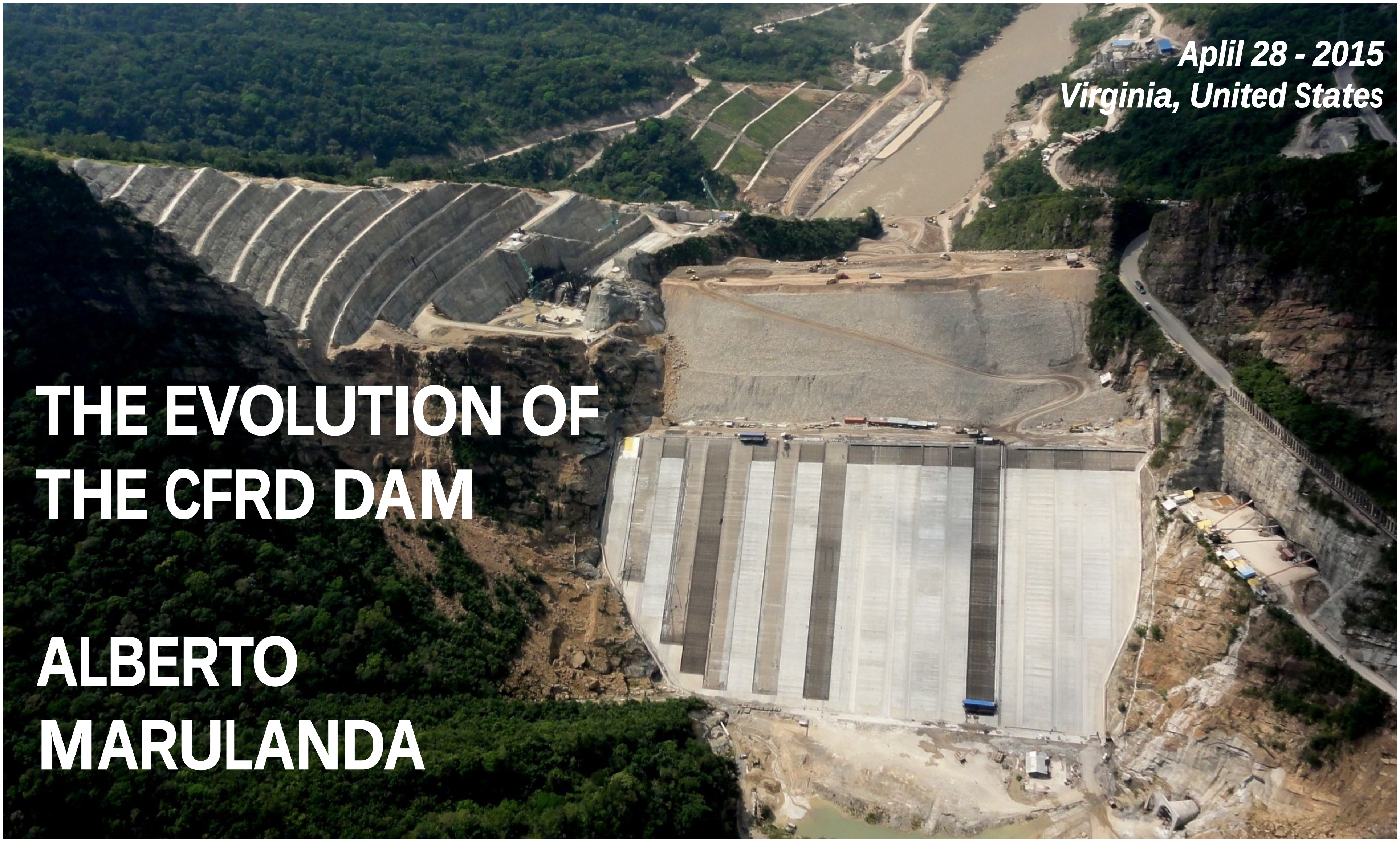


Aplil 28 - 2015
Virginia, United States

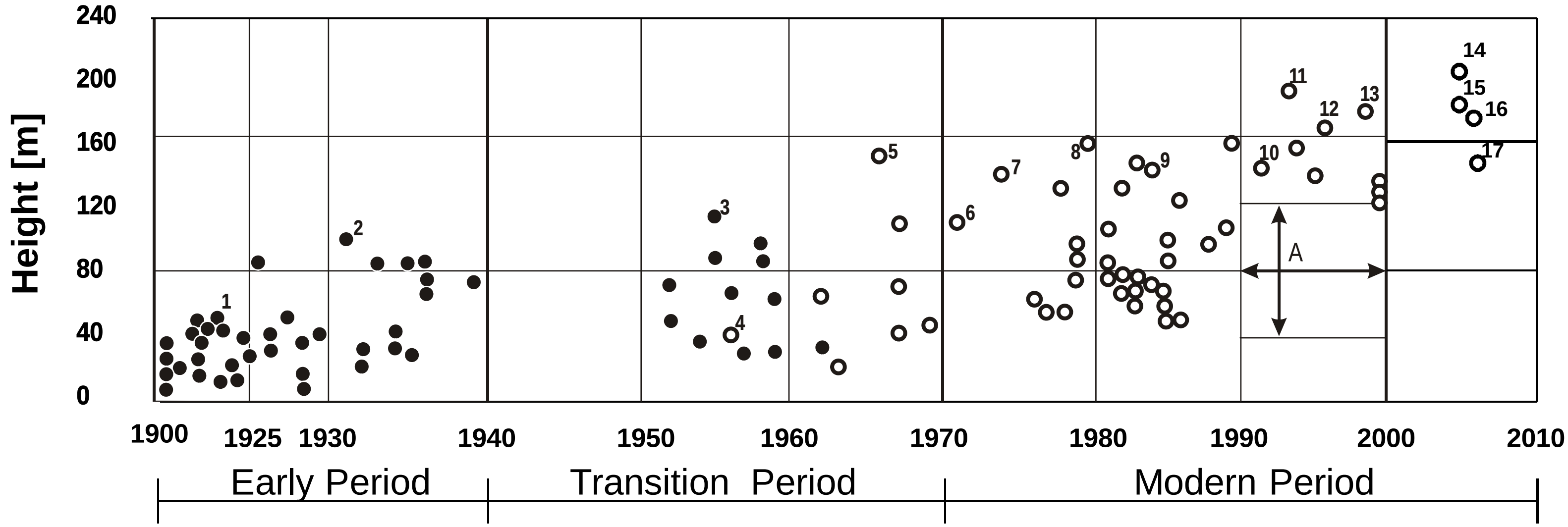
THE EVOLUTION OF THE CFRD DAM

ALBERTO MARULANDA

CIGB
ICOLD

INGETEC
INGENIEROS CONSULTORES

TREND IN HEIGHT OF CFRD DAMS WITH TIME

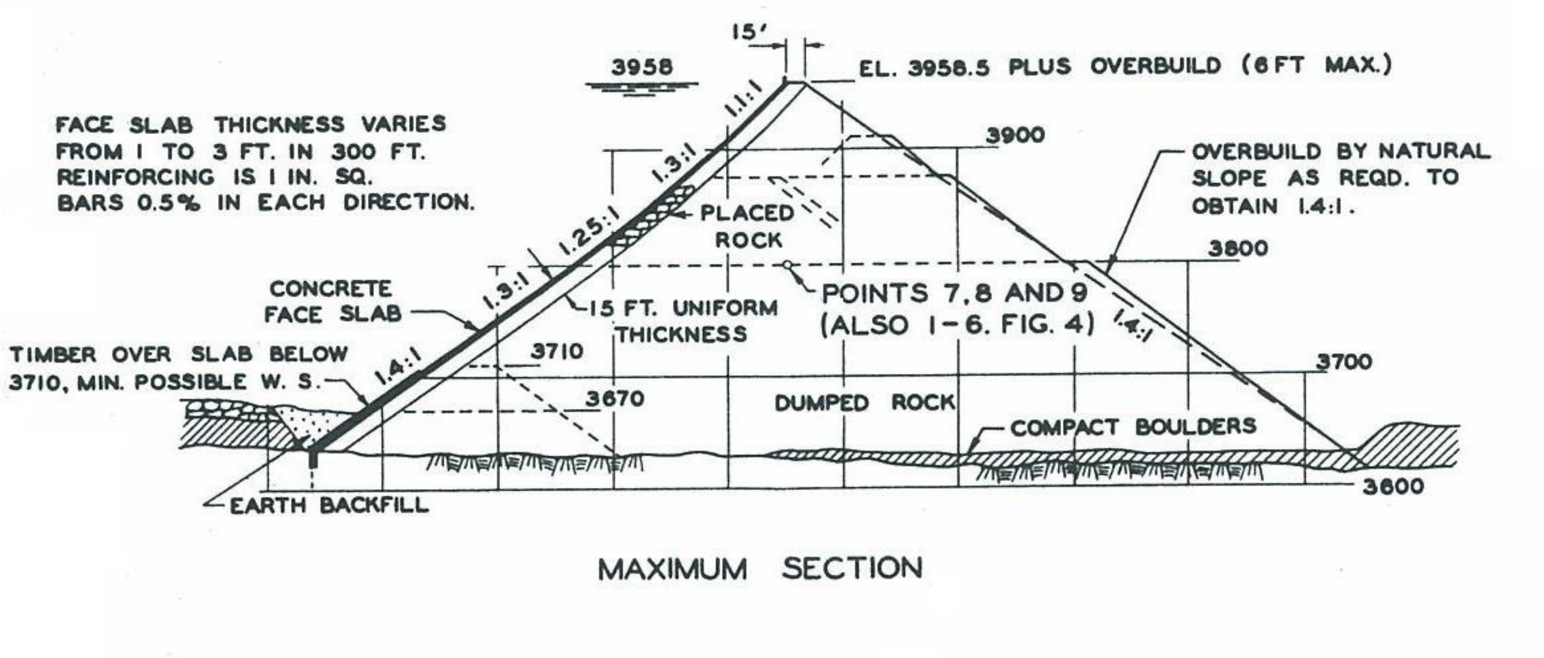


(Cooke, 1997, extended to 2006)

•	*Dumped Rockfill*	o	*Compacted Rockfill*
1	Strawberry Creek	2	Salt Springs
3	Paradela	4	Quioch
5	New Exchequer	6	Cethana
7	Anchicaya	8	Areia
9	Khao Laem	10	Segredo
11	Aguamilpa	12	Yacambu
13	Tianshenqiao	14	Campos Novos
15	Barra Grande	16	Cajon
17	Mohale		

A 68 CFRDs completed between 1990 and 2006, height 40 to 120 m

SALT SPRINGS DAM



For this dam, that was the tallest in the world for 23 years, vertical joints with openings of one inch were included and horizontal joints were built as construction joints.

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SALT SPRINGS DAM

Eight of the central vertical joints closed completely, and in three the concrete failed by crushing. Some of the horizontal joints also experienced concrete crushing. At the moment, it was believed that the major source of leakages were the cracks near the union with the abutment, where no perimeter joint was built. The cracks in the central compression zone have similarities with the ones observed in the recent incidents of the Brazilian dams of Barra Grande and Campos Novos and in the Mohale dam in Lesotho were sever cracking occurred during the first filling of the reservoir. Leakage in Salt Springs reached 450 l/s. The empirical solution adopted after Salt Springs consisted in increasing the number of joints and in introducing compressible materials, as done in Bear Creek [1].

