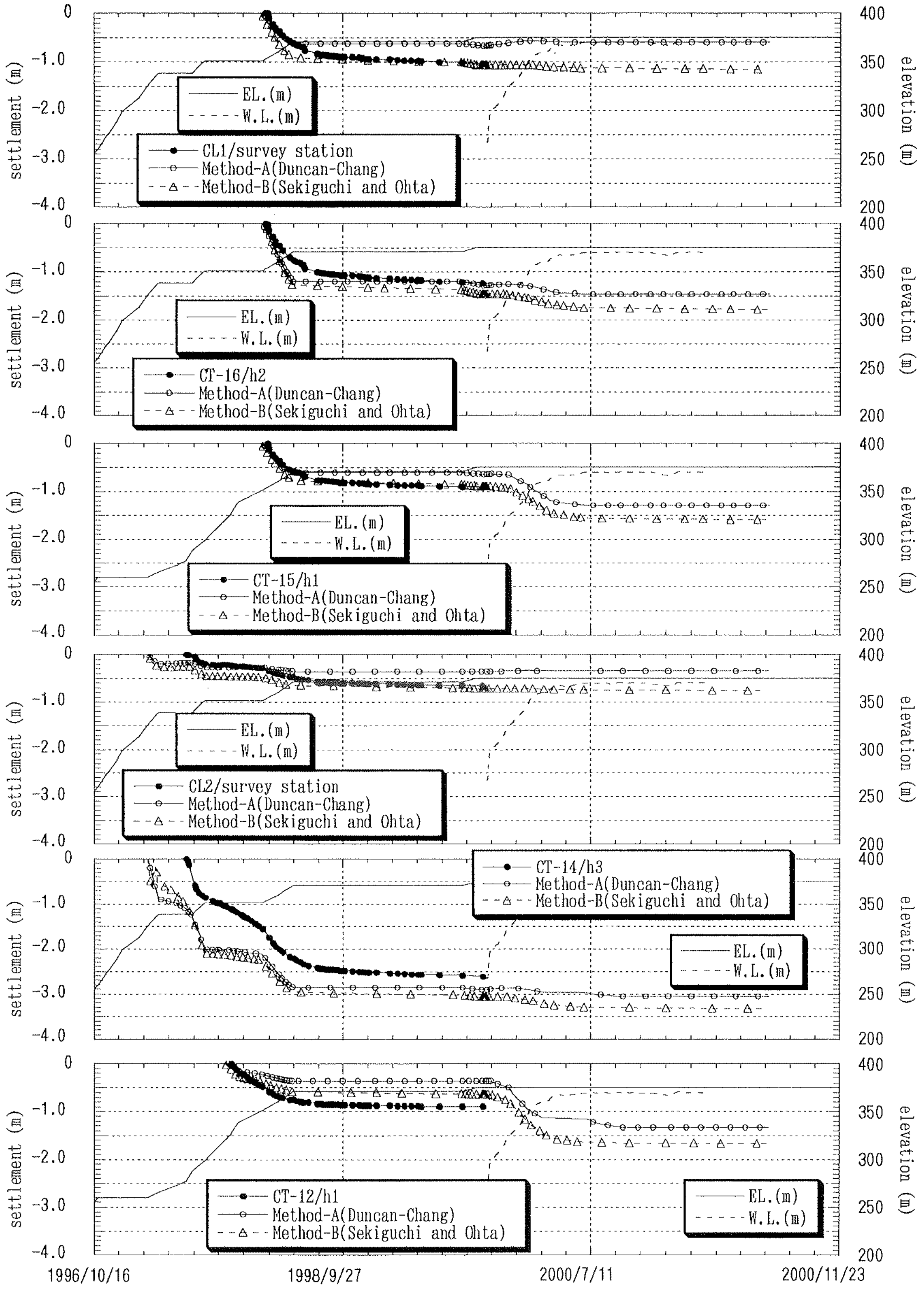

Fig.3 Comparison of measured and calculated settlements of dambody during construction and the first impounding (CL1,CT-16,CT-15,CL2,CT-14,CT-12)

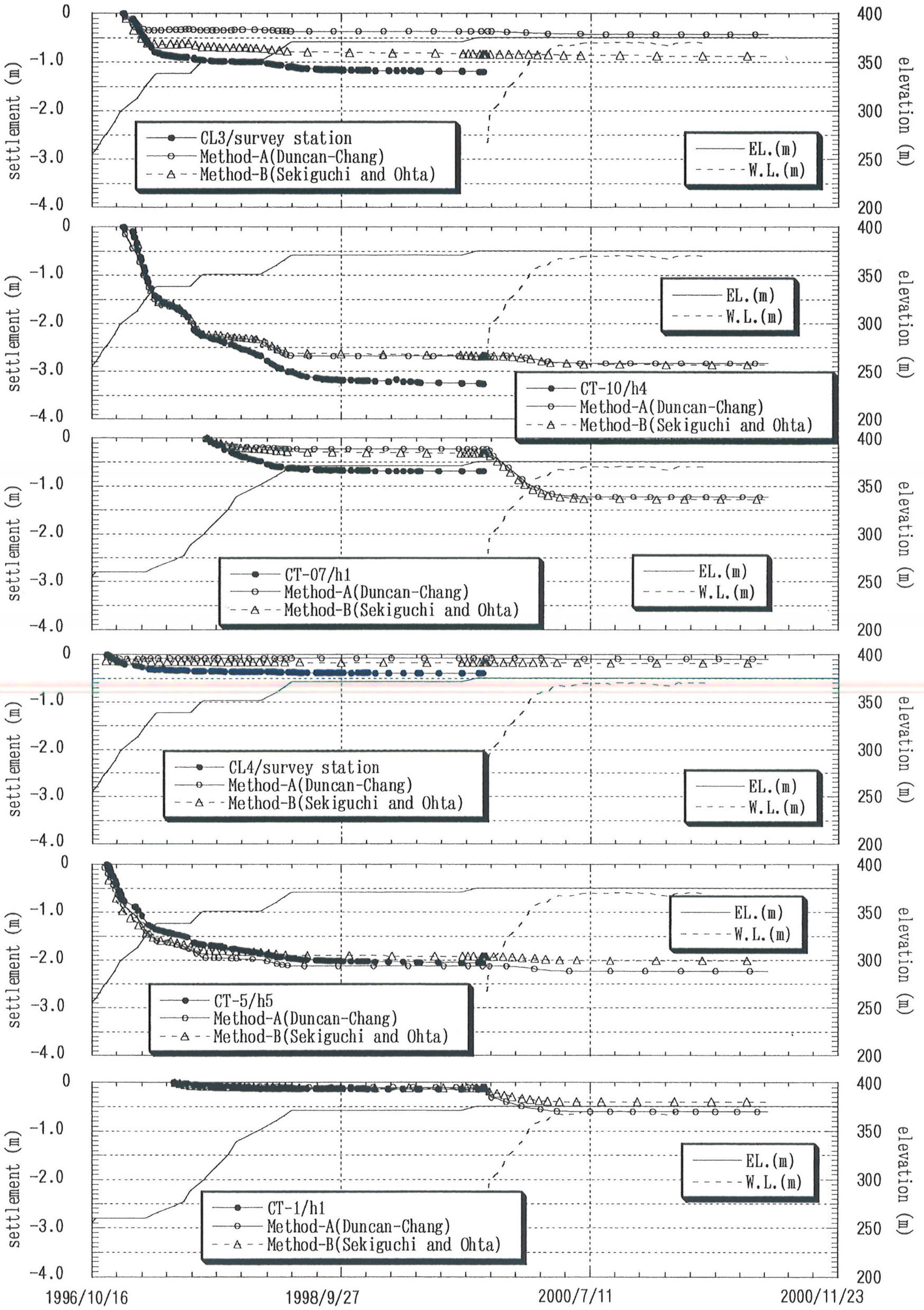

Fig.4 Comparison of measured and calculated settlements of dambody during construction and the first impounding (CL3,CT-10,CT-7,CL4,CT-5,CT-1)

3.2 Horizontal Displacements

- Comparisons between measurements of horizontal displacements and analytical results are shown in Figs. 5 and 6. In Figs. 5 and 6, measurements up to completion of the construction are plotted, and analytical results up to completion of the first impounding are plotted.
- As shown in Figs. 5 and 6, analytical results on the horizontal displacements of the dambody are likely to be almost reproducible in the Method-A and the Method-B, except for CL-12, CT-14, and CT-10.
- As with the settlements described above, the horizontal displacements during the first impounding exhibit a tendency of increasing at sections closer to the upstream end of the dambody, and the horizontal displacements at sections closer to the downstream end are not so much as the upstream end are. The horizontal displacements of the upstream surface of the dambody during the first impounding are estimated to be up to about 0.7 m in Method-A, and about 0.5m in Method-B.

