IRAPE DAM – STRESS AND STRAIN: NUMERICAL PREVISIONS AND MEASUREMENT RESULTS

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Abstract: The Irapé Hydroelectric Power Plant was built by CEMIG – Companhia Energética de Minas Gerais- on the Jequitinhonha river in the Minas Gerais State and its dam, with 208m, is the highest in Brazil. It is a central core rockfill type dam with 11.000.000 m³ fill volume and it is being operated. The construction was in charge of Consorcio Constructor Irapé formed by Constructoras Andrade Gutierrez, Norberto Odebrecht, Ivaí and Hochtief, and the project was carried out by Leme Engenharia and Intertechne. The dam was built in a narrow valley, with its low mid third formed by a closed canyon. To diminish the negative effects of this topography in the distribution of stresses in the core, was an important element to be considered for the definition of the zoning of the dam fill and for the selection of construction materials. An important deal of the instrumentation was based on the "Stress – Deformation" studies, carried out through bi and tridimensional modelling. This paper summarizes some peculiar solutions to the project and presents a comparison between the results anticipated in the numerical modelling and the values attained in the field.

Keywords: Central core rockfill dam of Irapé

1 Introduction

The results presented in this paper refer to the final stage of the dam construction, by the end of 2005. The reservoir filling began in December 2005. A typical section of the dam and the description of the construction materials are stated in Figure 1 and Table 1.

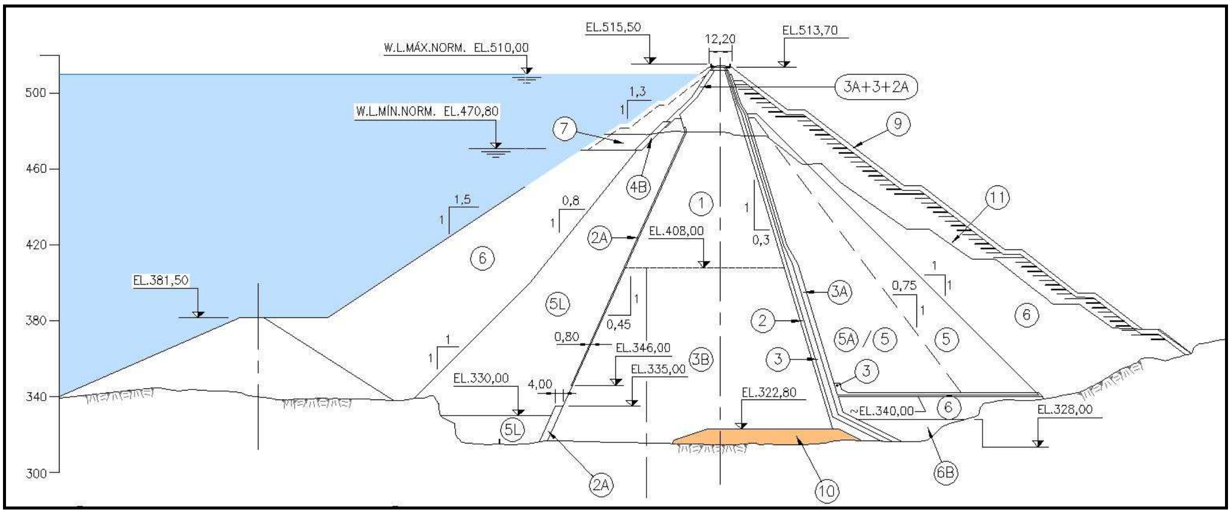

Figure 1. Typical Section of the Dam

Zones	Description	Layer thickness (cm)
1	Sandy Clayed Soils/Clayed sandy soils	20 a 25cm
2	Filter of natural sand	40
2A	Fine transition material	40
3	Fine transition	40
3A	Medium transition	40
3B	Clayey gravel	25 a 30cm
4	Coarse transition	40

4A	Coarse dumped transition	-
5	Rockfill with slight and medium decomposed rock	40
5A	Rockfill with medium and highly decomposed rock	40
5L	Random. Highly weathered rock and saprolite	40
6	Rockfill with slight to sound rock	80
6A	Dumped rockfill	-
6B	Rockfill of sound and slight decomposed rock	80
7	Rockfill protection	120
9	Rockfill of revetment	120
10	RCC Base	
11	Construction Joint	

Table 1. Construction Materials

The instruments considered in the project are piezometers, cells of pressure, cells of settlements hydraulics, electrical, magnetic and the swedish type, inclinometers, superficial topographic plates and flow meters.