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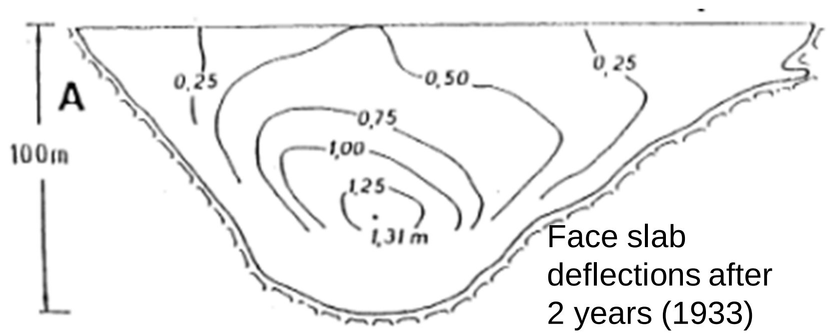
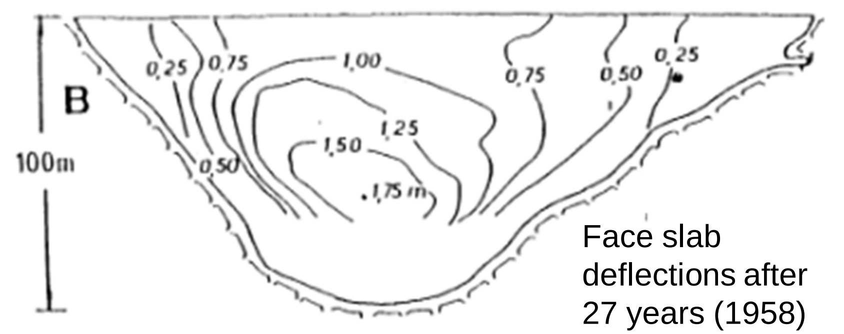
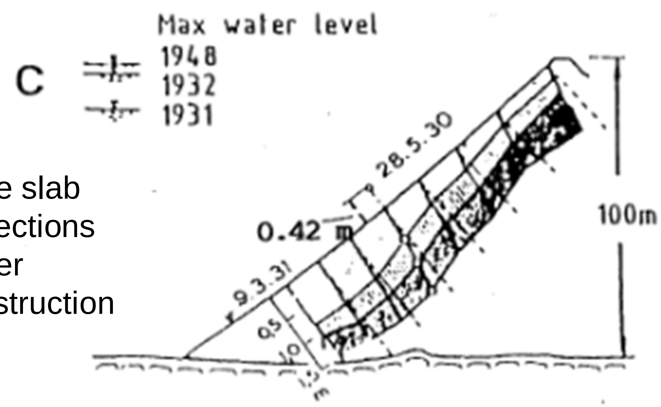
SALT SPRINGS DAM



Face slab deflections after 2 years (1933)



Face slab deflections after 27 years (1958)



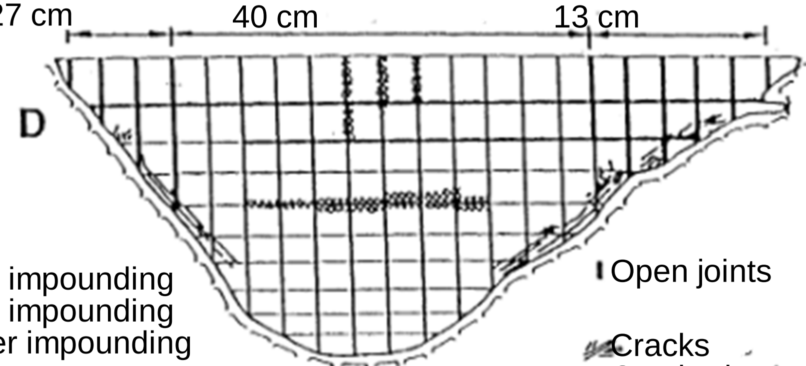
Face slab deflections under construction

1931, 1 yr after impounding
1932, 2 yr after impounding
1957, 27 yr after impounding

Joint openning 27 cm

Joint closure 40 cm

Joint openning 13 cm



Open joints

Cracks
Crushed concrete

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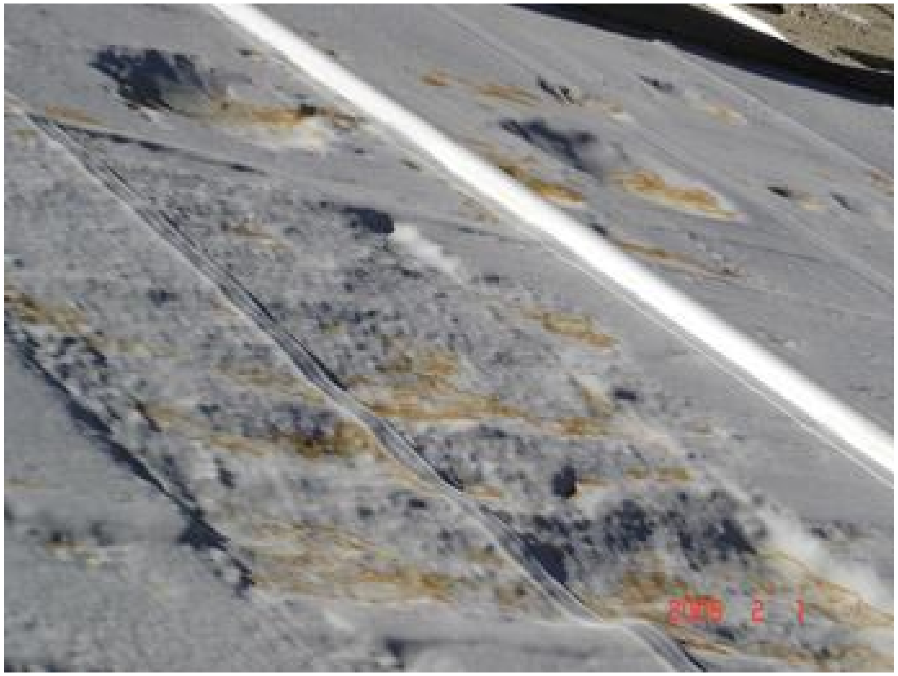
SALT SPRINGS DAM



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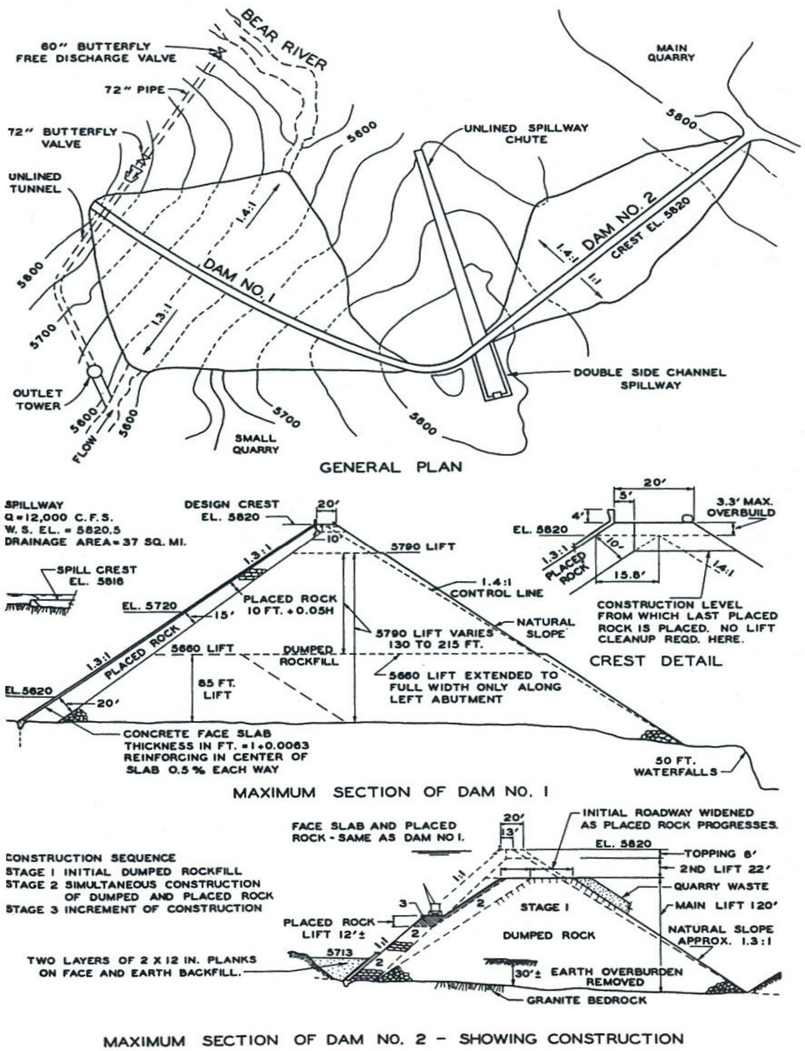
BEAR CREEK DAM



GENERAL PLAN AND SECTIONS – LOWER BEAR RIVER DAMS

The empirical solution adopted after Salt Springs consisted in increasing the number of joints and in introducing compressible materials, as done in Bear Creek. In addition, a perimeter joint and a hinges Slab parallel to the canyon were included to decrease the demands on the slab near the abutments. In the Bear Creek dam, leakage was reduced considerably (112 l/s) and also the cracking was limited to some cases were superficial spalling occurred in the walls of the joint.

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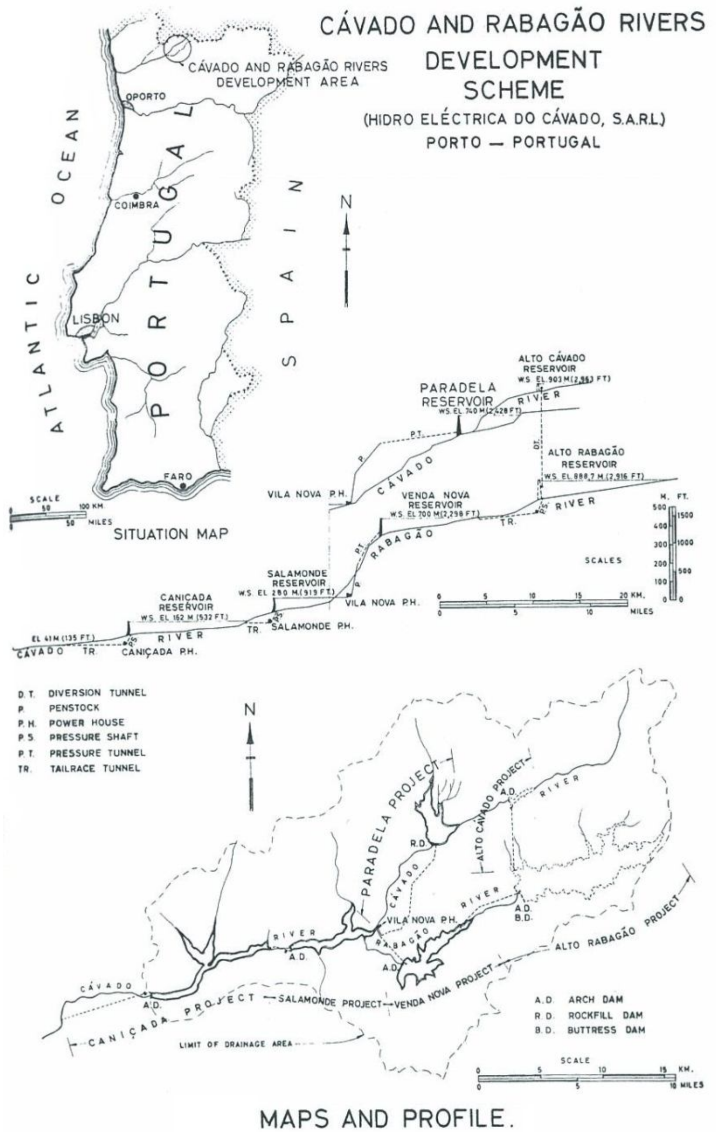
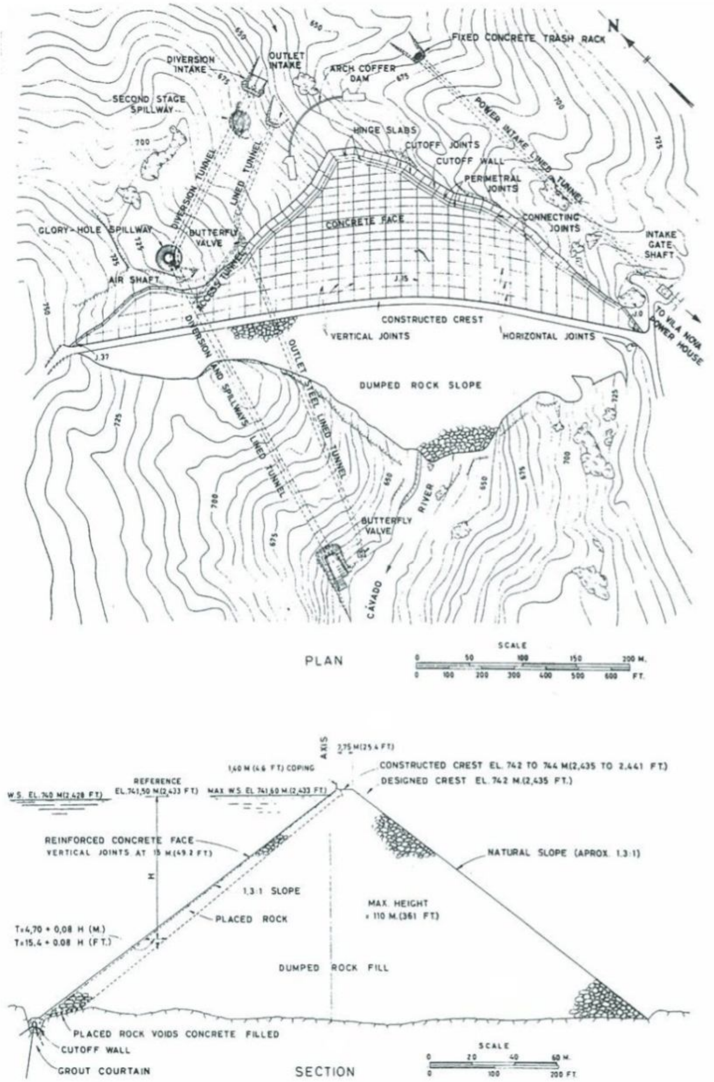
PARADELA DAM



The Paradela Dam (120 m) is the tallest structure built with the technology known until the 1950s. In reference [2] the behavior of the dam and the concepts considered for its design are presented. The design was absolutely empirical and based on additional elements that included the creation of new perimeter joints (two perimeter slabs were built, like the ones used in Bear Dam), opening of vertical joints with compressible elements (7 cm) and of 3 cm in the horizontal joints.