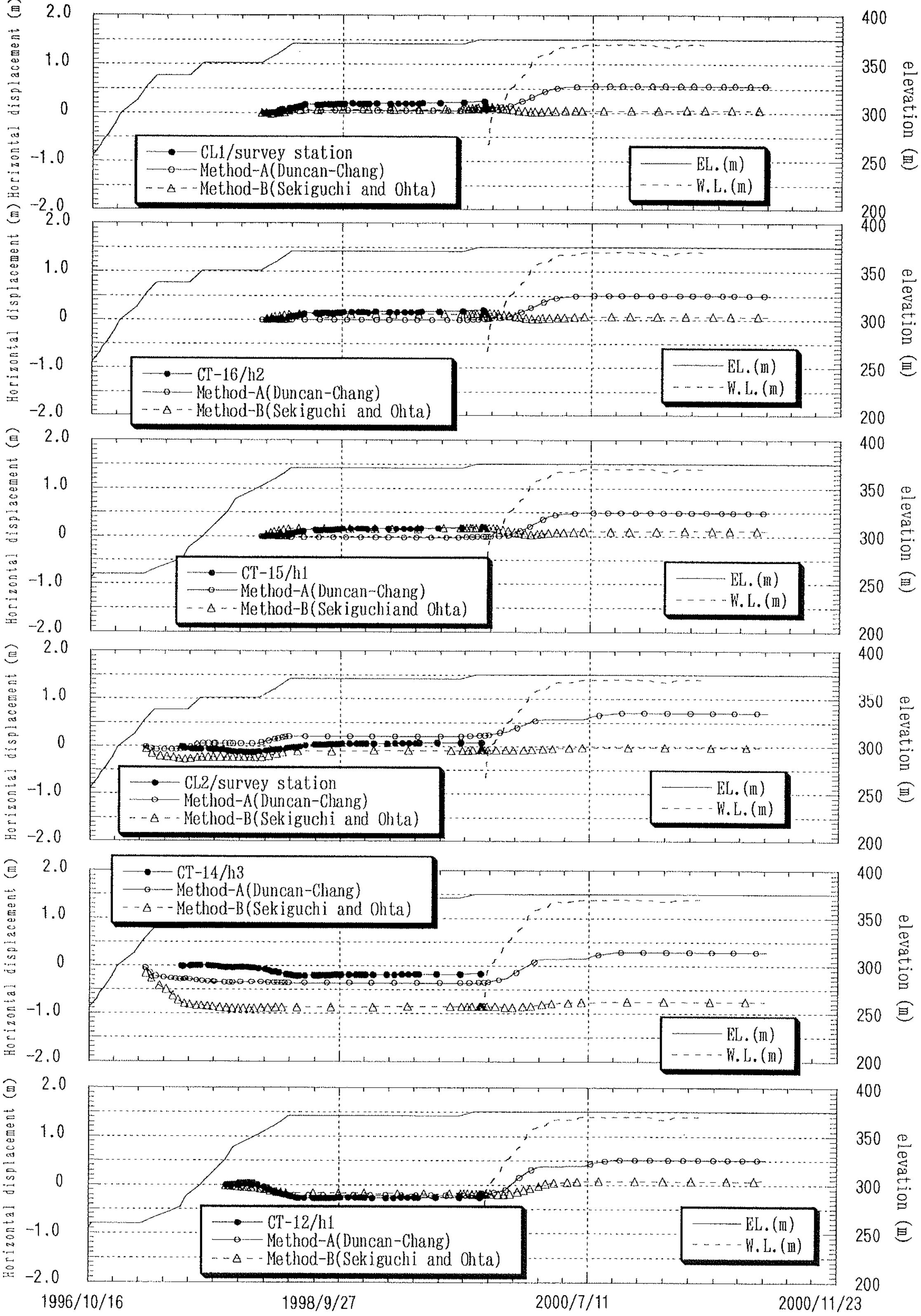

Fig.5 Comparison of measured and calculated horizontal displacements of dambody during construction and the first impounding (CL1,CT-16,CT-15,CL2,CT-14,CT-12)

Y.Uchita, Y.Hoshino, H.Yamaguchi, T.Ishiguro,:
Prediction of the upstream face deflection of a CFRD during its first impounding with two numerical models

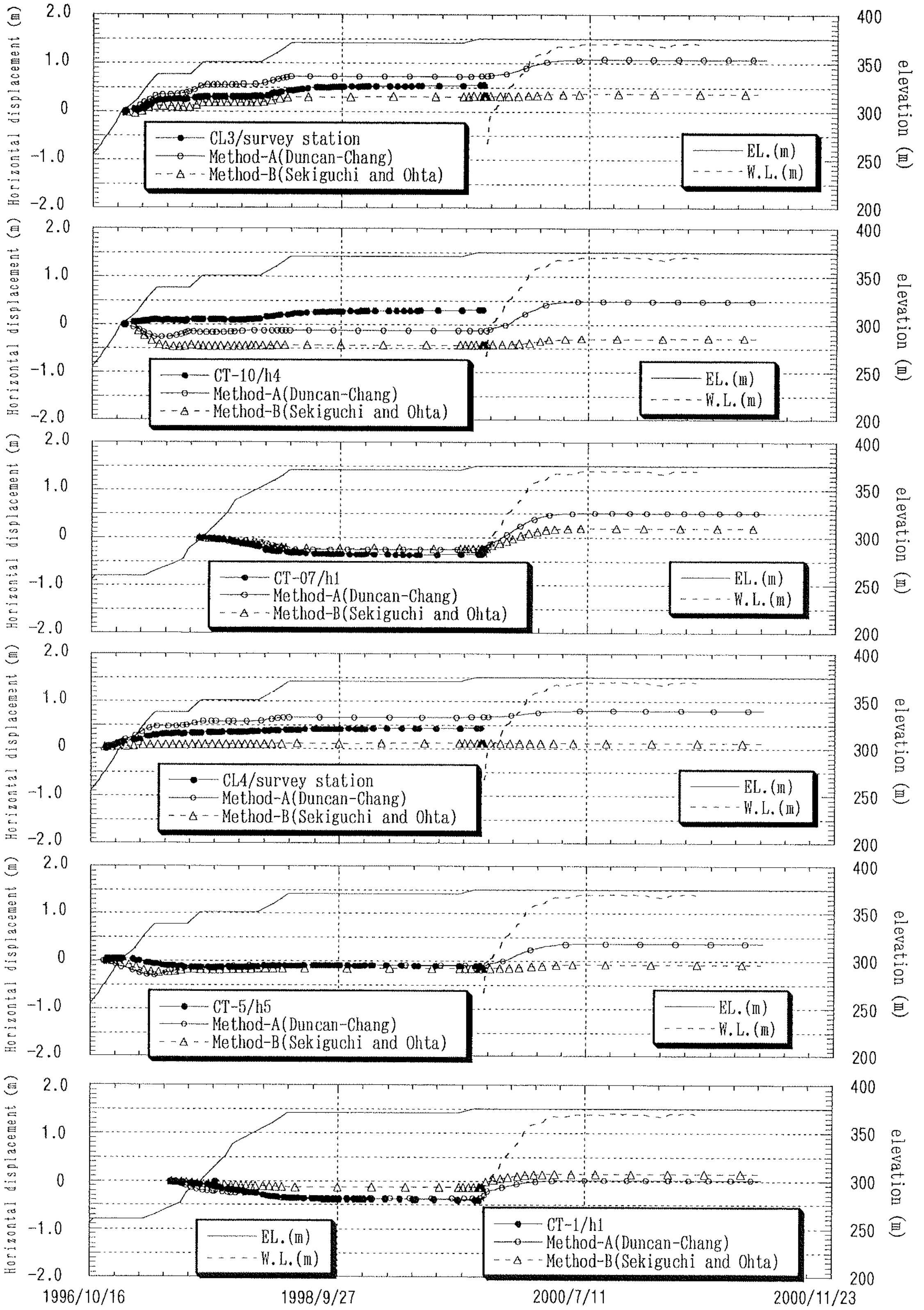

Fig.6 Comparison of measured and calculated horizontal displacements of dambody during construction and the first impounding (CL3,CT-10,CT-7,CL4,CT-5,CT-1)

3.3 Behavior of the Face Slab

3.3.1 Amounts of deformation

- The distribution of displacement normal to the face slab at the water level 371m is as shown in Fig. 7. This figure indicates that displacement reaches up to about 1.2 m in Method-A, and about 1.0m in Method-B. The position occurring the maximum value is near the center of the face slab in both methods, the difference of both methods is slight.
- The distribution of the displacements exhibits an almost smooth shape pattern from the plinth to the crest in Method-A. Meanwhile, an inflection point occurs at a distance from about 20 m from the plinth in Method-B. This position is equivalent to the boundary between the E0 layer and E1 layer of the rock zone, and the above-mentioned inflection is likely to occur due to the difference in rigidity between both layers.
-As shown in the "BW6_RESTEPCO.xls" file, the displacements in a direction tangential to the slab is very small in both methods.

3.3.2 Cross-sectional stresses

- Normal force, shear force, and bending moment are shown in Figs. 8 to 10.
- In Method-A, a normal force of about 9,000 kN, a shear stress of about 400 kN, and a bending moment of about 3,500 kN.m are exerted as cross-sectional maximum stresses in the range of a quarter or less of the dam height.
- In Method-B, a normal force of about 10,000 kN, a shear stress of about 300 kN, and a bending moment of about 2,000 kN. Each value of cross-sectional stresses is different in both methods, but the difference is slight in a fundamental tendency.
- In both methods, it can be seen from the distribution diagram of normal force that tensile force is exerted on the entire facing. Also, it can be seen from the distribution diagram of bending moment that the section of the facing that is near the plinth basically suffers convex deformation in the direction of the upstream. And the section far away from the plinth suffers convex deformation in the direction of the downstream.

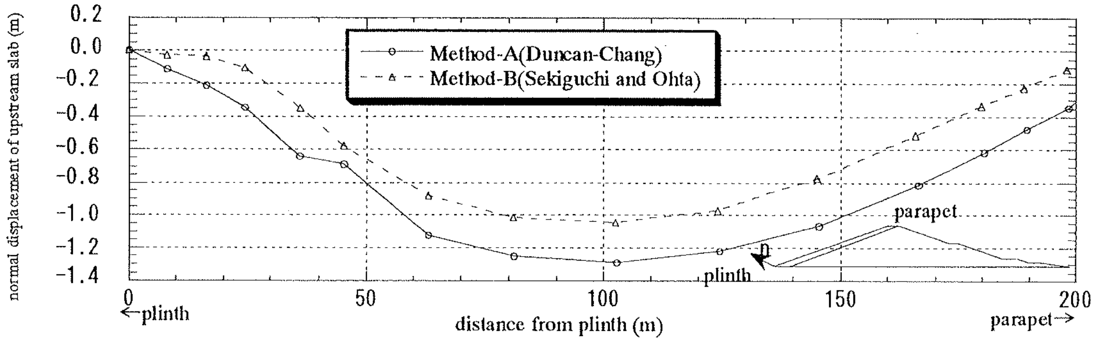

Fig.7 Normal displacement of upstream slab during impounding (W.L.=371m)