Figures 2 from the construction period, show excavation of geometric shaping carried out in the core dam area, emphazising some geometric characteristics of the location that determined the dam project: the subvertical rock walls and the extremely narrow bottom of the valley. Figure 3 shows a panoramic view of the dam just concluded.

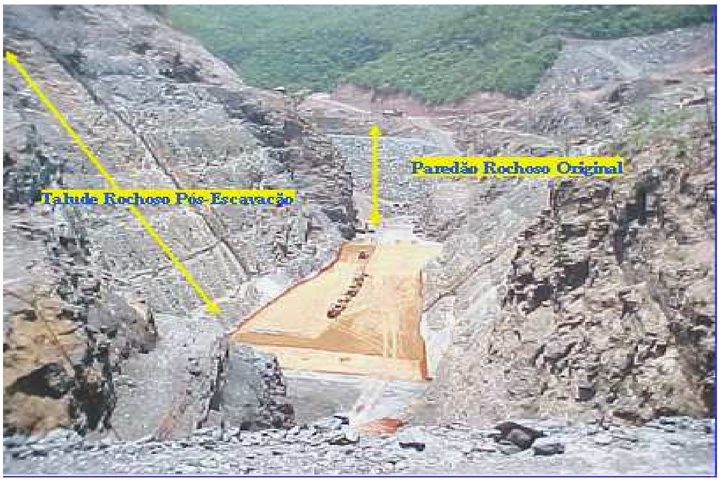

Figure 2. Core dam – Excavation for geometric improvements.

Figure 3. Panoramic of the completed dam

2 Relevant determining factors for the project and construction

Closed valleys with canyon geometry boost unfavorable inner stresses distributions in the rockfill dams cores, where bin effects already arise due just to a higher rigidity in the filter/transitions áreas.
Beyond this determining geometry in Irapé, the dam construction schedule was another challenge. It was established that the rockfill should rise aproximately 160m in only one year, including the wet season with an average duration of four months.
One of the directives of the project was, from the first studies stages, to adapt the earth core behavior to the construction needs, according to the availability of materials existing in the site, so as to guarantee its adequate performance considering the negative aspects of the river valley geometry.
A series of tests concluded in the adoption of a quite peculiar solution in dam engineering: “Build” or “fabricate” one part of the core earth material;
This decision was made considering the following facts:

- the need to use materials of lower compressibility to fill the inferior and closed portion of the valley (technical determining factor)
- The need to use more granular materials to improve productivity in the treatment of the layers and in the compaction process, besides the period to return to the constructión process after the wet season (determining factor)

The most common materials in the work site surroundings were clayed soils and deposits of sandy clayey materials and a limited fine gravel with fines. These gravel materials present a relatively low average of fine material, with a gap in the sand area. This fact determined a high permeability.
For this reason, in order to guarantee the provision of adequated materials, the constructor built and operated a soil plant, here the mixture of gravel with fines with the sand clayey material found in the region, replacing the absence of the sandy layer and improving the deformation and permeability of the produced material. With this procedure was possible to get enough quantities of this material to fill the narrowest region of the valley, about 40% of the dam height.
Figure 4 shows the installations used to obtain the most rigid materials: mixture of gravel with sandy clayey soils.

Figure 4. Soil Plant

3 Preliminary investigations on stresses and deformations and their evolution

The software used for the preliminary 3D analysis (with estimated parameters) was GEFDYN[1],[2], in its 6.3 version (University of Paris/EDF/Coyne et Bellier).

In the subsequent studies, bidimensional modellings were applied after consistencies analysis were completed, with parameters of tests or preliminary measurements in the site and in the inner geometry of the mass shell.

The construction of the dam was modelled in 8 stages with twenty three months total time for the shell laying. It is important to emphasize that the pace of the construction influences the development and dissipation of neutral pressures within the core (core consolidation). The analysis took into account a mechanic/hydraulics coupling in the different stages of the construction. No creep effect was modelled.

The influence of the incorporation of one layer of more plastic material was considered in the joints of the core with the foundation walls, based on the concept of “lubricant” material in the shell-brace walls, shell-foundation (as, for example, in the Mexican dam, Chicoasén[3],[4]. The purpose was – precisely- “to plasticize” and redistribute the stresses in the inferior parts of the shell, by reducing the relief in that área (A Coulomb type sliding interface was used in the modelling).