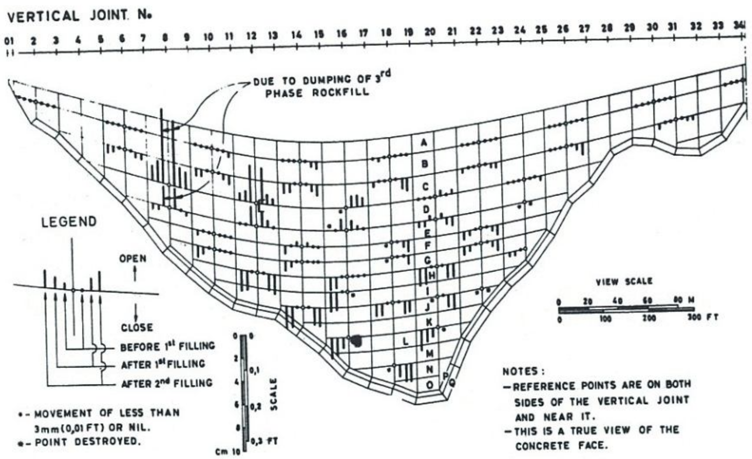
PARADELA DAM



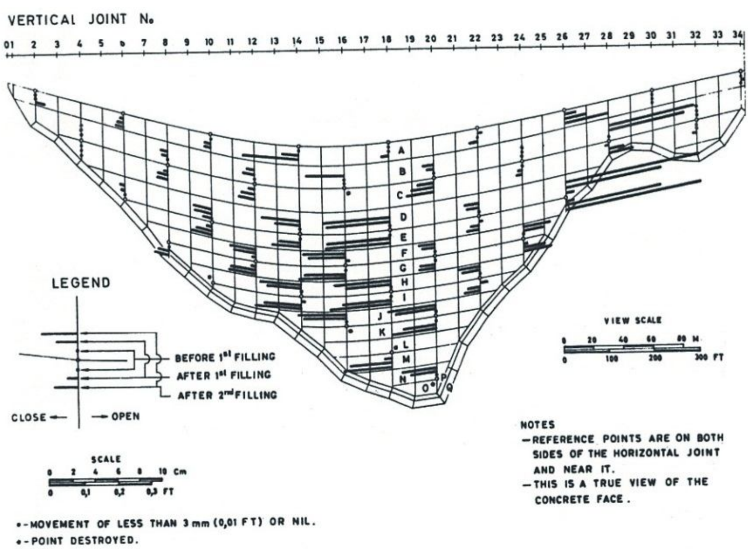
PARADELA DAM. PLAN AND SECTION.

During the repair after the first reservoir impounding, joint openings were further increased by cutting the border of the joints to widen the space for movements trying to relief the compression stresses in the concrete face. The materials used in the fillings were so compressible that, even before the reservoir impounding, it was observed that the joints between the slabs were closing as a result of the deformations in the filling by their own weight. With the reservoir load the deformations reached a value of 2.04 m in the direction normal to the face.

PARADELA DAM



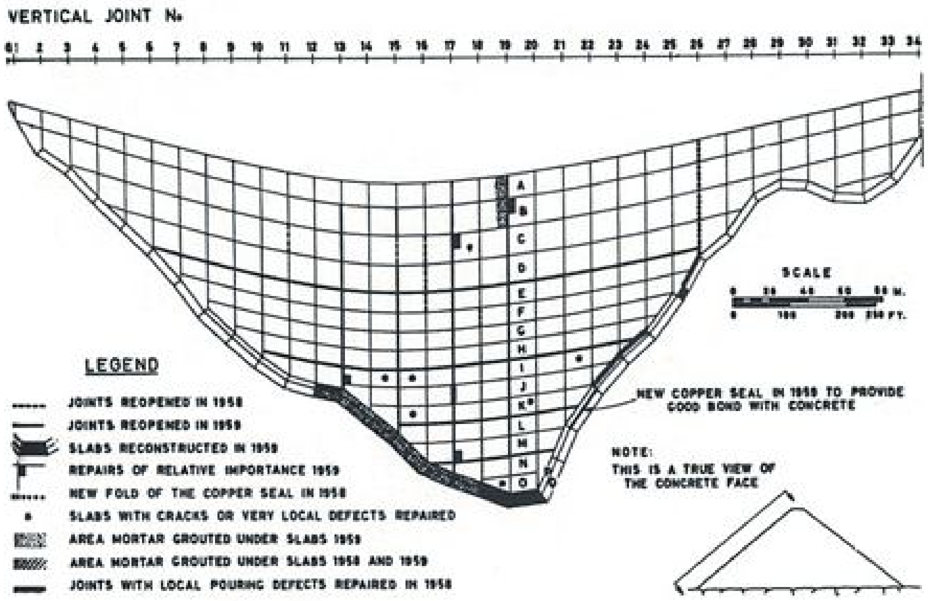
a)-OPENING AND CLOSING OF HORIZONTAL JOINTS



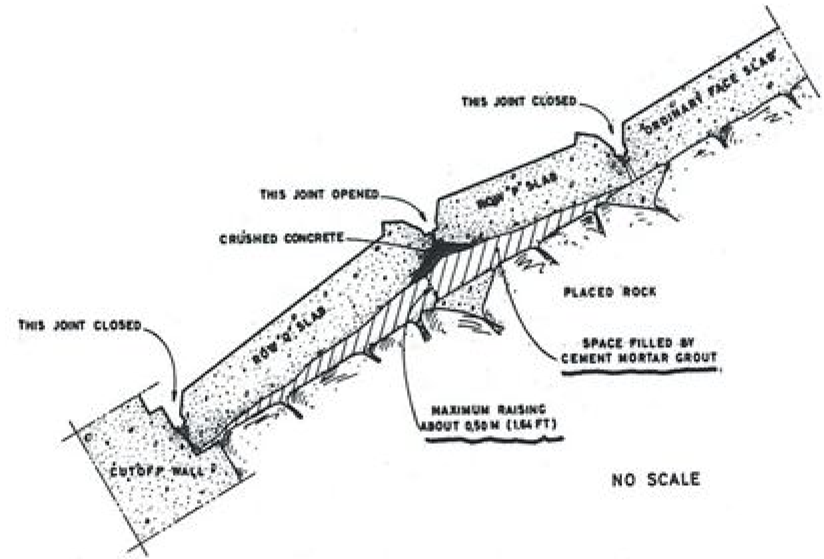
b)-OPENING AND CLOSING OF VERTICAL JOINTS

Excessive stresses generated by movements in the first and second impounding produced some cracks in the borders of the slabs. However, the largest cracks were reported in the perimeter slabs. The leakage reported in the first years of operation exceeded 3 m^3/s; confirming the capacity of the fill to manage high levels of leakage without the risk of failure of the structure.

PARADELA DAM



MAIN REPAIRS AND JOINTS REOPENEI



MOVEMENT OF PERIMETRAL SLABS

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LESSONS LEARNED FROM THE FIRST GENERATION OF CFRDS

- The relevant lessons from the first generation of these dams referred to the effects that the excessive movements of the fill generated on the face slabs. The deformations were associated with the low deformation modules that were obtained from the placement process. The experience in these dams seemed to indicate that in the evolutionary process of trying to eliminate the cracks, by compression, with the creation of more deformable joints, the cracks were reduced but the leakage increased as a result of the greater number of joints and the opening of those that did not closed. This is a very relevant assessment for modern dams that will be further discuss later in this paper.

LESSONS LEARNED FROM THE FIRST GENERATION OF CFRDS

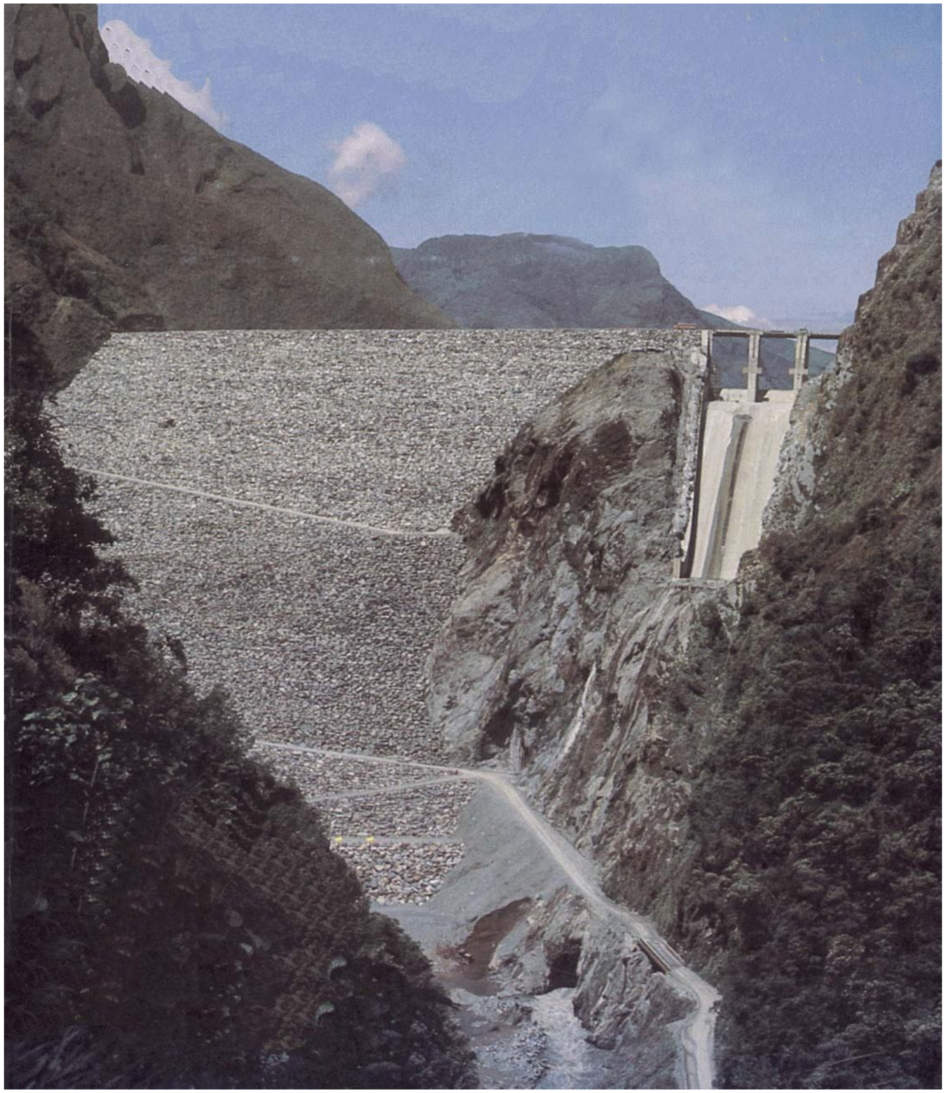
- After the construction of the Foz de Areia Dam the construction of several concrete face dams with heights below 130 m and rockfills basically composed of basalts started in Brazil. The process of placement and compaction was relatively homogenous in these dams with materials lifts in the dam of 1 m in the upstream shell and 2 m for downstream shell. All of these dams were considered successful experiences, even though, leakage between 300 and 500 l/s were recorded. Some of the dams, like in the Xingo case, experienced cracking in specific places that illustrated the relevance of precluding sudden changes in the deformation pattern of the slab, avoiding hard points behind the face (Marulanda and Pinto).

GENERAL TRENDS OF RECENT HIGH DAMS

- Between 1975 and 1990, central core rockfill dams of great heights and in very narrow canyons were built in Mexico (Chicoasen dam,260m) and in Colombia (Chivor 238m and Guavio 248m dam).