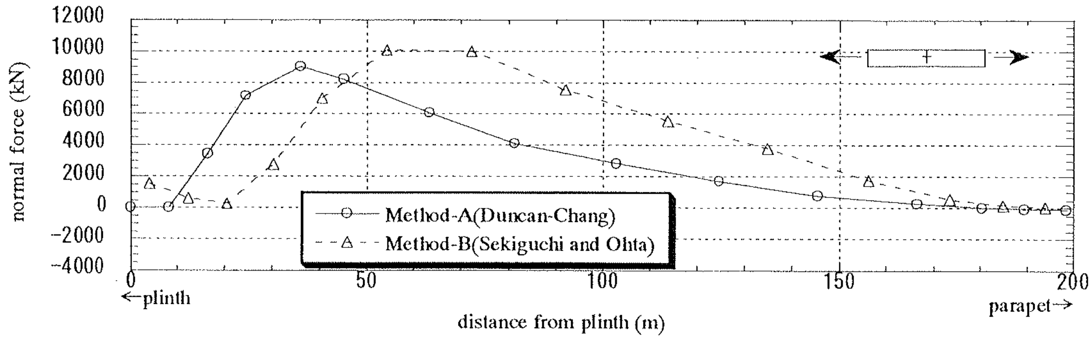

Fig.8 Normal force of upstream slab during impounding (W.L.=371m)

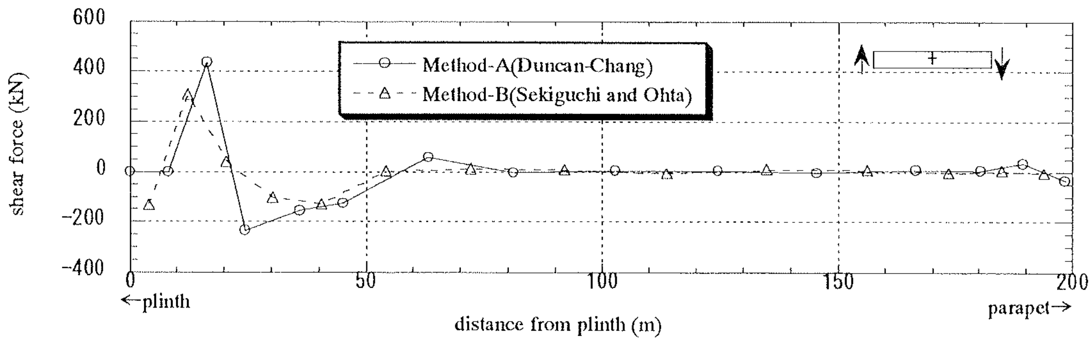

Fig.9 Shear force of upstream slab during impounding (W.L.=371m)

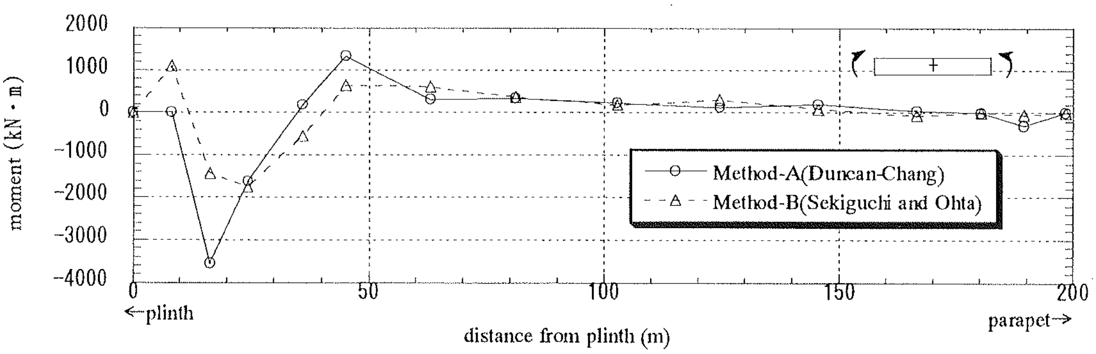

Fig.10 Bending moment of upstream slab during impounding (W.L.=371m)

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PREDICTION OF CONCRETE FACE ROCKFILL DAM BEHAVIOUR DURING THE FIRST IMPOUNDING – A CASE STUDY

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ABSTRACT: A numerical study of Itá Concrete Face Rockfill Dam (Uruguay River - SC, Brazil) is performed, following two numerical procedures : an elastic predictive analysis and an elasto-plastic back-analysis. Comparisons between in-situ measurements and results obtained by means of the Finite Element Method show deficiencies in elastic modelling and lead to two physical assumptions for the understanding of the atypical behaviour of Itá dam.

Key Words: Concrete Face Rockfill Dam, Non-linear numerical analysis, Upstream face deflections, Rockfill settlements, Rock blocks crushing.

1 INTRODUCTION

Concrete Face Rockfill Dams are commonly of empirical design. According to many studies, static finite element analysis is used to predict dam body displacements, in order to verify designs of upstream concrete face, perimetral joint or monitoring systems. Such analysis requires assumptions like boundary conditions, mechanical properties of materials and load cases, which identification is frequently more based on the "know-how" of the engineer, than on realistic aspects. For example, in many cases, experimental tests on rockfill materials are not available for various reasons (costs of such tests, availability of testing apparatus, samples representability...) and penalise both the quality and the accuracy of numerical analysis. The engineer is then naturally led to elastic modelling, in order to minimise effects of missing data and facilitate qualitative interpretation of results.

The purpose of the present paper is to highlight the necessity of elaborated numerical tools, both to design Concrete Face Rockfill Dams and to interpret their behaviour. Itá dam (Uruguay River, Santa Catarina, Brazil) which experienced large slab deflections during the first impounding, is used to illustrate the subject.

2 DESCRIPTION OF ITÁ CONCRETE FACE ROCKFILL DAM

The Itá dam is a Concrete Face Rockfill Dam with a maximum height of 125 m, an upstream slope of 1,3H:1V and a downstream slope of 1,2H:1V. The dam construction required a total volume of 8,7 millions m^3 including basalt of various quality. Many design developments were involved, such as the curb facing, face protection during construction, plinth geometry and the use of Jeene joints.

The Project construction started on March 1st, 1996. The river was diverted into the diversion tunnels on Sept. 30th, 1997 and the reservoir filling begun by the end of the year 1999. Commercial Operation of the first unit started on April 2000. During the reservoir impounding, seepage through the dam body was observed, reaching up to 1724 l/s (May 12th, 2000). The cause was cracks in the slab, located 10 to 15 meters above the plinth, on the two abutments. Leakage was reduced to 340 l/s by dumping artificial sand and silty-sandy material.

2.1 Construction materials and foundation conditions

2.1.1 Dam Foundation

The criterion used for the upstream one-third of the dam foundation was to expose weathered rock in 50% of the area, accepting some soil pockets. In the downstream two-thirds, the foundation on saprolite presented deformation modulus higher than those of the overlying rockfill. Rock blasting was limited to the plinth foundation. Along the 25 m downstream from the plinth, the rock was fully exposed, without blasting.

2.1.2 Downstream Zone

The downstream zone of a CFRD is delimited by the 2/3 downstream part of the dam. In this zone it was accepted almost the entire required rock excavation. The worse material composed by weathered rock was set in central regions and enveloped by good rock. Theses zones were compacted with 4 passes of a 9 ton vibratory roller in layers of 1,6 m. The average in-situ density reached about 2,12 t/m^3.

2.1.3 Upstream Zone

The upper one-third of dam, constituting the upstream zone, was compacted in 0,80 m layers except a 10 m layer of rockfill dumped in the riverbed. The dense basalt rockfill (min. 70%) and transitions were placed in a 35 m horizontal zone adjacent to the face slab. Compaction at this zone was with 9 ton vibratory roller, 6 passes and wetted at a rate of 100 l/m^3. In this zone the average density achieved was 2,12 t/m^3.

2.1.4 Transitions

The transition behind the face is important for various reasons in the design of CFRD such as : to prevent voids between concrete and rockfill, to control seepage in case of cracks, to protect the dam partially built in case of floods and to function as a pad for concreting the slab. Transition should also be as stiff as possible in order to reduce the slab deformations. Therefore, the use of some fines is desirable. In Itá, a 7 m wide combined transition with crushes (max. diam. 76,5 mm) and fine rockfill was compacted in layers of 0,40 m. "In situ" transition density was particularly high reaching up to 2,40 t/m^3. The crushed transition is 12 m wide adjacent to the plinth and rises with a slope of 2V:1H until reaching the width of 7 m.

2.2 *Concrete face*

The slab thickness was defined as e = 0,30+0,002 H. The concrete strength was specified as fck = 21 MPa at 90 days. The reinforcing is distributed along the neutral line of the slab section following the criterion St = 0,004 Sc for the vertical direction and St = 0,003 Sc for the horizontal direction, St being the steel section and Sc the concrete section. Each slab has a 16 m width.

3 INSTRUMENTATION

The dam design includes extensive monitoring of the following type :

- Upstream concrete face :
    - slab deflection : slabs n°20 (ch. 25+0,00) and n°29 (ch. 32+8,19) have been respectively instrumented with 10 and 15 electro-levels to measure displacements normal to the concrete slab.
    - joint opening : joint meters are installed at plinth-slab joint for slabs n°20, 29 and 45, at slab-slab joints between slabs n°7 to 10 and n°44 to 49, and at slab-parapet joints.

- Dam body :
    - settlement : settlement was measured in two dam sections, corresponding the to upstream slabs n°29 (main section – ch. 32+8,19) and n°45 (right bank – ch. 42+8,19), respectively with 16 settlements gauges installed at four different elevations and 10 settlements gauges installed at three different elevations. At each level in each section, the first instrument is located close to the upstream face thus allowing for comparison with data from the concrete slab.
    - horizontal displacements : horizontal plate gauges have been installed at the same locations as the settlement gauges.

- Downstream face :
    - 19 Geodetic survey gauges have been installed on a grid defined by 5 elevations and 7 vertical sections, corresponding to slabs n°6, 14, 20, 29, 35, 40 and 45.

The transversal section studied herein is the main section located at the 32+8,19 chainage and corresponding to slab n°29. Figure 1 shows the instruments position on the main section.

4 NUMERICAL MODELS AND PARAMETERS

Two different numerical procedures are used for the analysis concerning the dam behaviour during impounding : a predictive analysis, based on construction phase monitoring measurements and using an elastic-perfectly plastic model, and a back-analysis, based on construction and impounding phase monitoring measurements and using a multi-mechanism elasto-plastic model.