The initially assumed “convencional” parameters of the materials, were:

Material	**Module of Deformability E (MPa)**		**Poisson Coefficient**	**Specific Humid Weight (kN/m3)**	**Porosity**	**Saturation (%)**	**Specific solid Weight of grains (kN/m3)**	**Permeability (m/s)**
	Min	**Max**						
Clay sandy soils-Borrow Area 1	20	50	0,30	20,0	0,42	95,0	27,5	1 x 10^{-7}
Sandy clayey soils-Borrow Area 2	50	70	0,30	20,0	0,42	95,0		
Lubrification Layer	15	15	0,40	18,5	0,42	95,0		
Clayey Gravel	80	120	0,30	21,0	0,23	-	27,0	5 x 10^{-7}
Filters/Trans.	100	120	0,30	19,0	-	-	-	Free drain condition
5	35	65	0,25	20,5				
6	60	80	0,25	21,0				

Table 3. Geotechnical Parameters

The results of tridimensional modellings were evaluated with a higher qualitative approach, to indicate the critical áreas. Thus, it can be asserted that:

- Important stresses reliefs were noted in the following core áreas: the lower área of the core side form brace; in the core área located in the canyon, downstream; in both abutments highs, in the dam crest área.
- As for settlement, the maximum value from calculations would be 1,2m in the core.

The reliefs are increased when an elastic linear model is used. The more adequate elastic-plastic models reduce such effects, even though the results supported the usual practice of shaping corners in the foundation topography to reduce concentrated bendings.

Displacements in the abutments área were visibly faced to the valley bottom and turned into local lengthwise stressess in the dam crest área. This behavior is essentially bidimensional and showed the need to use a more plastic material in the upper part of the core, specially in the abutments áreas, in a stripe of about 10,0 m wide, to provide better conditions to face this mechanic cinematic condition.

The results showed that the use of the lubricant layer in the narrow canyon region área was adequate to reduce bendings. In the model, then reasserted in the project, a 3 m width was simulated, and it turned out to be enough for the anticipated behavior, where an advantage of about 20% in the stresses transmitted to the lower portions of the dam core was attained. In the prototype, it was decided to use a quite argilaceous, wetter material and limited compaction (CGméd=98% PN, CGmín=95% and hot ≤h≤ hot+2%)

3.1 Qualitative validation of the bi dimensional model

Together with the tri dimensional modelling, simpler, current and faster bi dimensional modellings were carried out. For these, GEFDIN as well as SIGMA softwares from Geo-slope Canada were used. The 2D results showed here only refer to the SIGMA software, in final construction phase.
Comparative studies of the 3D and 2D modelling results, and with a little abridged and rough approach, it is possible to assert that bi dimensional modellings, with restrictions, are "qualitatively valid" since, in a geometrically similar case, the following representation was adopted. (Figure 5)

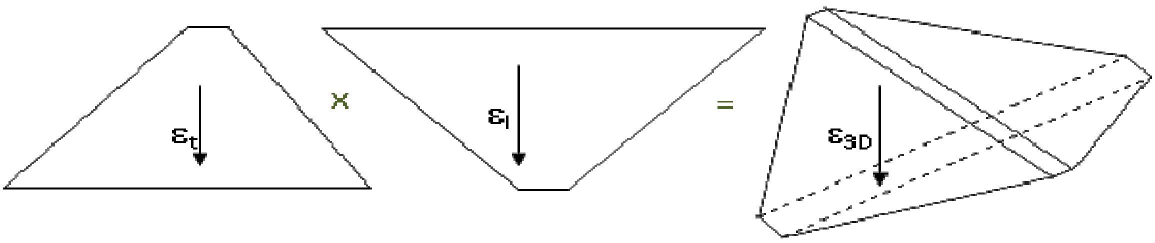

Figure 5. Illustrative representation of superpositions effects

That is to say (εt; $\Delta\sigma t$) x (εl; $\Delta\sigma l$) = $\varepsilon 3D$; $\Delta\sigma 3D$, the real stressess (or deformations) relief can be tested through the superposition of the bi dimensional effects attained from the cross and lengthwise or longitudinal sections analysis. Later on, the efficiency of this simplification will be checked.