GUAVIO DAM (248m)

CHIVOR DAM(238M)



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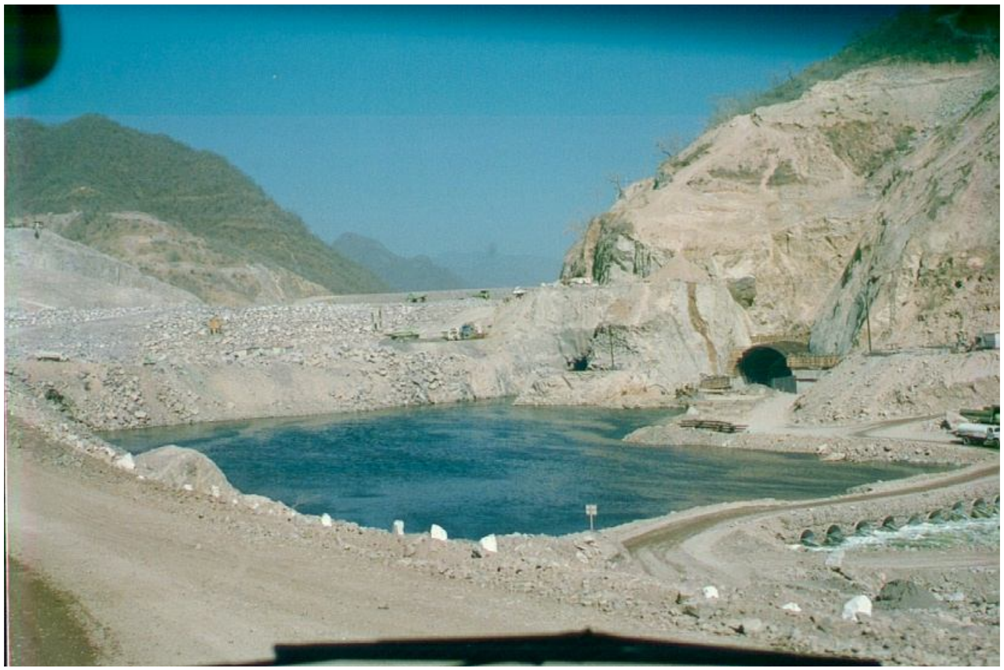
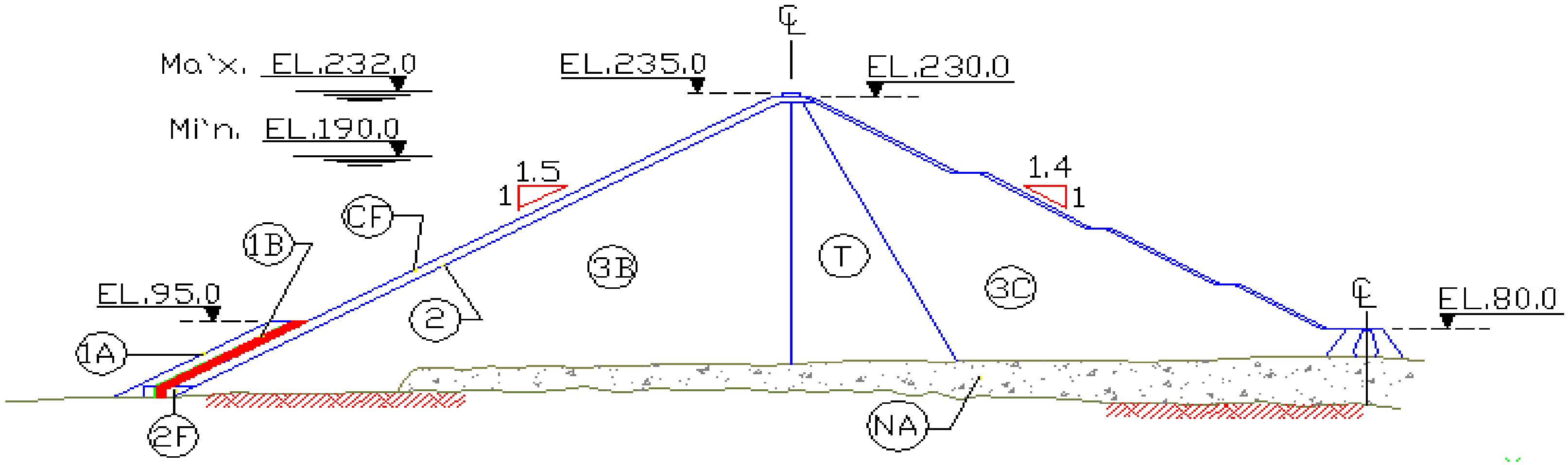
- The behavior of these Colombian and Mexican dams demonstrated the fundamental concepts of the rockfill behavior postulated since the 70's by Marsal [3]. The strength of a rockfill depends on the hardness of its particles, but its compressibility depends much more on gradation and compaction process. Materials with particles of less strength could have less compressibility if they are properly gradated and compacted. The process of grain breakage substantially decreases by adding water during compaction and by the presence of good gradation where the finest materials fill the gaps between grains.

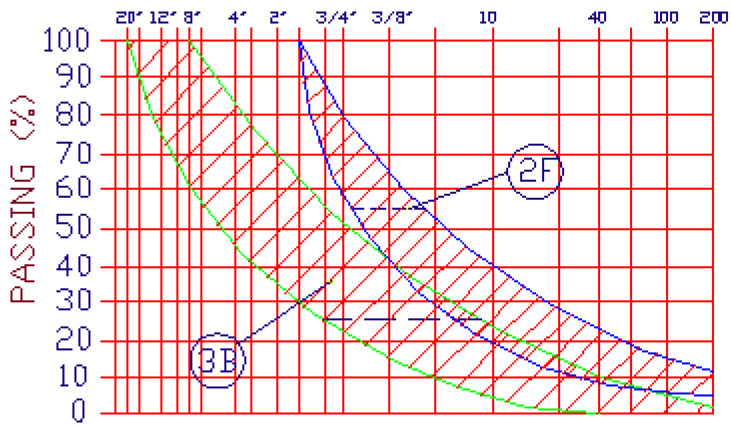
AGUAMILPA

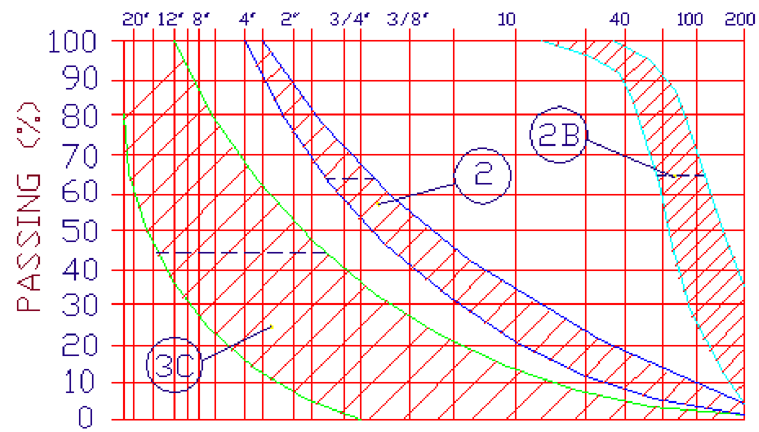


Aguamilpa Dam that for more than 15 years was the tallest dam(190m) of its type in the world. Even though the shell upstream of the dam was built with gravels of high deformation modulus, the greater compressibility of the rockfill used in the downstream shell generated an unusual situation by introducing a non-uniform pattern of deformations in the upper part of the face

AGUAMILPA







AGUAMILPA DAM

Maximum Section and Material Gradations

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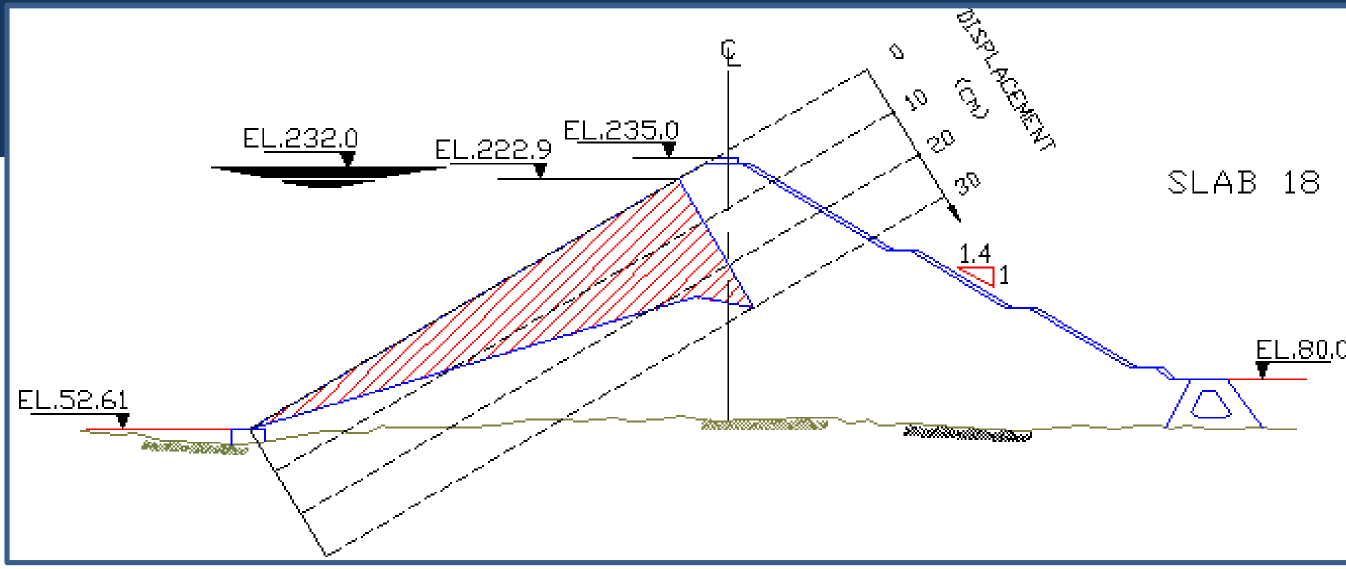
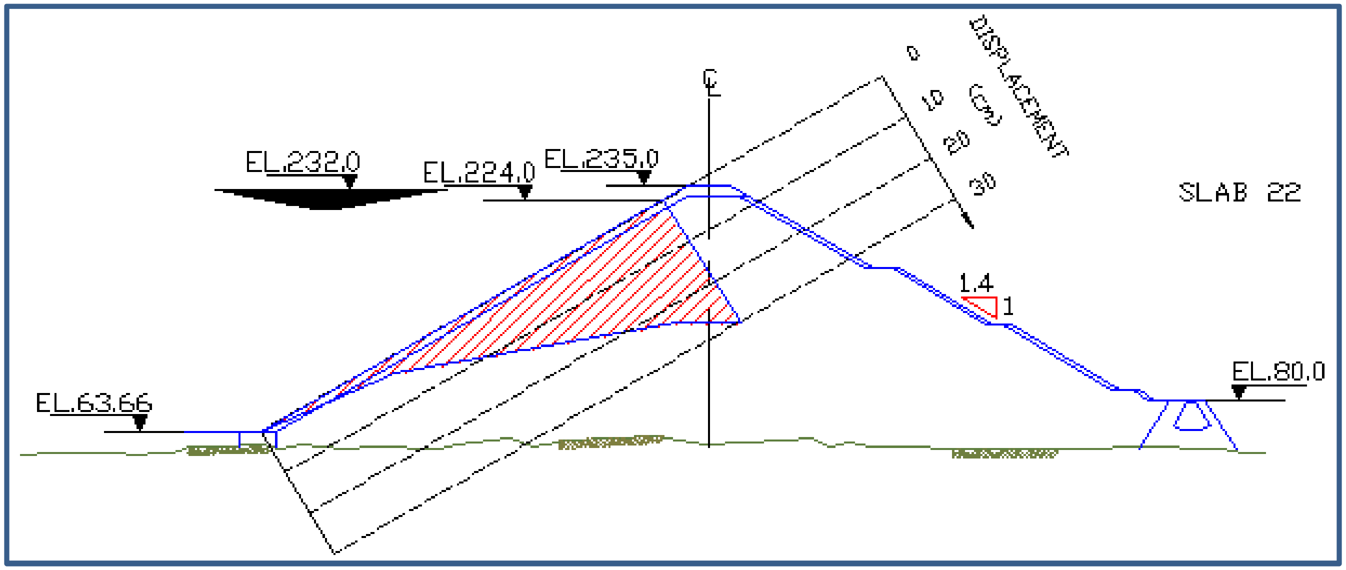
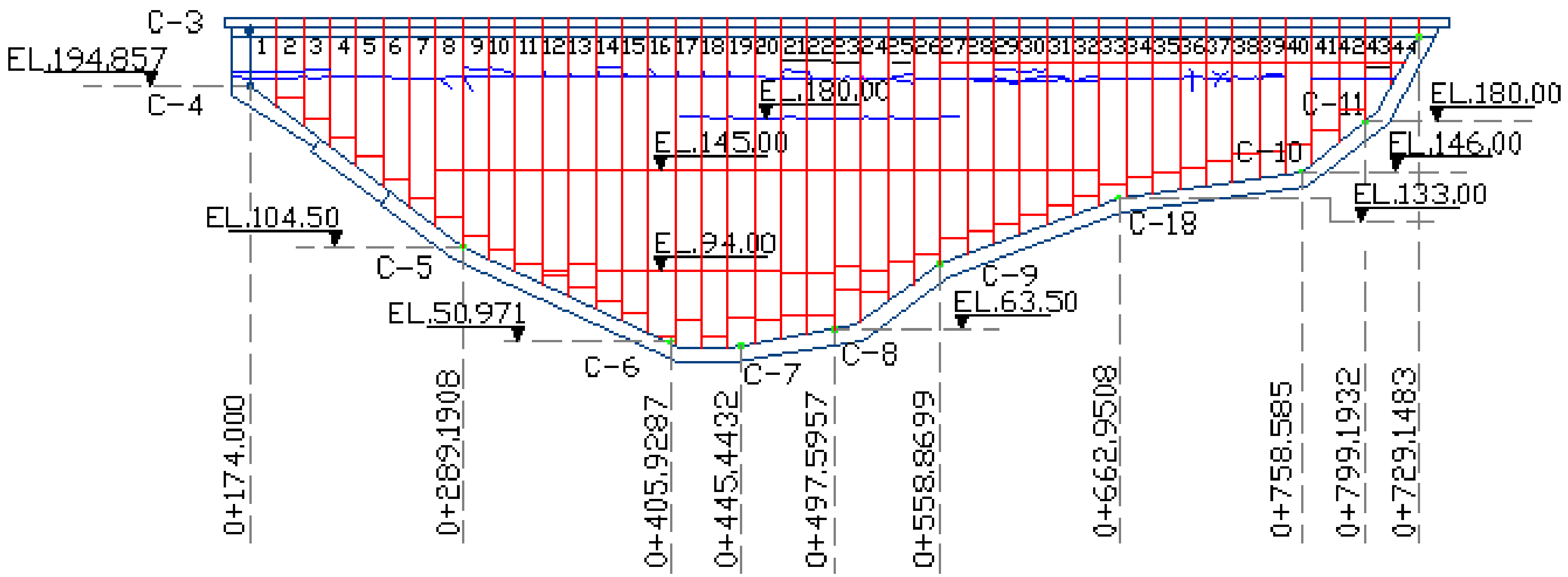