4.1 *Elastic-perfectly plastic model*

The elastic-perfectly plastic Drucker-Prager constitutive model inscribed into the Mohr-Coulomb model was adopted. Plastic deformations were calculated considering an associated flow rule. The parameters used for the analysis by means of this model are listed in Table 1.

In several large CFRD, it has been determined that the "vertical modulus" of rockfill (that means "vertical elastic equivalent modulus") measured during construction differs from the one normal to the concrete face observed during reservoir impounding. Such difference of modulus is attributed to probable effect of anisotropic deformability of compacted rockfill or to stress state spherization inducing a deviatoric discharge (that involves a higher modulus), without real understanding of the physical mechanisms. In the analysis presented herein, the elastic modulus during the impounding phase is assumed to be 100% greater than the elastic modulus during the construction phase, and the Poisson's ratio is reduced to 0,1, according to usual numerical CFRD studies.

Table 1 : Parameters for the elastic-perfectly plastic Drucker-Prager constitutive model.

	T1		E0		E1		E2/E3		E1A		E2A	
(note)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
γ (kN/m^3)	24,0	24,0	21,9	21,9	21,2	21,2	21,2	21,2	21,2	21,2	21,2	21,2
E (MPa)	100	200	60	120	40	80	15	30	10	20	10	20
ν	0,3	0,1	0,3	0,1	0,3	0,1	0,3	0,1	0,3	0,1	0,3	0,1
ϕ	45	45	45	45	45	45	45	45	45	45	45	45
c (MPa)	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1	0,1
ψ	40	40	40	40	40	40	40	40	40	40	40	40

(1) Construction phase
(2) Reservoir impoundment phase

4.2 *Multi-mechanism elasto-plastic model*

A constitutive non-associated elasto-plastic model is proposed within the framework of isotropic hardening plasticity using effective stress and characteristic state concept. In this model, four mechanics are defined : three plane strains mechanisms associated to three perpendicular planes for the deviatoric behaviour, and a mechanism for the isotropic behaviour. The four mechanisms are coupled. A cinematic condition is introduced in the deviatoric mechanisms to allow the creation of volumetric plastic strains under deviatoric loading.

Water addition effects, as consequences of lubrication and rock breakage at block contacts, are fully integrated in the four mechanisms. The parameters used for the analysis by means of this model are listed in Table 2.

Table 2 : Parameters for the multi-mechanism elasto-plastic model.

	T1	E0	E1	E2/E3	E1A	E2A
γ (kN/m^3)	24,0	21,9	21,2	21,2	21,2	21,2
K (MPa)	266	200	133	133	133	133
G (MPa)	122	91	61	61	61	61
n	0,5	0,5	0,5	0,5	0,5	0,5
ϕ	36	36	36	46	46	46
ψ	46	46	46	46	46	46
β	-10	-10	-10	-10	-10	-10
P_{c0} (MPa)	-0,9	-0,9	-0,58	-0,33	-0,25	-0,25
a_m	0,0013	0,0016	0,0022	0,0026	0,0032	0,0032
b	0,3	0,3	0,3	0,8	0,8	0,8
r_{ela}	0,001	0,001	0,001	0,001	0,001	0,001
r_m	0,001	0,001	0,001	0,001	0,001	0,001
c_m	0,004	0,006	0,008	0,0036	0,004	0,004
d	2	2	2	2	2	2
r_{isoela}	0,001	0,001	0,001	0,001	0,001	0,001
α	1	1	1	1	1	1

5 MODELLING CONSTRUCTION PHASES

The dam construction in layers has been simulated respecting the construction phases as follows :

- construction of the internal priority section up to El.348, according to Figure 2 ;
- construction of the upstream part and transition zone ;
- construction of the concrete face.

The average layer thickness involved during the numerical procedure is about 7 meters, excepting the first 20 m layer of dumped rockfill in the riverbed.

All model parameters were adjusted with in-situ measurements during the construction phase in order to approach the real stress/strain state in the dam body at the end of construction. Figures 3 to 10 show comparisons between computed settling and in-situ measurements along horizontal profiles, respectively located at El.272, El.301, El.325 and El.350, at various stages of construction. Calculated settlements follow the trend and magnitude of values observed in-situ.

In each analysis, the concrete face was modelled by beam elements, connected on upstream nodes of elements constituting transition material. The construction of the concrete face was done in layers, in 5 steps.

6 MODELLING RESERVOIR IMPOUNDING

The effect produced by reservoir impounding has been simulated applying hydrostatic pressure load along the upstream concrete face. Computation results are discussed below.

7 DISCUSSION

7.1 Dam monitoring during impounding phase

The main conclusions of dam monitoring analysis during impounding phase are :

- Large movements of the crest and the downstream face ;
  Crest settlements on the main section reached 62 cm, three months after the end of reservoir filling. The horizontal displacement on the same monitoring point reached 40 cm. On the downstream face, settlements and horizontal displacements monitored varied respectively between 25 cm and 50 cm, and between 20 cm and 35 cm, from El.300 to El.350.

- Maximum deflection located on the upper part of the dam ;
  A maximum deflection value of 70 cm was measured close to the dam crest, in monitoring section located on Right Bank. On the main section, the maximum deflection reached 50 cm in the mid-height of the dam, approximately.

- Bending of the concrete face close to the upstream toe of the dam ;
  The deflection evolution along a slab in a transversal section, shows a curvature inversion in the lower part of the slab, corresponding to concrete face cracking zone observed during submarine inspection. The underwater robot investigation was decided after a consequent leakage increase that occurred when the reservoir level reached El.360. It was found a cracking zone extended along the two banks. Cracks appeared systematically close to the toe of the slabs and parallel to the plinth. Average and maximum opening of the observed cracks were about 5 mm and 10 mm respectively.

7.2 Rockfill behaviour

In order to compare the difference between the rockfill behaviour estimated with FEM modelling and the in-situ rockfill behaviour, it was proposed to analyse :
- the settlements along three horizontal lines, respectively on El.300, El.325 and El.350, corresponding to settlement gauges line position[1], at different water levels ;
- the displacements isovalues at the end of the impounding.

[1] the monitoring cabin at El.272 has been out of order since Oct/00. Various instruments connected to this cabin were previously damaged by an inundation that occurred during the reservoir filling .

7.2.1 Elastic-perfectly plastic model

As shown on figures 11 to 15, results obtained by the Drucker-Prager model are different from in-situ measurements, according to the following:

- settlements values are underestimated, especially downstream from the dam axis with a maximum difference of 25 cm by the end of the impounding (Jul/00), reaching up to 40 cm in Oct/00 ;
- the model does not respect the trend of displacements. The isovalues of calculated displacements show a maximum displacements location, close to the upstream face, at 2/3 height of the dam approximately, that doesn't correspond to the realistic scheme (maximum values located at the dam crest).

The numerical behaviour can be easily explained considering the stress trend in the dam body : downstream from the dam axis, the stress increment calculated doesn't allow to create movements as significant as those observed, with adopted elastic moduli. To obtain displacements of the same order of magnitude on the downstream face for example, a decrease in elastic moduli is required.

The only way to justify a decrease in rockfill deformability modulus under low stress variation, is considering the occurrence of collapse phenomenon. This phenomenon is characterised by a rockfill volume reduction under constant load, with the addition of water, which can be physically explained by the coupled action of lubrication and rock breakage at rock contacts. But this assumption requires water entry in the dam body during the reservoir filling, such as infiltration through the concrete face, infiltration through the foundation, heavy rains or downstream cofferdam overtopping. In the case of Itá, monitoring observations reveal consequent flows into various concrete face cracks and downstream water levels exceeding the downstream cofferdam crest elevation, that could justify water inflow and therefore, a probable collapse event.

7.2.2 Multi-mechanism elasto-plastic model

A second analysis is performed with a multi-mechanism elasto-plastic model, following a back-analysis scheme. As shown on figures 16 to 20, the trend and magnitude of values observed on the downstream face of the dam are respected by the modelling.

In order to verify if the material behaviour, as modelled in the numerical analysis, is physically admissible, tests were simulated using parameters adjusted by the back-analysis. Oedometric and triaxial curves obtained are represented on figures 21 to 24. The behaviour of materials E2A and E3 is similar to the behaviour of weathered rockfill (Veiga Pinto 1983), as represented on figure 25. Main characteristics of this behaviour are :

- a convex oedometric curve, i.e. an inverted curvature in comparison with the classic concave curve that traduces the material rearrangement leading to a more compact state ;
- a missing peak failure on triaxial curves for low confining pressures ;
- high volume reduction during the shear phase of triaxial test.

This kind of behaviour is typically associated to an intense particle crushing or grain breakage. This phenomenon increases basically with a poor quality mineralogy, an uniform gradation, a high blocks angularity and the presence of water (Anthiniac 1999, Veiga Pinto 1983), and could lead to consequent volumetric deformations induced by blocks rearrangement.

In the light of the modelling, it is found that the occurrence of rock blocks crushing, due to poor quality rockfill composing the downstream part of the dam, can fully explain the excessive deformations observed during the impounding phase.

7.3 Concrete face behaviour

It was proposed to analyse concrete face deflections and concrete Stress State along the face (bending moment, normal force and shear force).

Displacements perpendicular to the concrete face are calculated from vertical and horizontal displacements of upstream nodes. As shown on figure 26, the bending of the slab in the main section is satisfactorily estimated, just as well with the Drucker-Prager model, as with the multi-mechanisms elasto-plastic model, despite the bad results of the elastic modelling in the downstream part. The origin of this bending is the location, few meters below the concrete face, of a sub-vertical interface (1H:2V) between two materials with heterogeneous mechanical properties (E0 and E1). This upstream toe dam zoning created a strong support for the slab that, associated to greater deformations above this zone (due to the presence of materials with greater deformability below the transitions) caused the bending.

Bending moments, normal forces and shear forces, developed in the concrete slab at the end of the reservoir filling, are directly obtained from beam element results of finite element analysis. The calculated bending moments along the slab are represented on figure 27. Critical bending moments are concentrated around El.260, corresponding to the normal projection on the face, of the lower E0/E1 interface point.