4 Instruments measurements

4.1 Settlements

From the records of the instruments located in the core - filters/transitions interface área were observed that the settlements have a reasonable uniformity, with no indication of marked diferential coefficients. In the core region, where processed material resulting from the mixture of natural and sandy gravel was used, under EL. 408,00, the settlements were lightly inferior to those of the adjacent materials, over the mentioned level. Here, using sandy clay soils, the behavior was the opposite. However, the differences were of a low intensity.
The maximum settlement was observed in the RM-307 magnetic sensor, locadet in the axis of the dam, elevation 442,60 , with 1,91m or approximately 1% of the dam height. This value exceeded the estimates of the tridemensional numeric modelling which anticipated a maximum of about 0.7% or ~1,2m; it turned out to be quite close to the bi dimensional modelling estimates –a maximum of 1,85 m.
To monitor the performance of the dam construction joint in the downstream face shell, of about 100 m height, settlements cells (Swedish boxes) were placed before and after the joint plane. All the instruments installed to monitor the behavior of the shell in the joint plane, showed settlements compatible between those installed both up and downstream the joint. This fact indicates that the adopted construction treatment and details resulted in an adequate combination of materials in the surrounding area

4.2 Stresses

4.2.1 Maior Principal Stresses

In the next Figures 6 and 7 are shown the results of the total pressure cells located in the longitudinal section along the axis of the dam and in the maximum height cross section, where the vertical stresses are represented with bars.

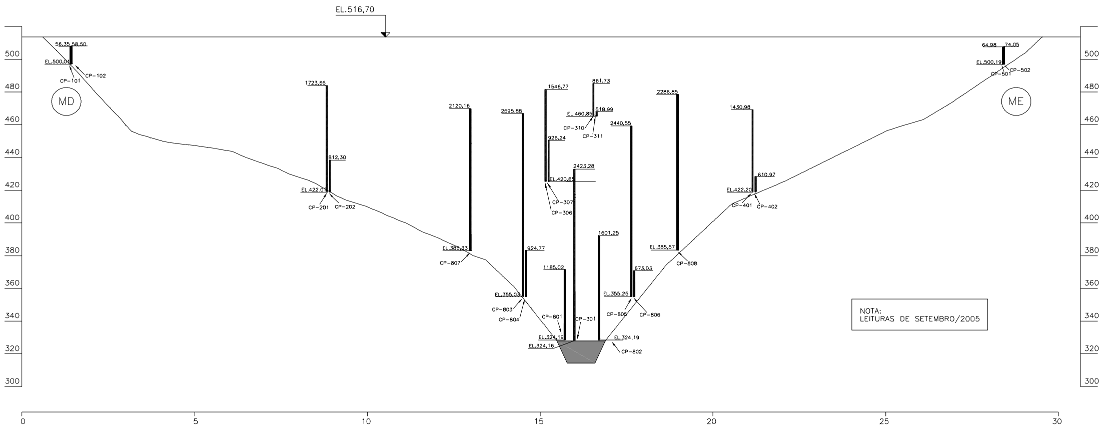

Figure 6. Maior Principal stress

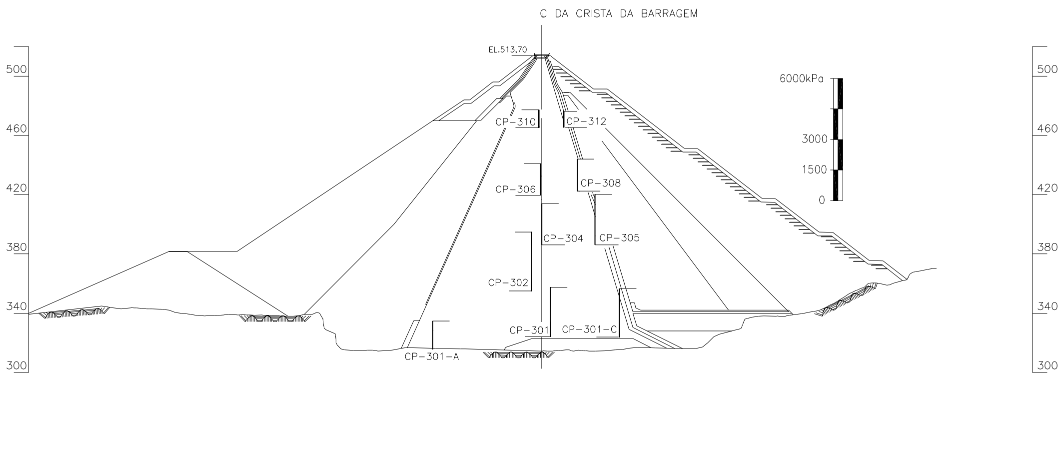

Figure 7. Maximum height cross section – Maior Principal stresses.