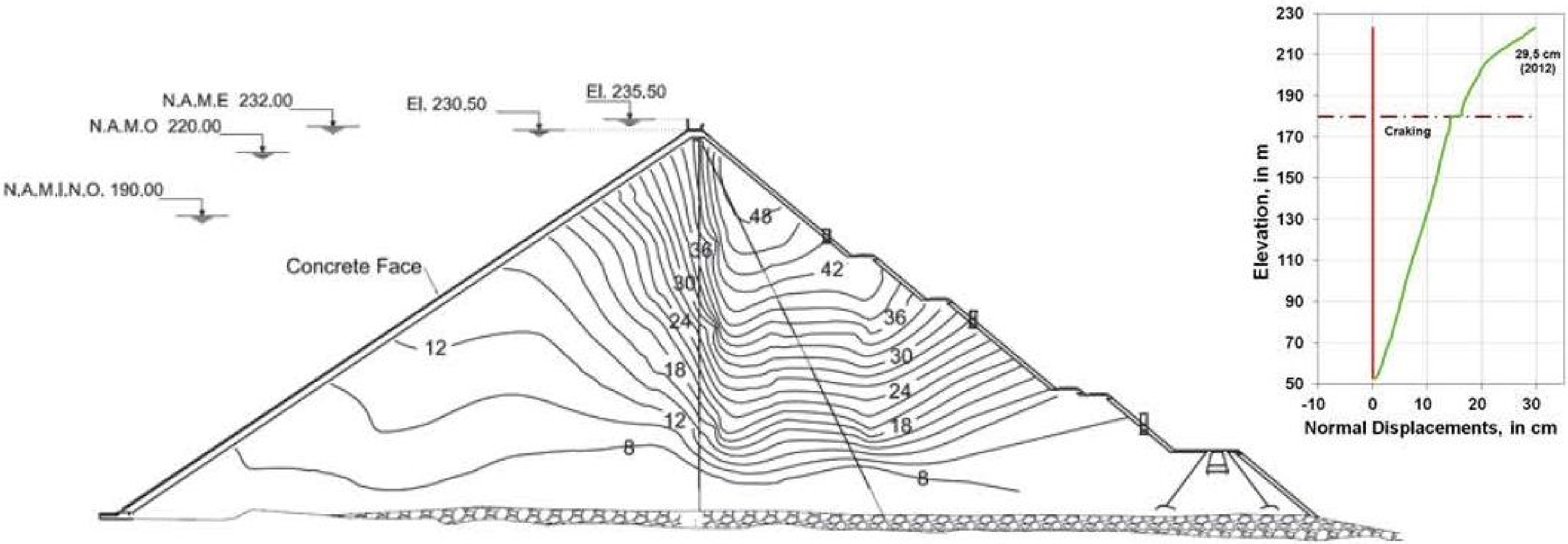


AGUAMILPA DAM
Normal Face Displacements

AGUAMILPA



AGUAMILPA DAM
Cracks in the Concrete Face



Settlement contours at Aguamilpa Dam as of October 2011
(First filling is included)

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Towards the final years of the last century there was a clear tendency to consider that the concrete face dams could be viable for heights of more than 200 m, basically without mayor changes in the configuration and design procedures commonly used. In some cases it was considered the ideal dam, and therefore, to some people there was no limit to its height.

Sherard & Cooke 1985 CFRD ASCE Symposium:

"The CFRD is an appropriate type in the future for the very highest dams. For a 300m high CFRD constructed of most all rock types, acceptable performance can be predicted, based on reasonable extrapolation of measurements on existing dams"

Sherard & Cooke 1985 CFRD ASCE Symposium:

"For CFRD with compacted rockfill and a compacted upstream face, the thickness increment was decreased to 0.003H, and even to 0.002H or less. These slabs have given satisfactory performance, an there is a current general trend toward thinner slabs."

Sherard , 1985 CFRD ASCE Symposium:

"….The writer believes that it is likely that the not distance future evolution of the CFRD could arrive at a constant slab thickness of the order of 8 to 10 inches, even for high dams, with simpler and more economical joint seals."

Cooke 2000 Beijing Symposium:

"There has since been no experience to change that conclusion. There have been leakage incidents, and for the CFRD "acceptable performance" can include a leakage incident."

"Experience with existing dams has not identified areas in design which require significant change in design practice for the next generation of higher dams, 190-230m"

- Towards the end of the last century, a new stage in the evolution of CFRD dams started. Considering that the height limitations for a dam of this type were overcame, the more or less simultaneous construction of very high dams started in China (TQ1), Brasil (Barra Grande and Campos Novos) and Lesotho (Mohale).

GENERAL TRENDS OF HIGH CFRD´s in 1998-2005

- About 190 m high
- Without anti-spalling reinforcement along compression joints
- Thickness reduction of the concrete slab
- In low-seismicity areas, outer slopes increased to 1.3 to 1 and 1.25 to 1.
- Use of an extruded concrete curb as surface protection before placing the slab
- Aguamilpa, a gravel fill dam, is not an adequate precedent for the behavior of high rock fill dams. Settlements were very low due to high modulus of fill

CAMPOS NOVOS DAM

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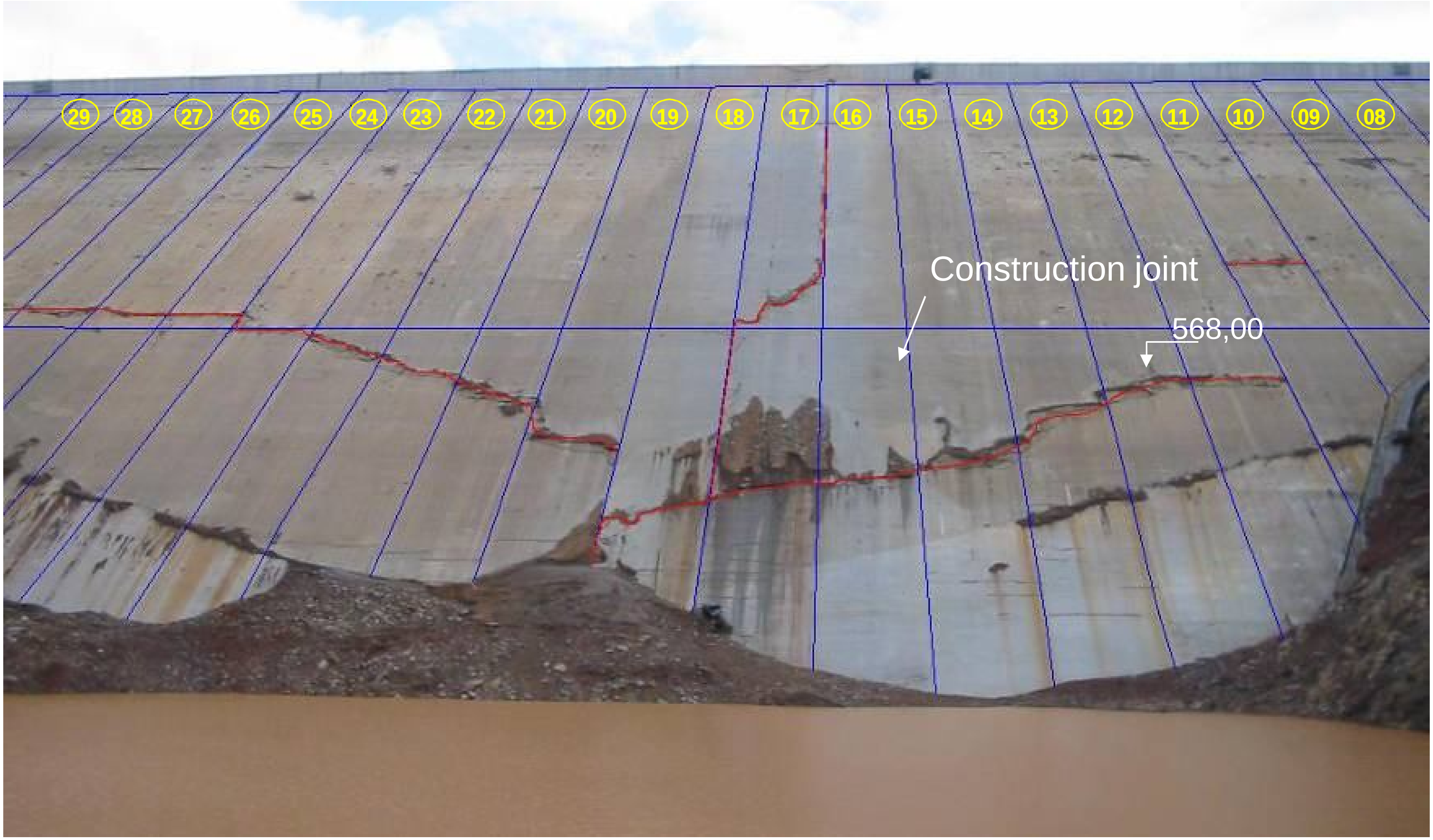
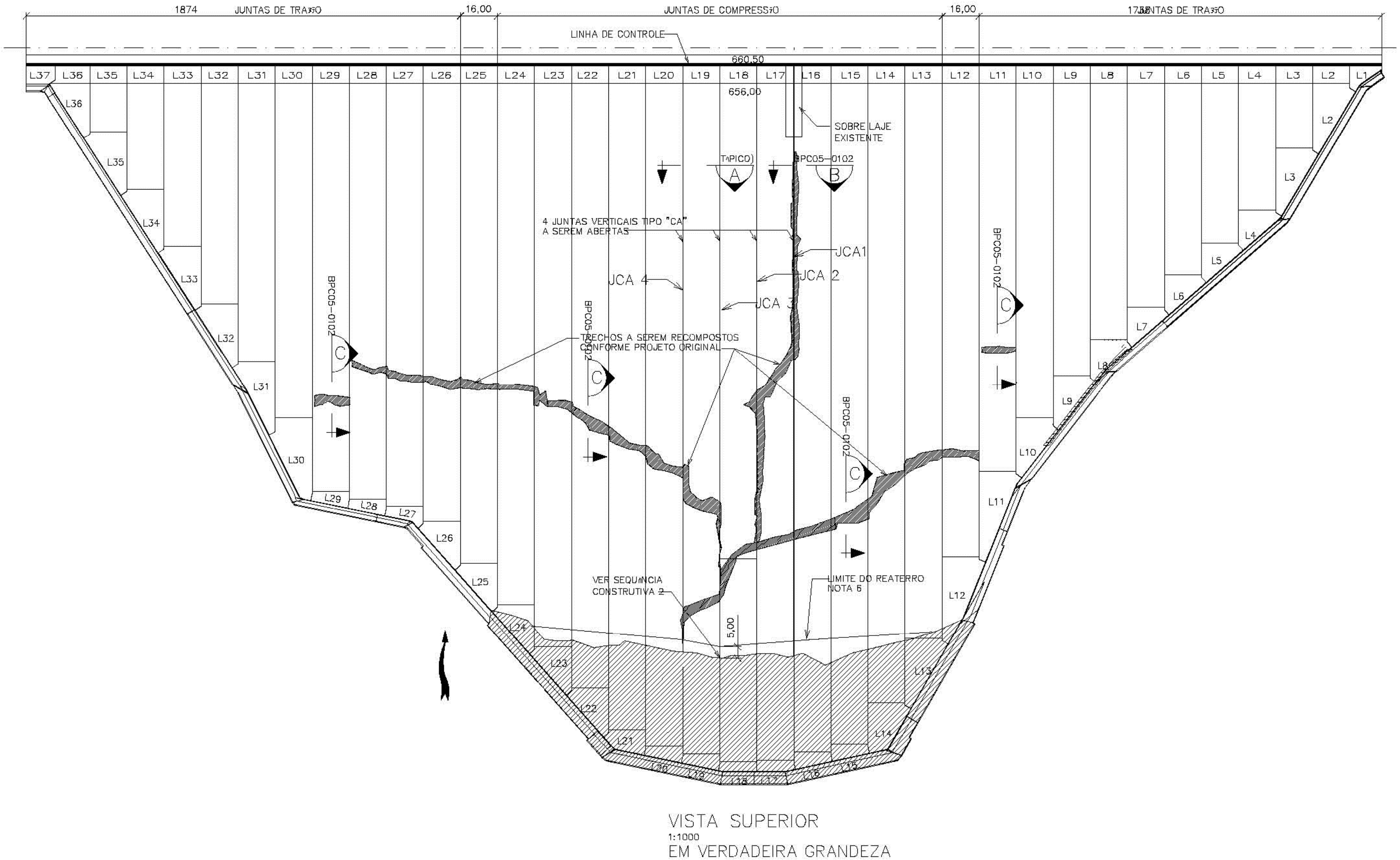
CAMPOS NOVOS: GENERAL CONCRETE FAILURE MAPPING