Normal force evolution, as shown on figure 28, reveals a tension zone in the lower 20 m of the face and a compression zone above. Considering the boundary condition imposed to the slab base (horizontal and vertical displacements null), the slab tensions, close to the plinth, are overestimated. However, it is interesting to note the presence of a constant tension level around El.260, in accordance with the maximum bending moment. The use of a plinth/slab joint modelling would probably lead to a maximum tension zone close to El.260.

Figure 29 shows the evolution of shear force along the slab. The magnitude of variations is very small and the maximum shear force reaches about 1tf.

8 CONCLUSIONS

The behaviour of Itá Concrete Face Rockfill Dam during the impounding phase is characterised by large movements of the crest and the downstream face, and by a concrete face bending close to the plinth.

Two numerical procedures were used to simulate the entire behaviour of the dam, including both construction by layers and reservoir filling : a predictive analysis, based on construction phase monitoring measurements and using an elastic-perfectly plastic model, and a back-analysis, based on construction and impounding phase monitoring measurements and using a multi-mechanism elasto-plastic model.

Both analyses provided a good representation of the concrete slab bending. It is found that the origin of this bending is the presence, few meters below the concrete face, close to the upstream dam toe, of two materials with a very significant difference in deformability (E0 and E1).

In contrast, the excessive deformations observed in the downstream part of the dam, are largely underestimated by the Drucker-Prager analysis. Two hypothesis are proposed to explain the lack of the elastic modelling during the impounding phase :

- Intense grain crushing for materials E1A, E2A and E3, due to a poor rockfill mineralogy (weathered rockfill) ;
  The occurrence of intense grain crushing in the dam body, leads to incremental volumetric deformations greater than expected, during the impounding phase. Experimental tests on equivalent rockfill material are planned in order to justify this assumption.

- Wetting collapse of rockfill materials due to water infiltration through the concrete face and/or overtopping of the downstream cofferdam.
  Wetting collapse phenomenon consists in a reduction of the material volume, under constant load, with the addition of water. Physically, this phenomenon is explained by the coupled action of the lubrication effects induced by water, and rock breakage at block contacts, that is amplified by the presence of water on rock surface. Water inflows in Itá could be justified by the appearance of cracks on the concrete face, close to the upstream toe, with consequent leakage, and by downstream water levels that reached elevations higher than the crest of the downstream cofferdam during the impounding phase.

It is noteworthy that the phenomenon of grain crushing is present in these two assumptions. A combination of weathered rockfill and wetting collapse is certainly the most probable configuration occurred.

Based on these understanding aspects, the necessity of elaborated elasto-plastic model in order to represent the behaviour of CFRD is highlighted. In the case of Itá, an elastic modelling leads to an incorrect trend of displacements and a large underestimation of settlements. Therefore, numerical analysis of CFRD can not be restricted to the elastic model without prejudice to the physical phenomena understanding : future improvements in CFRD designs require numerical analysis using elaborated elasto-plastic models.

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FIGURES

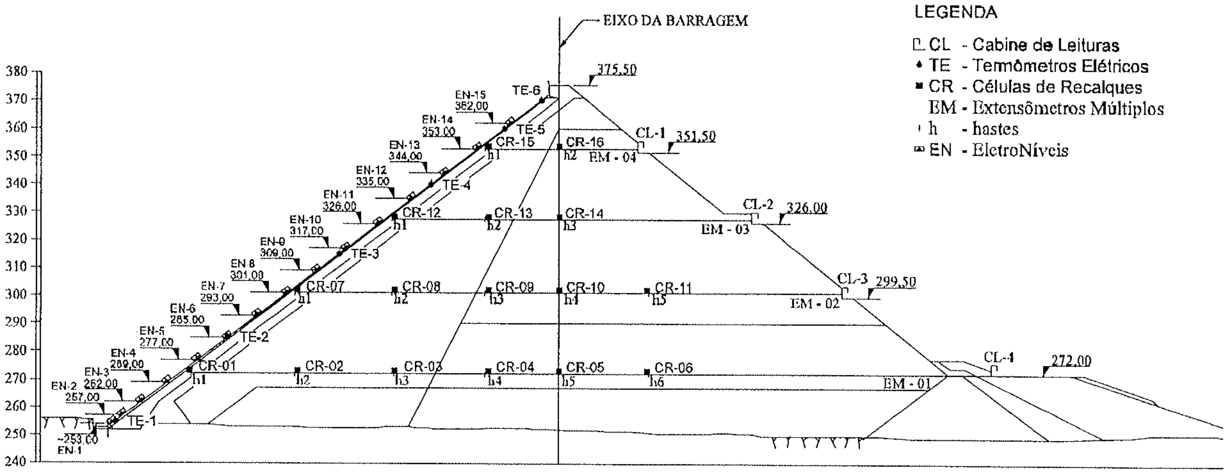


Figure 1 – Monitoring Equipment in the main section.

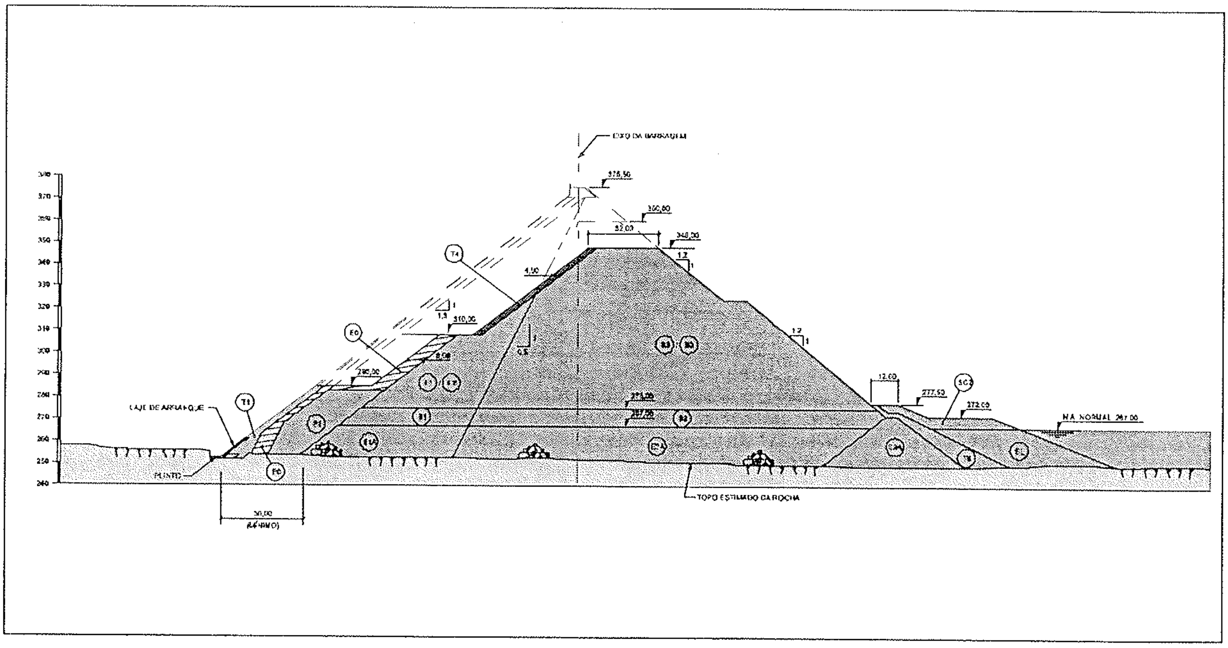
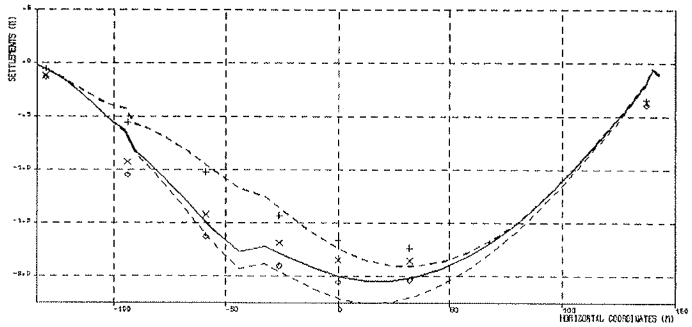
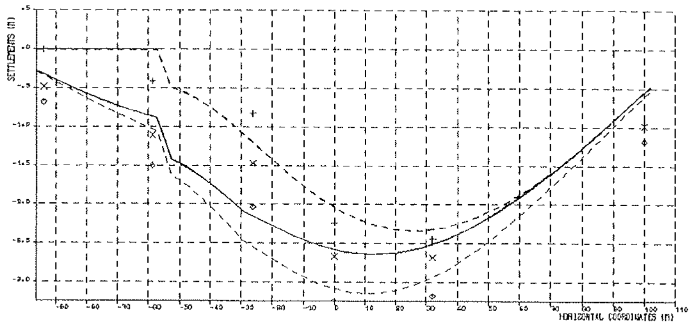


Figure 2 – Priority section and Dam zoning



STEP 1 27/05/98 EL.300M/346M
STEP 2 17/09/98 EL.346M
STEP 3 29/11/99 EL.375.5M
+ MEASUREMENTS IN-SITU 27/05/98
x MEASUREMENTS IN-SITU 17/09/98
◇ MEASUREMENTS IN-SITU 29/11/99

MEASUREMENT CABIN CL-4



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STEP 2 17/09/98 EL.346M
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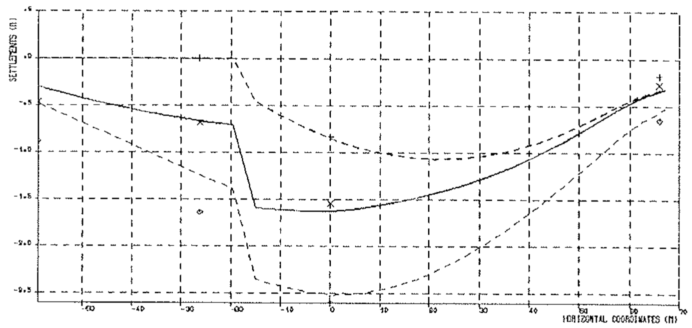
x MEASUREMENTS IN-SITU 17/09/98
◇ MEASUREMENTS IN-SITU 29/11/99