A comparative between measured stressess values and the theoretical ones estimated through element finites analysis are shown in Table 4.

Instrument	Section	Stresses	Installation Level	Estimated stresses through the 2D analysis (kPa)	Stresses measured in the instruments (kPa)	Relationship measured/ estimated
CP- 301	Transversal 3-3	Vertical	324,16	3300	2413	0,75
CP- 301-A		Vertical	315,60	3050	**1378**	**0,45**
CP- 301-C		Vertical	324,01	3400	**2371**	**0,70**
CP-302		Vertical	355,03	2800	2887	~1
CP-304		Vertical	385,72	2000	2018	~1
CP-305		Vertical	385,76	2150	2477	~1,15
CP-306		Vertical	420,86	1300	1552	~1,20
CP-308		Vertical	421,68	1400	1568	~1,10
CP-310		Vertical	460,85	700	871	~1,20
CP-312		Vertical	460,48	800	787	~1
CP-801	Longitudinal	Vertical	324,19	2000	**1184**	**0,6**
CP-802		Vertical	324,17	2100	1598	0,8
CP-803		Vertical	355,03	2650	2595	~1
CP-805		Vertical	355,25	2700	2444	~0,90
CP-807		Vertical	386,33	2600	2125	0,8
CP-808		Vertical	385,57	2600	2294	~0,90

Table 4. Total pressure cells- Relationship between measurements and estimates parameters

An analysis of the oddest values is presented next, that is to say, those values in which the relation is too far or fairly far from the unit value (CP's 301-A, 301-C and 801).

The most important discrepancy was shown by cell CP-301-A. A comparison with the estimation of the tridimensional model[5] is shown in Figure 8, next (it should be noted, therefore, that in Figure 8, the principal major stresses are represented, a bit different from vertical stress).

The estimated stress would be of about 1500 and 2000 kPa, in 3D modelling. In 2D modelling it was 3.500 kPa. The prototype stress measure was of about 1400 kPa. The 3D model result was close to that of the sensors. A criteria of superposition of results from plain cross and longitudinal analysis suggested in item 3, should be now noted.

Figure 9, that follows, shows the distribution of Principal major stresses resulting from plain[6] analysis lengthwise or longitudinal section. Figure 10, correspondingly, in the cross section.