Campos Novos CFRD, Brasil, 2006

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JOINT 16 - 17

CONCRETE RAISED UP -26 CM

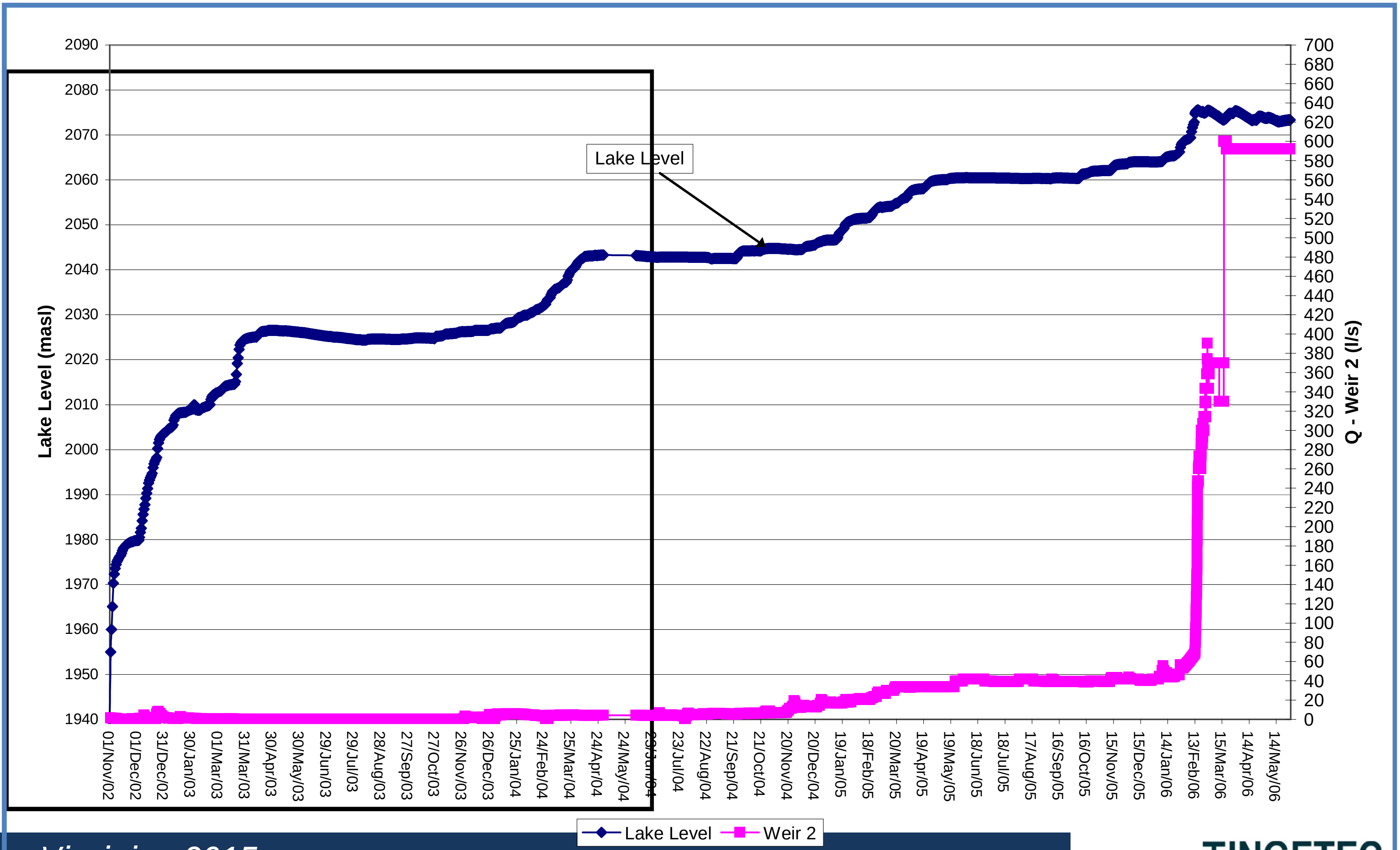
ZONE CLOSE TO THE INCLINED CRACK IN SLAB 17

TRANSVERSAL CRACK

Inclined cracks in slabs19, 20, 21, 22

Mohale dam embankment: face slab

Incident at the Mohale Dam (2006)



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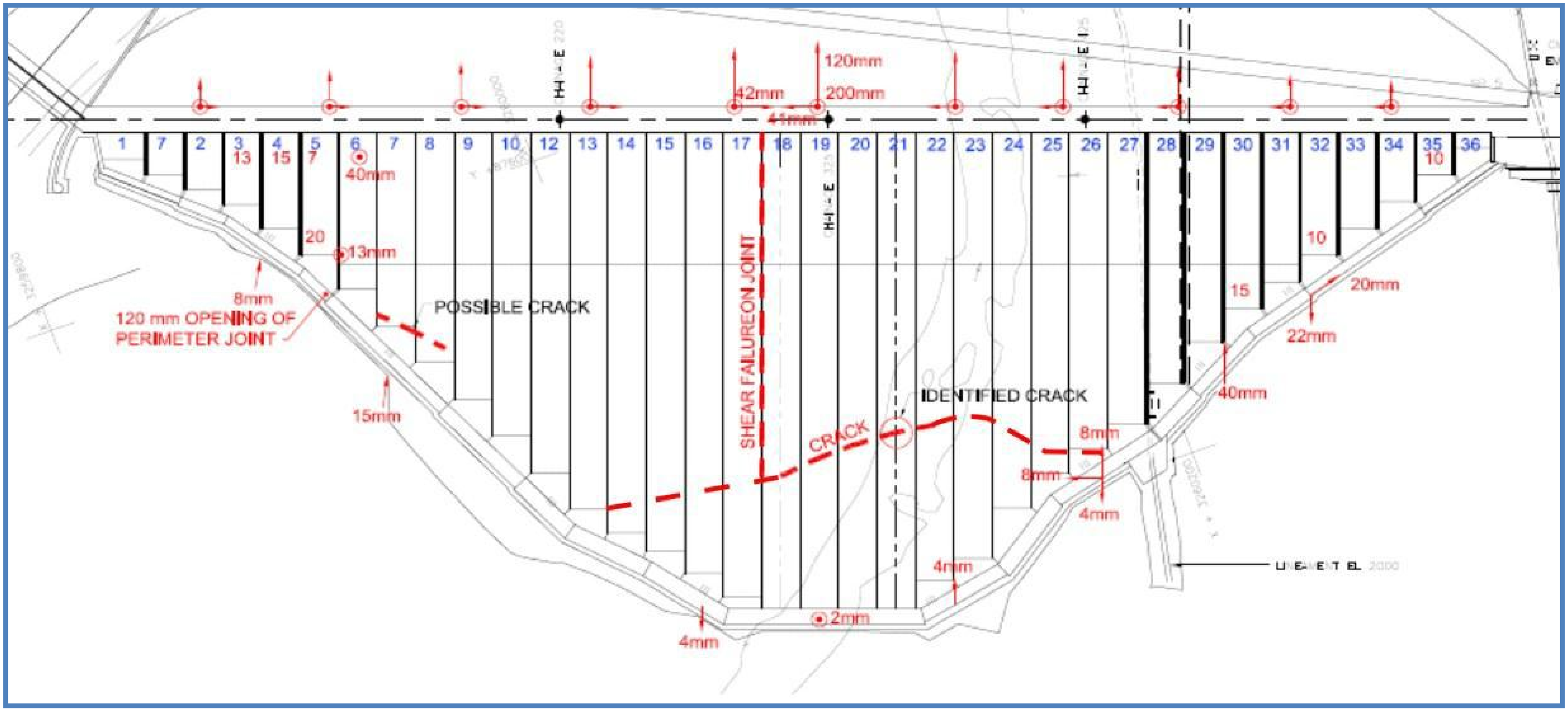
Shear failure along joint between Slabs 17/18

Location of cracks and failed joint



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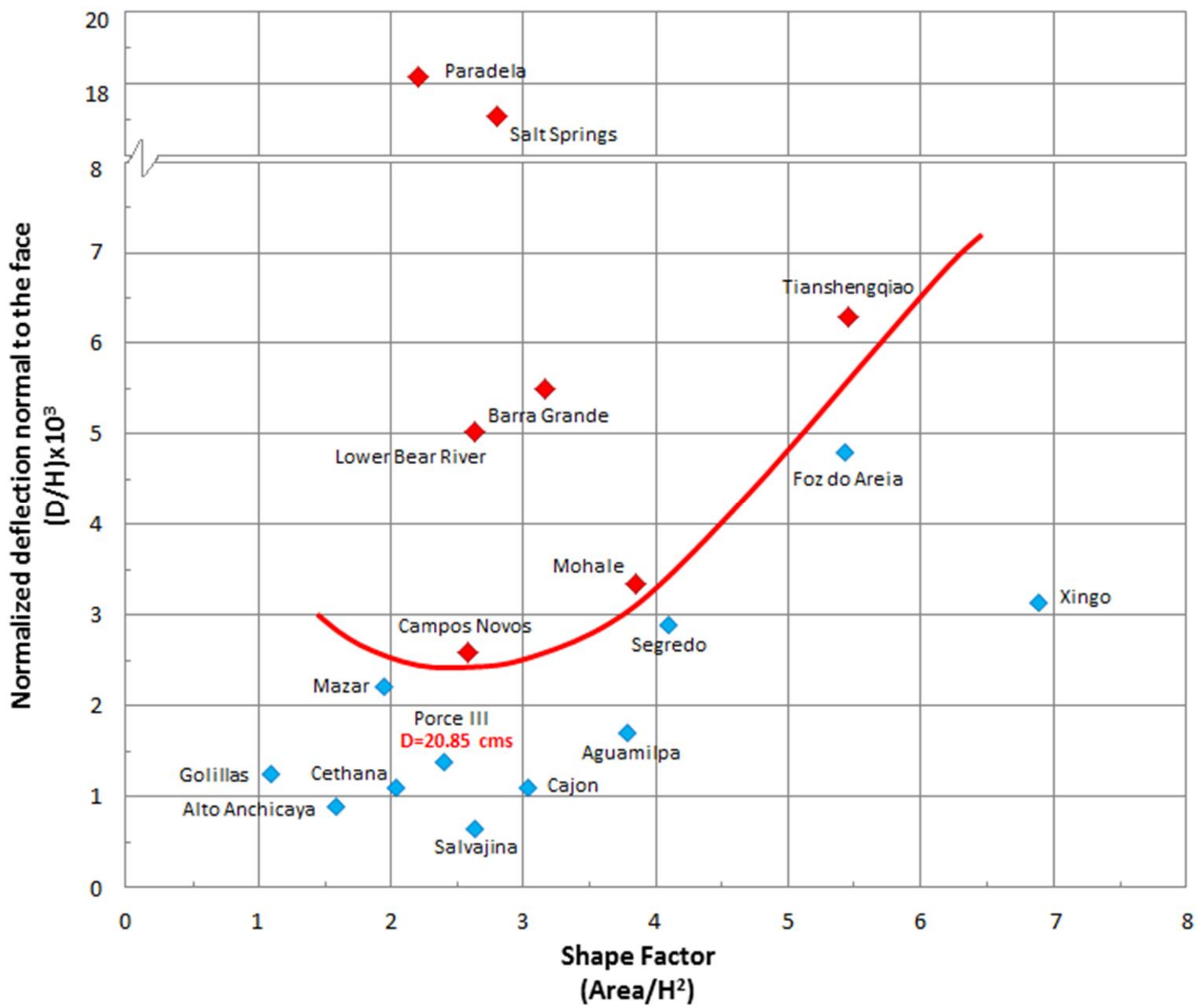

20
18
8
7
6
5
4
3
2
1
0
Paradela
Salt Springs
Tianshengqiao
Barra Grande
Lower Bear River
Foz do Areia
Mohale
Xingo
Campos Novos
Segredo
Mazar
Porce III
D=20.85 cms
Aguamilpa
Golillas
Cethana
Cajon
Alto Anchicaya
Salvajina
Normalized deflection normal to the face
(D/H)x10³
0 1 2 3 4 5 6 7 8
Shape Factor
(Area/H²)