MEASUREMENT CABIN CL-3



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STEP 2 17/09/98 EL.346M
STEP 3 29/11/99 EL.375.5M
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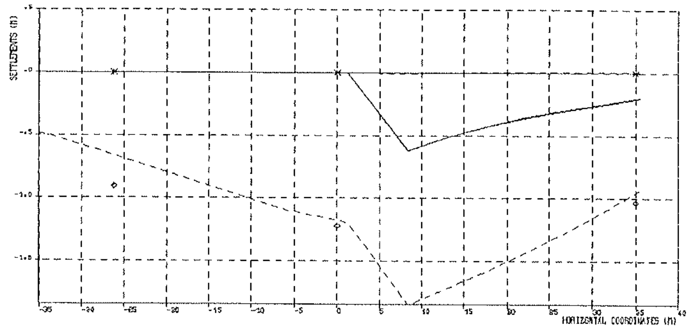
◇ MEASUREMENTS IN-SITU 29/11/99

MEASUREMENT CABIN CL-2



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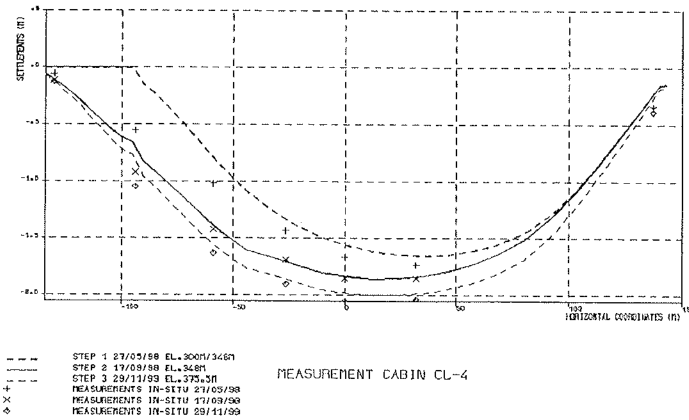
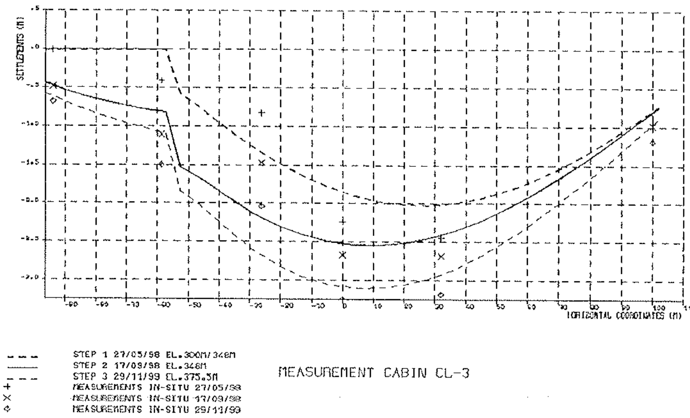
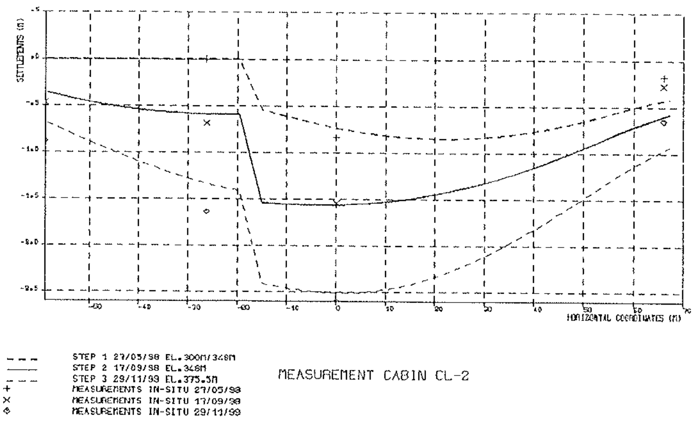
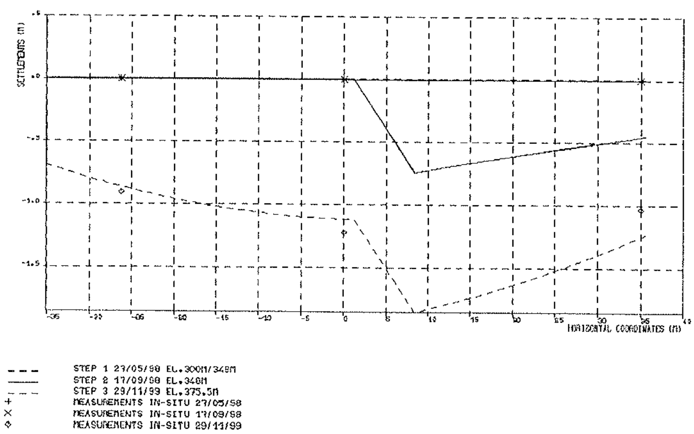
Figures 3 to 6 : Settlements along horizontal profiles – Construction by layers – Drucker-Prager.

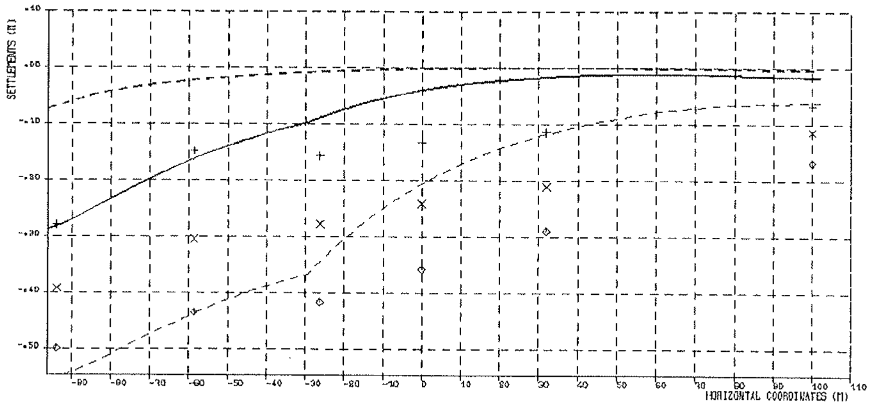








Figures 7 to 10 : Settlements along horizontal profiles – Construction by layers – Multi-mechanisms.



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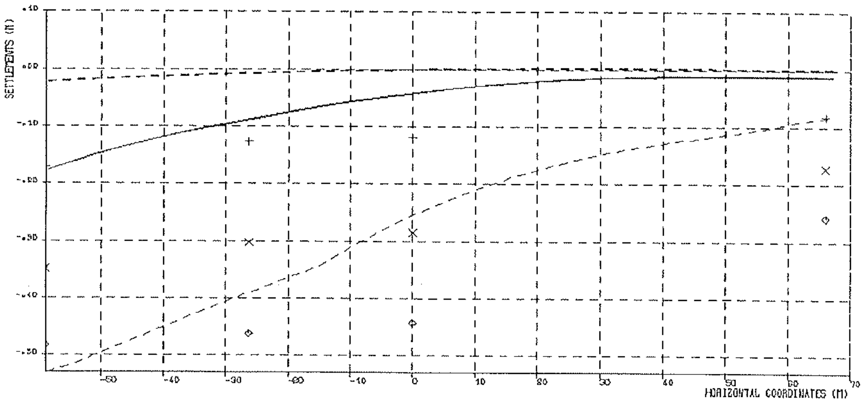
STEP 2 20/04/00 WL.364.0m
STEP 3 06/07/00 WL.370.0m
\+ MEASUREMENTS IN-SITU 08/03/00
× MEASUREMENTS IN-SITU 20/04/00
◇ MEASUREMENTS IN-SITU 06/07/00

MEASUREMENT CABIN CL-3



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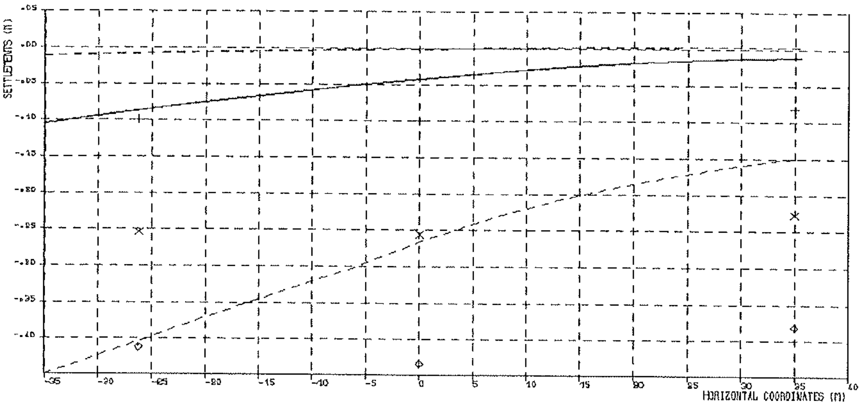
◇ MEASUREMENTS IN-SITU 06/07/00

MEASUREMENT CABIN CL-2



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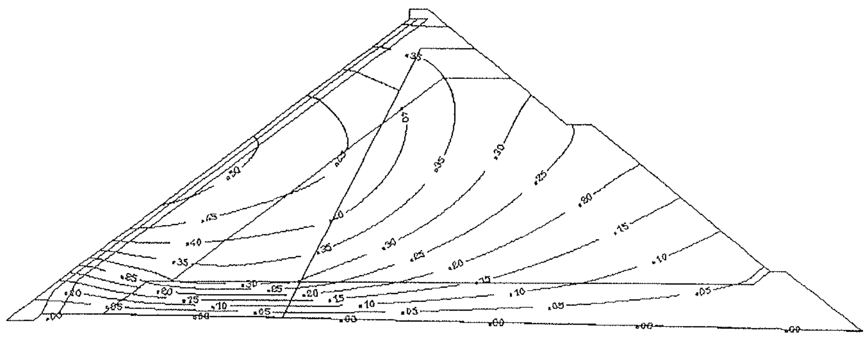
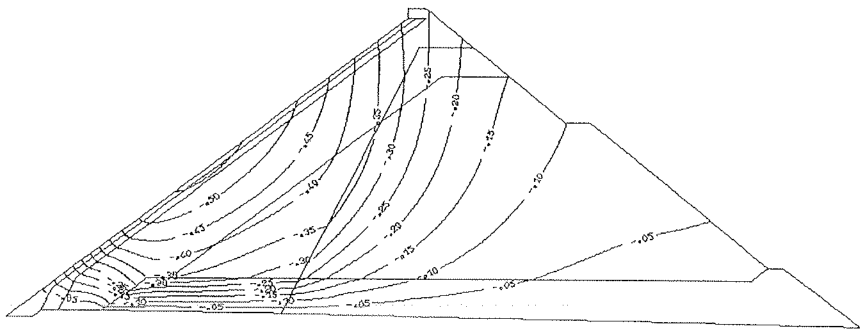
\+ MEASUREMENTS IN-SITU 08/03/00
× MEASUREMENTS IN-SITU 20/04/00
◇ MEASUREMENTS IN-SITU 06/07/00

MEASUREMENT CABIN CL-1

Figures 11 to 13 : Settlements along horizontal profiles – Impounding phase – Drucker-Prager.