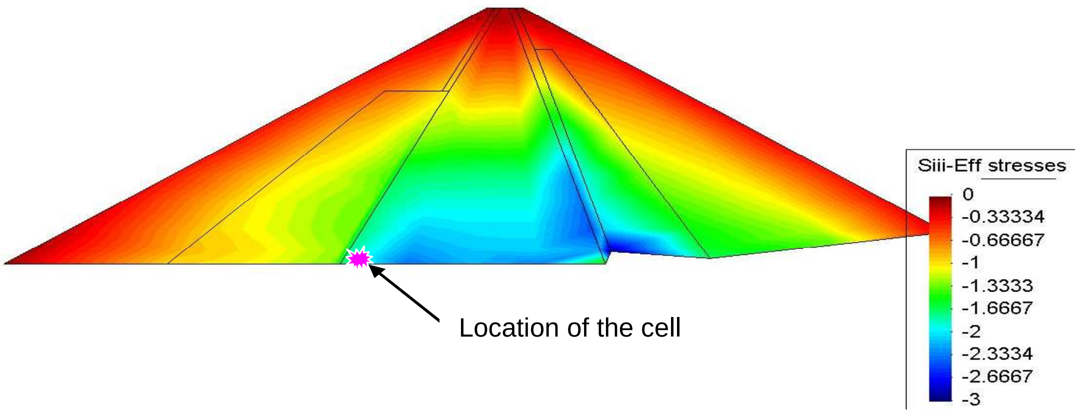

Figure 8. Maximum height cross section – Maior Principal stress

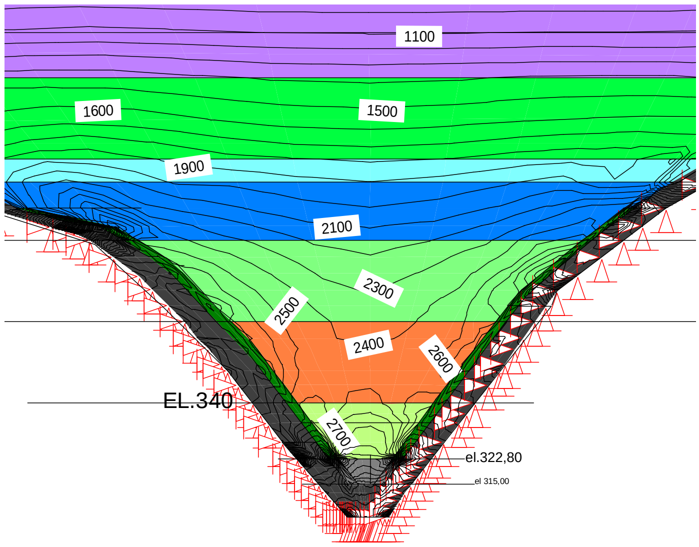

Figure 9. Longitudinal Section – Maior Principal stresses

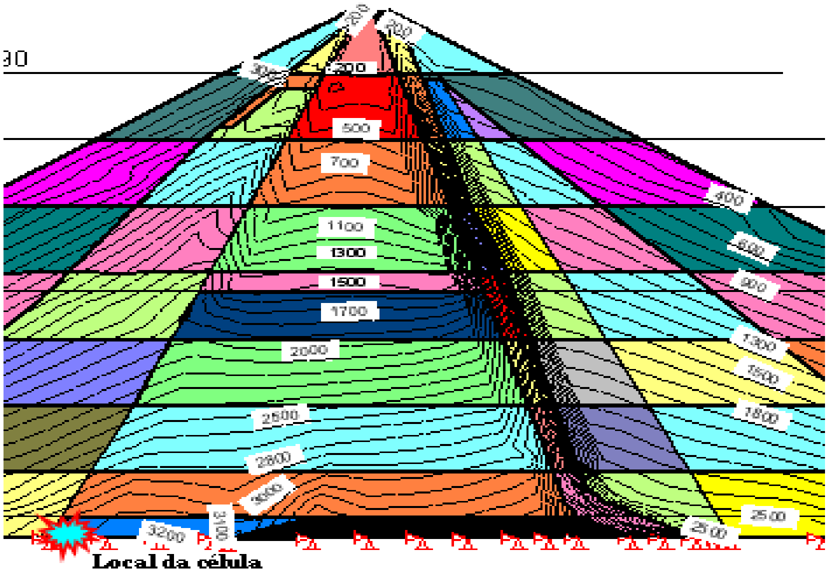

Figure 10. Maximum height cross section – Maior Principal stress.

To assess the efficiency of the procedures suggested to reach the superposition of bi dimensional effects, let us suppose that the estimated stress over the instrument is of about 3500 kPa.
Then

- Cross section: stresses relationship = 3000/3500 = 0,85
- Longitudinal section: stresses relationship=2600/3500 = 0,74
- σ3D = 0,85 x 0,74 = 0,6 σteorical (γ.H) or 0,6 x 3500 = ~2.000 kPa

In the case of cell 301-C the measured value was 2.400 kPa (this value actually obtained in the tri dimensional model). In the cross section 2D analysis, a value of about 3,400 kPa was set (0,95 of the "theoretical" stress) and aproximately 2,600 kPa (0,75) of the "theoretical" in the longitudinal section. Thus, through simplifications, the resulting stress of 2.500 kPa was obtained, that is to say, equivalent to the measured value and also to the 3D model value.
In the case of CP-801, the proportion noted for the other sensors is also observed.
In the next figure is clearly ilustrated the above mentioned relation between the values obtained in the 2D and 3D model. The product of the two major principal stresses in the axis of the dam figured out in the cross and longitudinal section of the 2D model expressed as percentagem of the teoretical one is equal to the 3D result in the same point.