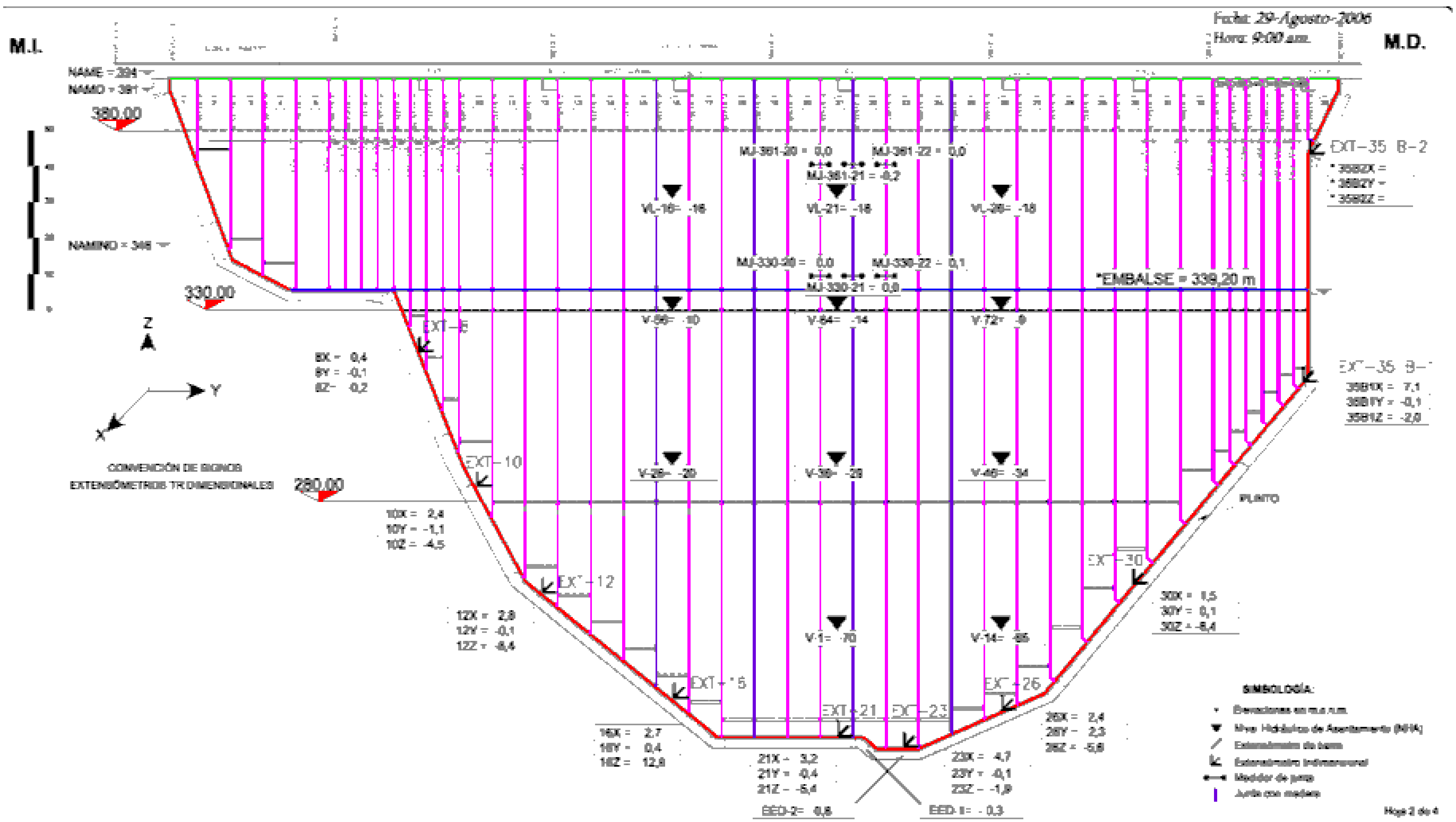
- When the three serious incidents of the dams in Brasil and Lesoto took place, two very high dams were being built, one in Mexico (Cajón, 188m) and one in Island (Karajnukar, 200 m). The rockfill materials used in these dams were different from the ones used in the problematic cases.

- Concept of obtaining required modulus between certain limits and with good gradation depending on compaction effort. Low strength particles can still produce and acceptable rockfill if well graded.
- Modulus between 100MPa and 50 MPa can be obtained even with weak particles. Compaction effort in terms of number of passes, thickness of layers and weight of vibrating roller. Water sluicing is also a must.
- Obtaining good gradation with hard rocks is very difficult and expensive. In basalts is even more difficult if they are columnar basalts.

- Most difficult aspect to evaluate is the effect of compaction in the materials characteristics. How much does it degrade.
- Fill test is advisable. Degradation is easily measured. Compaction parameters can be evaluated using odometer and plate load tests not to obtain final modulus but differences among them.
- Fill design should be optimized based on available materials. Hauling distances should be optimized. This is standard practice even for dams in narrow canyons where variable rock conditions are expected in quarries.

CAJON CFRD: MEXICO

CAJON DAM. FACE JOINT MEASUREMENTS
FIRST FILLING OF THE RESERVOIR (EL. 309.2)



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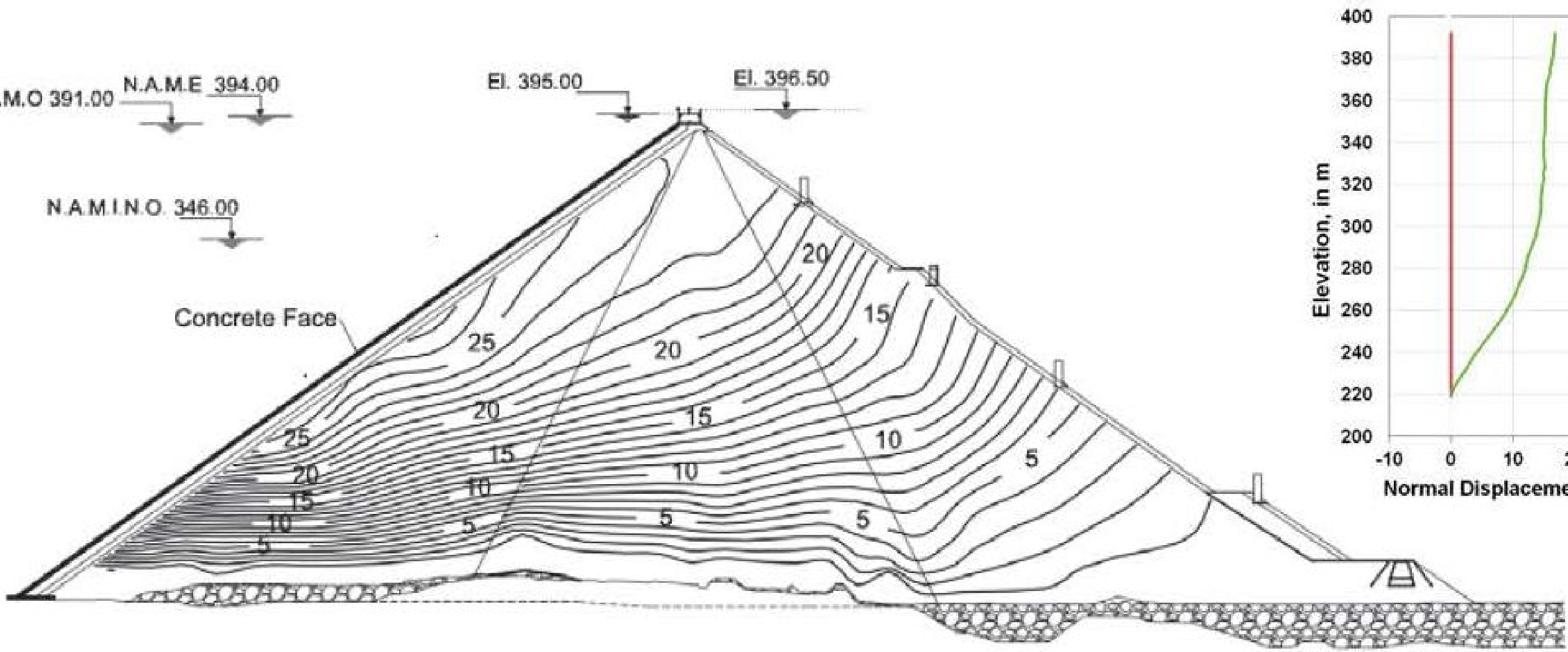
CAJON CFRD: SLUICING OF ROCKFILL

ROCKFILL: MATERIAL PROPERTIES

			Compresive Sttrength			Weigth	Gs	Absorption			LA Abrasion		
			Mean	Max	Min			Mean	Max	Min	Mean	Max	Min
			[Mpa]			[kN/m^3]	[g/cm^3]	[%]			[%]		
Cajón	Shallow Rock (<18m)	Dry	70.5	97.6	46	23.6		4.42	4.31	4.54			
		Saturated	52.9	73.2	34.5								
	Underground Rock (>18m)	Dry	124.7	131.89	111.05	23.3		4.42	4.31	4.54			
		Saturated	104.7	110.8	93.3								
Barra Grande	Specification		<50				<2.55	<3			<25		
	Vesículo Amigdaloidal	Saturated	99.4	189	16		2.6	2.2	4.8	0.3	25	33	18
	Dense Basalt	Dry	119.2	165.7	60.8		2.84	0.76	1.91	0.2	15	15	15
		Saturated	101.9	204.2	35.4								



Settlement contours measured at El Cajón Dam as of May 2011
(First filling is included)

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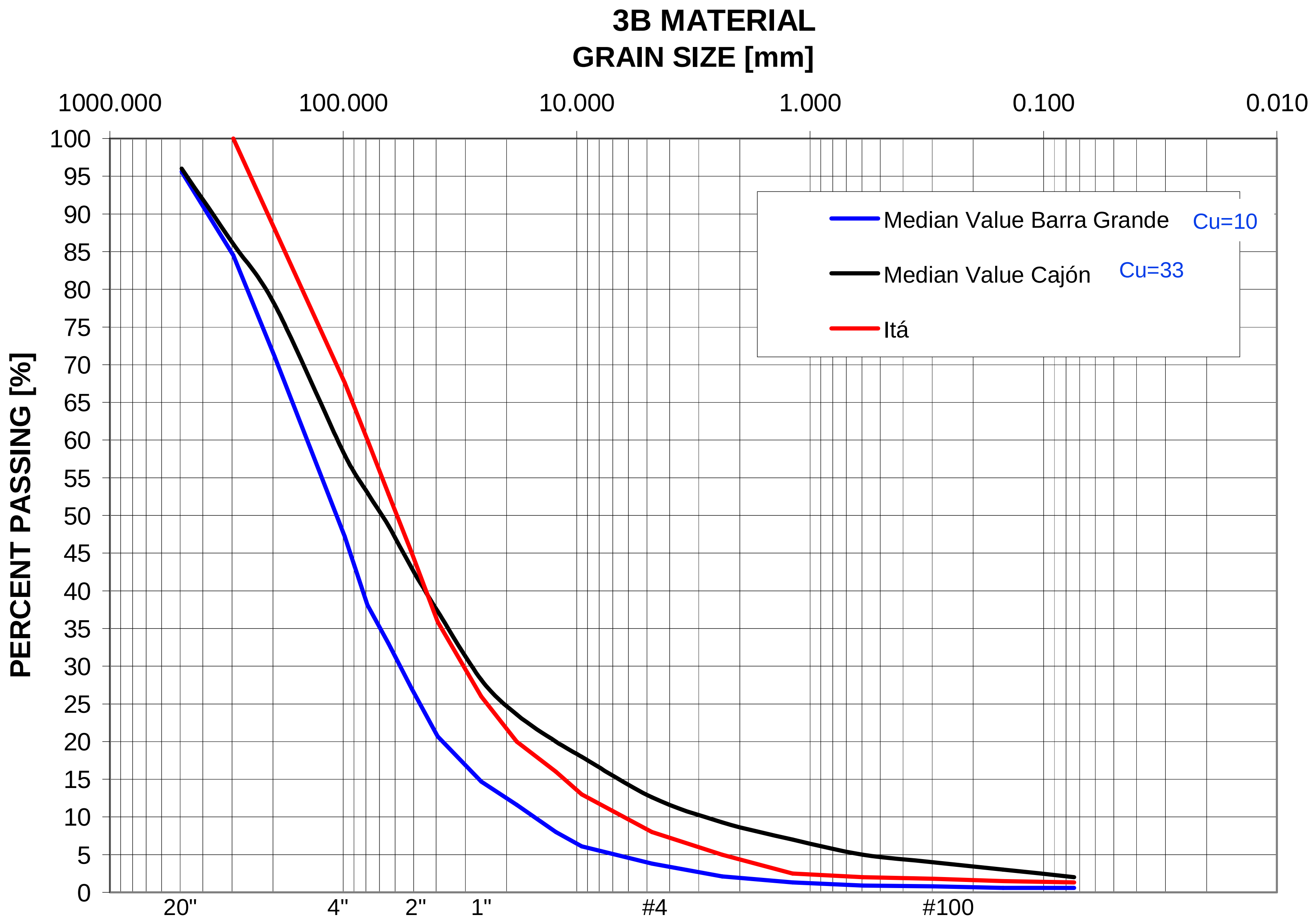
CAJON DAM. FIRST FILLING OF THE RESERVOIR