GEOMETRICAL SCALE : .00070 0 20
ARIABLE DEPLY ETAPE 4 TEMPS = 0.9000E+01



GEOMETRICAL SCALE : .00070 0 20
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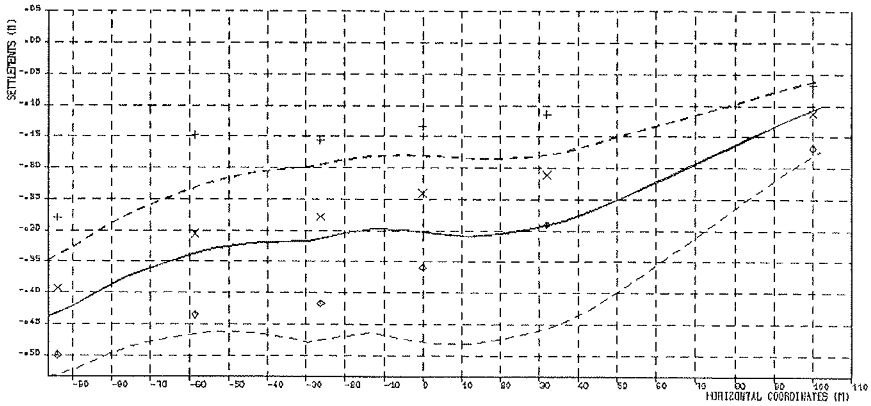
Figures 14 to 15 : Isovalues of displacements at the end of the impounding – Drucker-Prager.



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STEP 2 20/04/00 WL.364.0M
STEP 3 06/07/00 WL.370.0M
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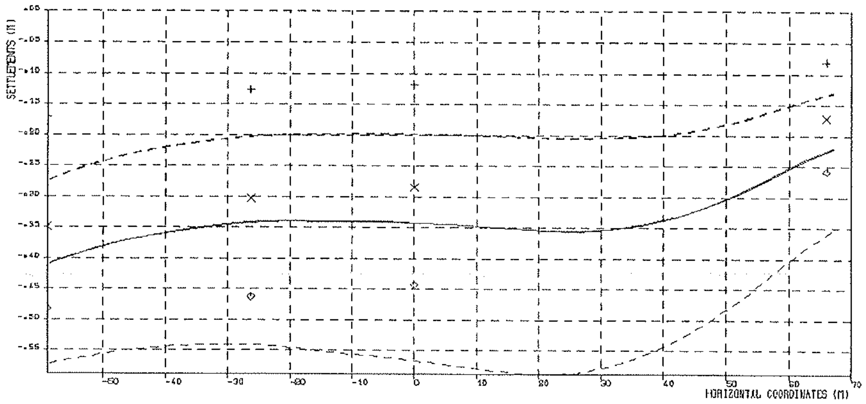
× MEASUREMENTS IN-SITU 20/04/00
◇ MEASUREMENTS IN-SITU 06/07/00

MEASUREMENT CABIN CL-3



STEP 1 08/03/00 WL.350.0M
STEP 2 20/04/00 WL.364.0M
STEP 3 06/07/00 WL.370.0M
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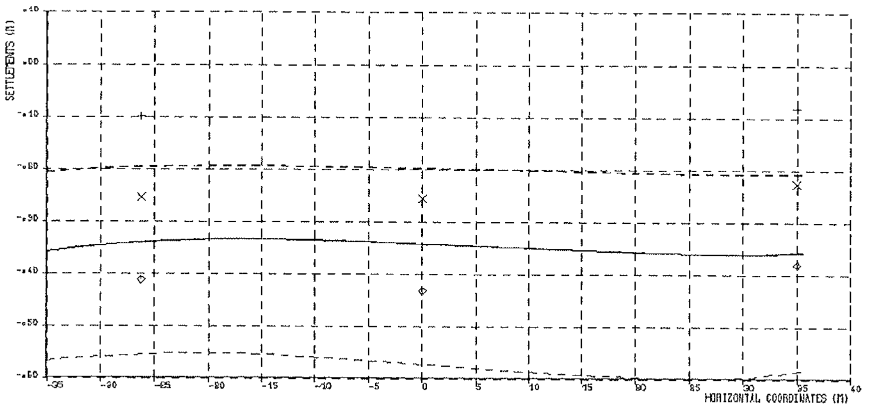
× MEASUREMENTS IN-SITU 20/04/00
◇ MEASUREMENTS IN-SITU 06/07/00

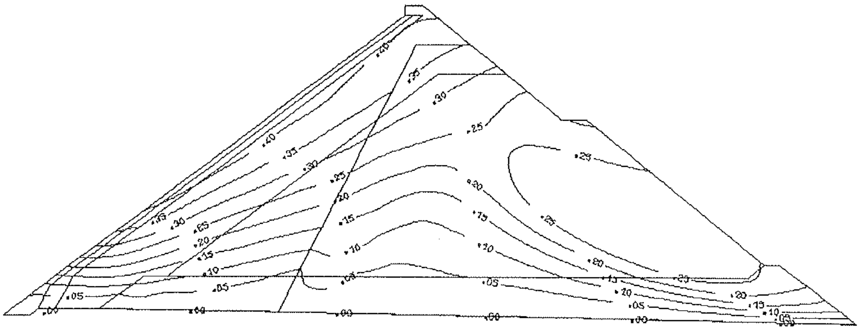
MEASUREMENT CABIN CL-2



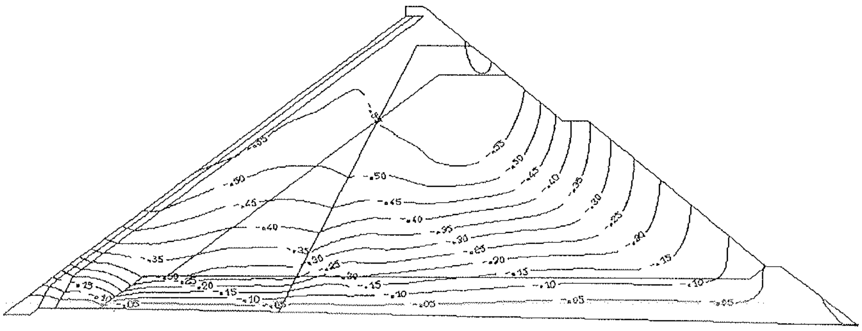
STEP 1 08/03/00 WL.350.0M
STEP 2 20/04/00 WL.364.0M
STEP 3 06/07/00 WL.370.0M
\+ MEASUREMENTS IN-SITU 08/03/00
× MEASUREMENTS IN-SITU 20/04/00
◇ MEASUREMENTS IN-SITU 06/07/00

MEASUREMENT CABIN CL-1

Figures 16 to 18 : Settlements along horizontal profiles – Impounding phase – Multi-mechanisms.