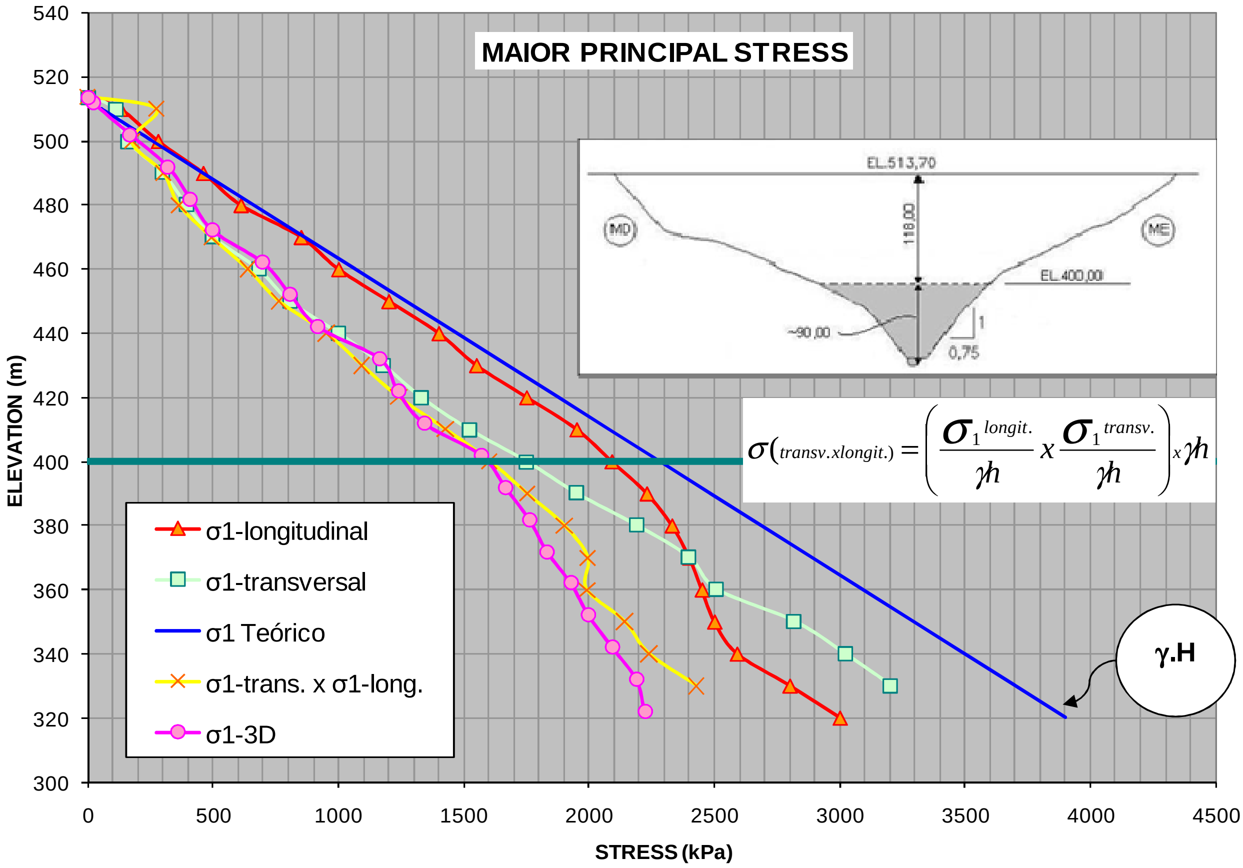

Figure 11. Major principal stress .The relation between 2D and 3D analysis..

4.2.2 Minor Principal Stresses

Figure 12 shows the distribution of minor stresses obtained from 3D[5].

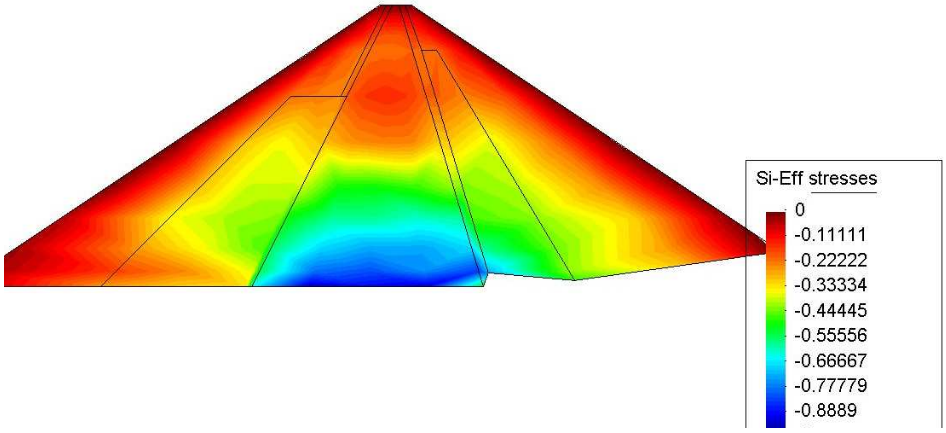

Figure 12. Maximum height cross section – Minor Principal stresses.

The CP – 301-B sensor that measures horizontal stress, pair-cell CP – 301-A, registered a horizontal stress of approximately 600 kPa by the end of the construction. The 3D modelling result, expressed above, were quite close to this value. The value in the 2D modellings was 1.100 kPa, but using the previous relation of 0,6, the measured value is also closely reached.

The CP-301-D, pair cell from 301-C showed, in the prototype, a little more favourable situation than the one anticipated in the 3D modelling. The results in the plain model are not different from these.

Table 5[6], that follows, mainly emphasizes the relationship between horizontal and vertical stresses, measured by the cell pairs. The mean value measured in the dam shell was of about σ_H / σv = 0,45.

Instrumento	Seção	Posição da Tensão Medida	Cota de Instalação	Cota do Aterro	Relação entre tensões σh/σv
CP - 101	1-1	Vertical	500,21	513,700	
CP - 102		Vertical	500,01	513,700	
CP-201	2-2	Vertical	422,03	514,100	0,47
CP-202		Horizontal	422,01	514,100	
CP- 301	3-3	Vertical	324,16	515,240	
CP- 301-A		Vertical	315,60	471,450	0,41
CP- 301-B		Horizontal	315,60	471,450	
CP- 301-C		Vertical	324,01	488,510	0,54
CP-301-D		Horizontal	324,04	488,510	
CP-302		Vertical	355,03	515,240	0,38
CP-303		Horizontal	355,03	515,240	
CP-304		Vertical	385,72	515,240	
CP-305		Vertical	385,76	495,980	
CP-306		Vertical	420,86	515,240	0,60
CP-307		Horizontal	420,85	515,240	
CP-308		Vertical	421,68	503,790	0,51
CP-309		Horizontal	421,65	503,790	
CP-310		Vertical	460,85	515,240	0,60
CP-311		Horizontal	460,67	515,240	
CP-312		Vertical	460,48	507,580	0,58
CP-313		Horizontal	460,30	507,580	
CP-401	4-4	Vertical	422,63	514,080	0,43
CP-402		Horizontal	422,61	514,080	
CP - 501	5-5	Vertical	500,19	513,680	
CP - 502		Vertical	500,16	513,680	
CP-801	CL da Crista da Barragem	Vertical	324,19	508,150	
CP-802		Vertical	324,17	511,950	
CP-803		Vertical	355,03	500,000	0,36
CP-804		Horizontal	355,03	500,000	
CP-805		Vertical	355,25	494,000	0,28
CP-806		Horizontal	355,25	494,000	
CP-807		Vertical	386,33	514,090	
CP-808		Vertical	385,57	514,19	

Table 5. Relationship between Horizontal and Vertical Stresse Measures

Conclusions

The presented data and tests carried out, lead, in short, to the following main conclusions

- The results of tridimensional mathematical modelling (with GEFDYN software) and its constituent models led to results that turned out to be pretty coherent with the prototype measurements.
- The application of concepts analysed in the modelling, and their application in the prototype, together with the continuous improvement of construction procedures stimulated by the construction staff, were fundamental to the achievement of this degree of coherence between modelling and prototype. It is important to emphasize that this situation is difficult to state in literature on the subject.
- Among the applied concepts, it is possible to note the distribution of materials according to their rigidity; the use of a "lubricant" clay layer in the core-rock wall joint; the geometrical excavations /shapings on the canyon rock walls; the permanent search, by the work-group at the site and its engineering, to maximize for the compatibility of the materials deformabilities, including the use of materials which had been discarded at the beginning of the works.
- the tridimensional modellings with stage construction evaluation are not an usual practice in such projects, but the results showed that in similar cases, especially with the unfavourable geometry of the valley, such analysis are fundamental.
- such tridimensional modellings, more expensive, complex and long lasting, are absolutely justified. Even more, the permanent search for the constitutive models and adequate parameter analysis is an objective that should be considered by academic and technical circles.
- The simulation presented as to "validate" bidimensional analysis of results through the superposition of perpendicular or "ortogonal" effects turned out to be adequate in that case. However, it could not be generalised, and be a useful and acceptable procedure during the first stages of the Project, as an agile instrument of solutions comparison, for instance, or even more, in more advanced stages of the Project, in case of a consistency analysis properly tested.

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