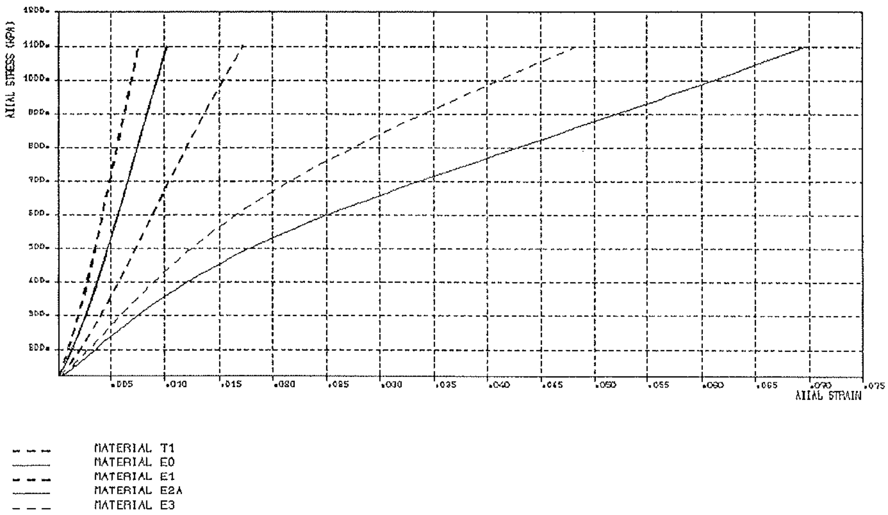


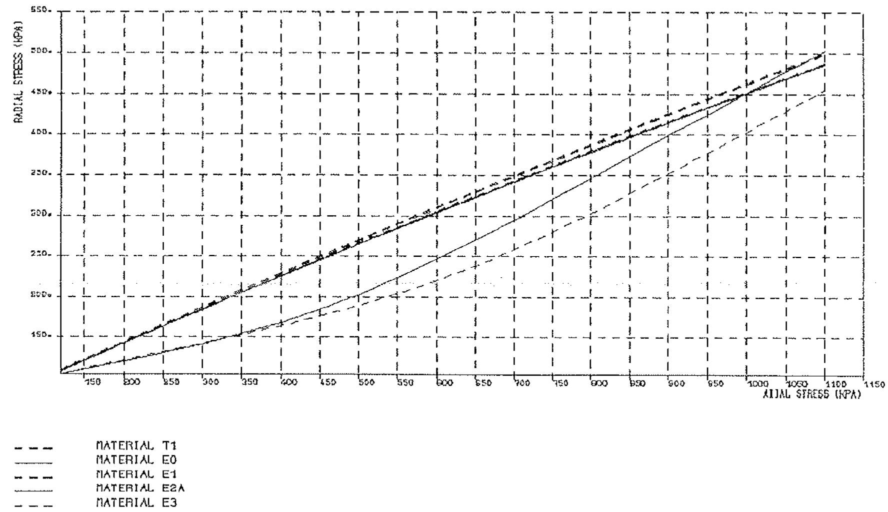
GEOMETRICAL SCALE : .00070 0 20
IARIABLE DEPLY ETAPE 8 TEMPS = 0.1300E+02



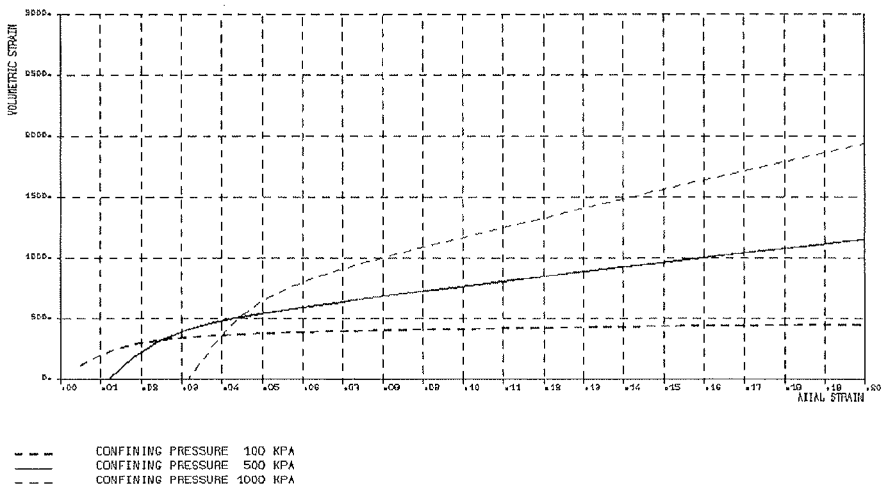
GEOMETRICAL SCALE : .00070 0 20
IARIABLE DEPLZ ETAPE 8 TEMPS = 0.1300E+02

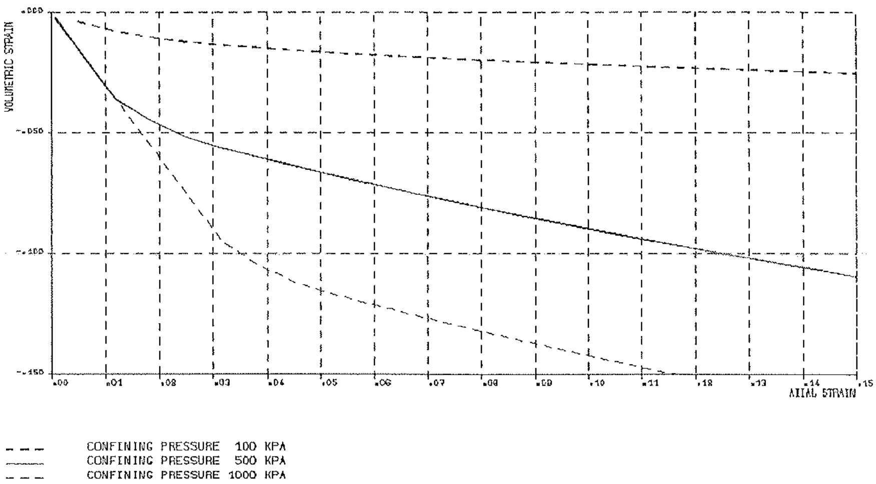
Figures 19 to 20 : Isovalues of displacements at the end of the impounding – Multi-mechanisms.





Figures 21 and 22 : Oedometric tests simulation with the multi-mechanisms model.





Figures 23 and 24 : Triaxial tests simulation with the multi-mechanisms model.

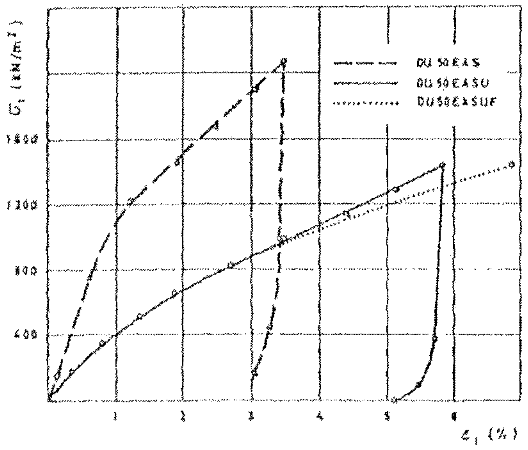


Fig. 2.29 — Ensaios de compressão unidimensional DU50. Relação tensão-deformação. Enrocamento alterado

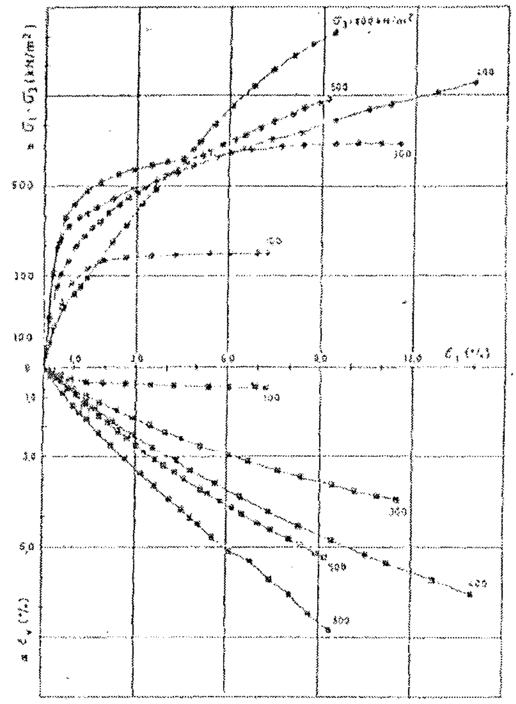


Fig. 2.52 — Ensaios triaxiais. Relação tensão-deformação. Enrocamento alterado molhado

Figures 25 and 26 : Oedometric and triaxial tests on a weathered rockfill (Veiga Pinto 1983)

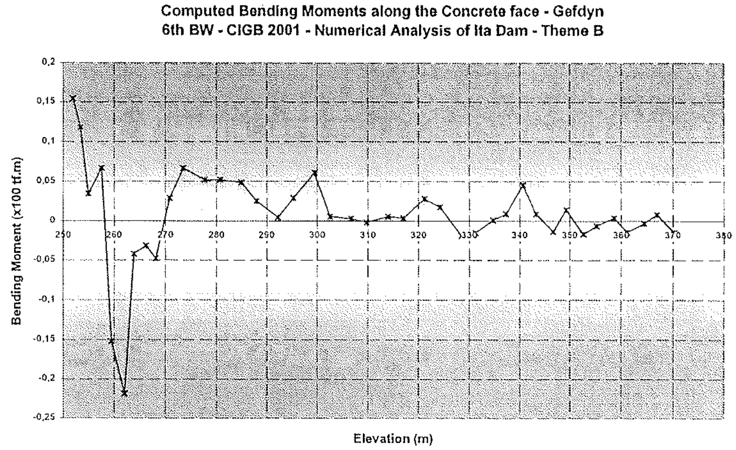


Figure 27 : Calculated bending moment along the concrete face at the end of the impounding

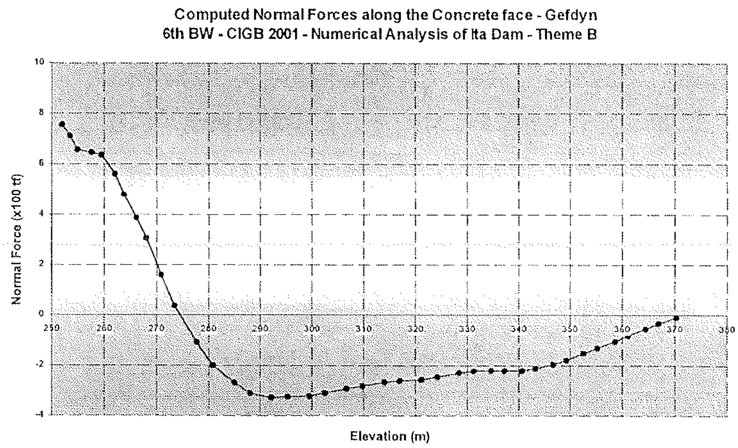


Figure 28 : Calculated normal force along the concrete face at the end of the impounding

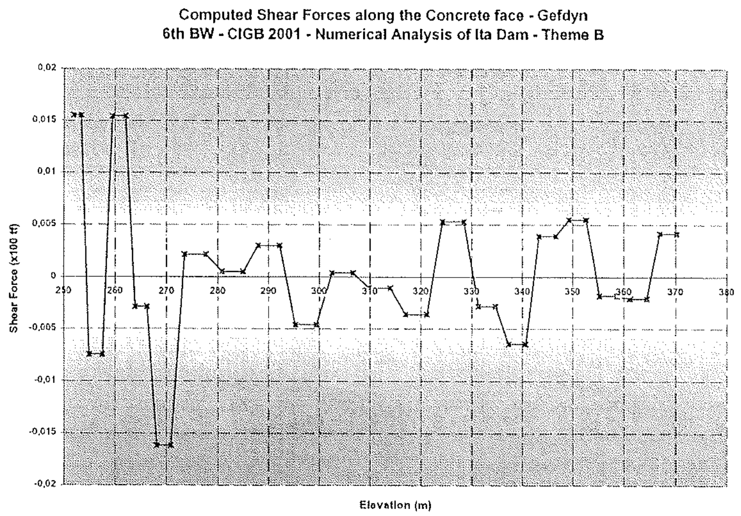


Figure 29 : Calculated shear force along the concrete face at the end of the impounding.