ROCKFILL: MATERIAL PROPERTIES



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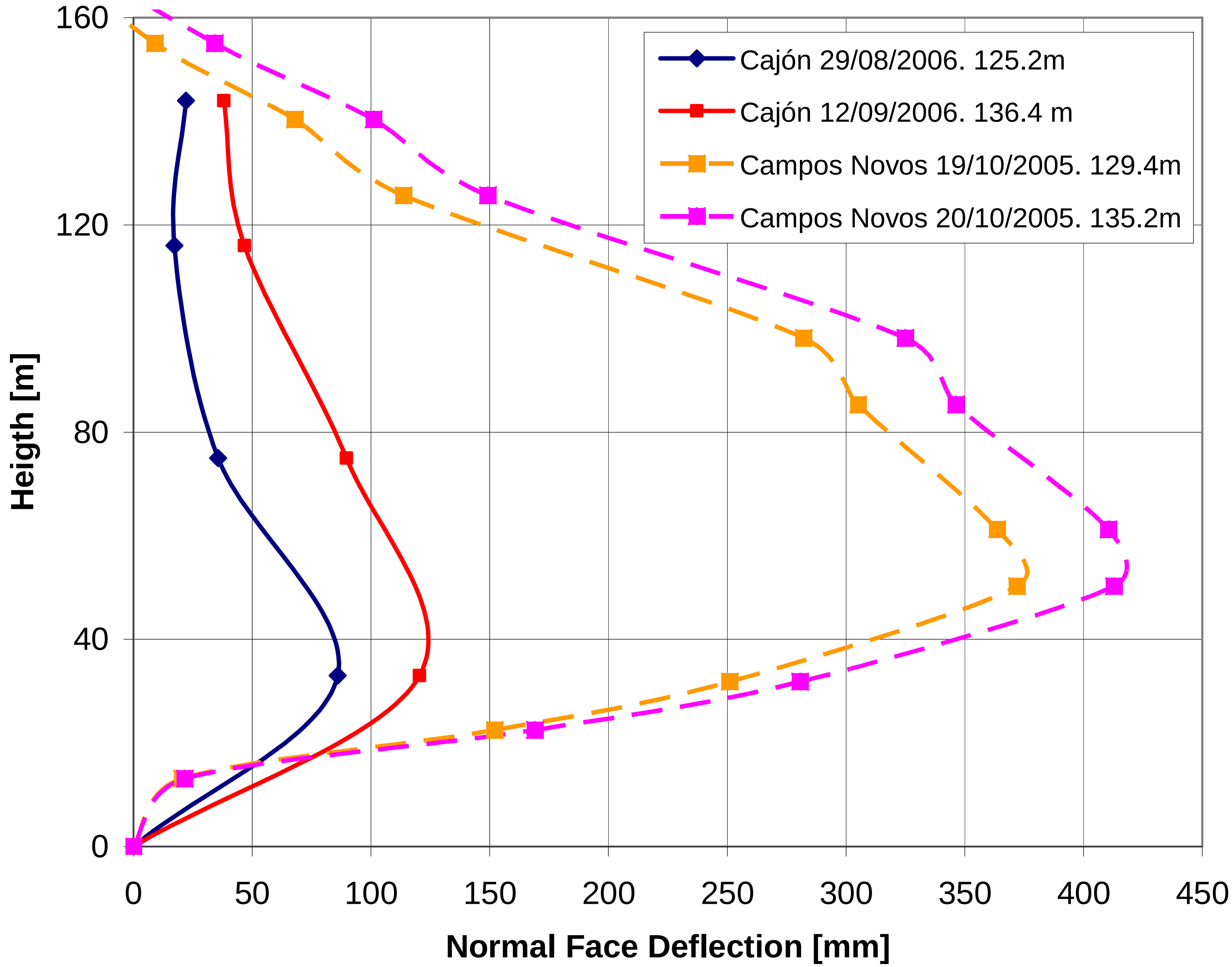
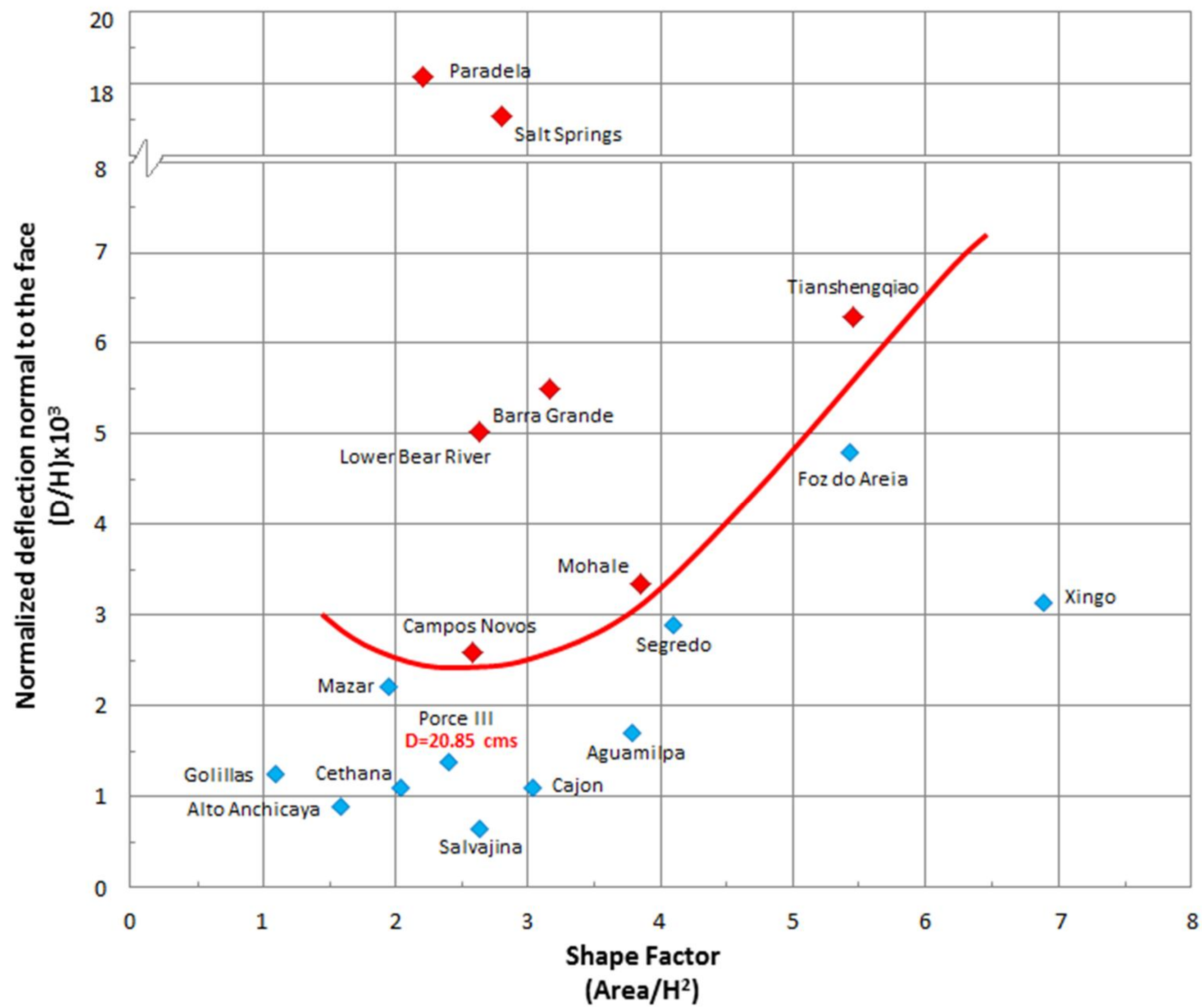
FACE DEFLECTION @ CENTER SLAB



INGETEC INGENIEROS CONSULTORES


Normalized deflection normal to the face (D/H)x10³
Shape Factor (Area/H²)
Paradela
Salt Springs
Tianshengqiao
Barra Grande
Lower Bear River
Foz do Areia
Mohale
Xingo
Campos Novos
Segredo
Mazar
Porce III
D=20.85 cms
Aguamilpa
Golillas
Cethana
Cajon
Alto Anchicaya
Salvajina
0
1
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18
20

ANALYSIS REQUIREMENTS

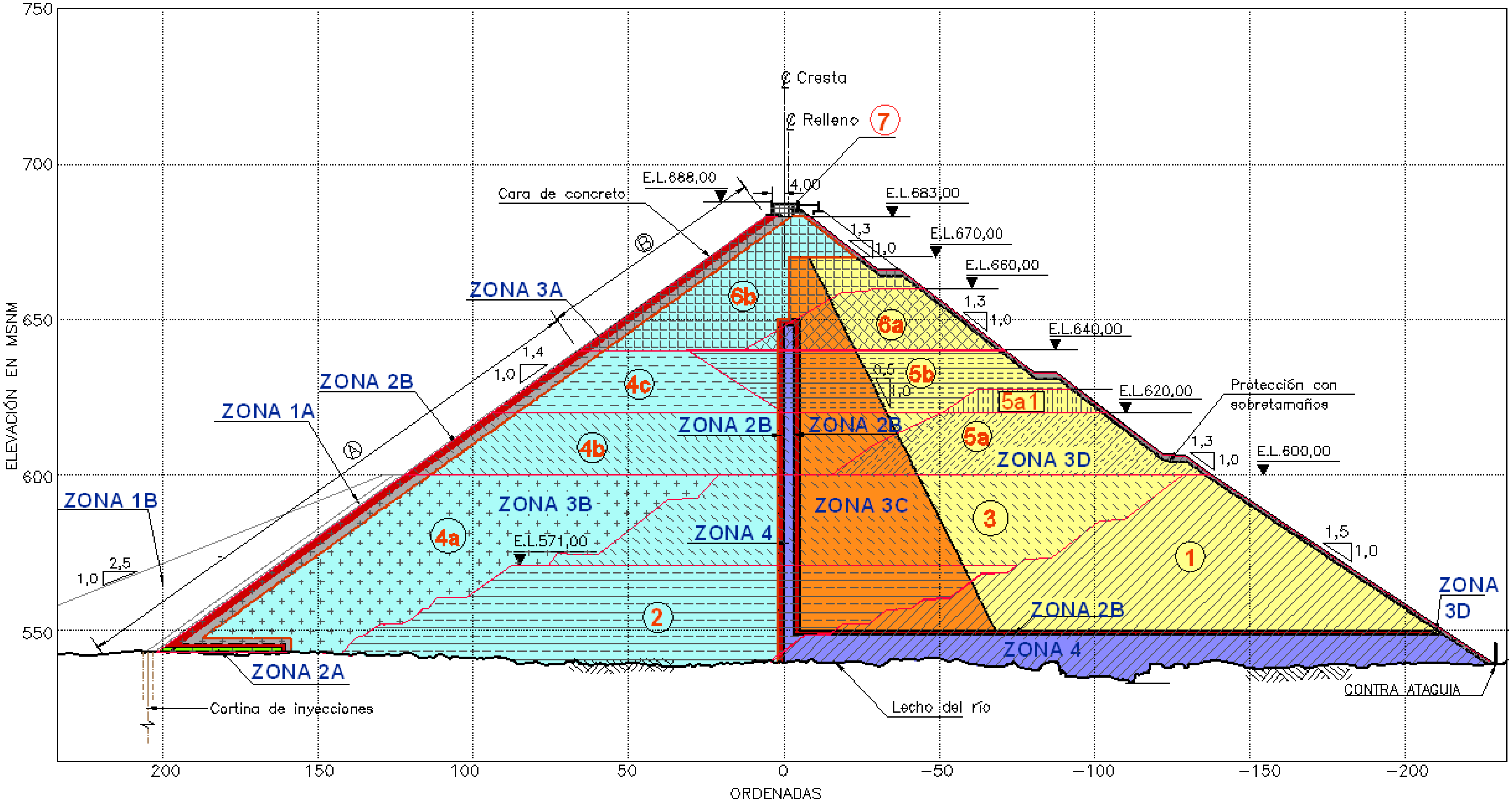
- Development of a three dimensional model
- Construction sequence
- Modeling the structural elements
- Constitutive models for geomaterials
- Incorporation of interface behavior between different elements of the structure

Paris Benchmark Workshop

Title	Authors	Company/Institution
Analysis of a concrete face rockfill dam including concrete face loading and deformation using program package SOFiSTiK	Gjorgi Kokalanov Ljubomir Tanĉev Stevcho Mitovski Slobodan Lakoĉević	Civil engineering school of Skopje.
DIANA Analysis of a concrete faced rockfill dam	Gerd-Jan Schreppers Giovanna Lilliu	TNO DIANA, Delft NL.
A CFRD case using 3D modelling	C. Nieto J-C. Philippe M. Werst P. Anthiniac	Tractebel Engineering-Coyne Et Bellier. Gennevilliers Cedex, France.
Analysis of a concrete face rockfill dam including concrete face loading and deformation	C. Marulanda E. Leon	INGETEC, Colombia.

In general terms, it was concluded from the 10th Benchmark workshop that predicting rockfill behaviour relatively well is possible with the available numerical models. The main difficulty when modelling the CFRD behaviour is the interaction between the rockfill and the structural elements (i.e., face, plinth, joints, curb).

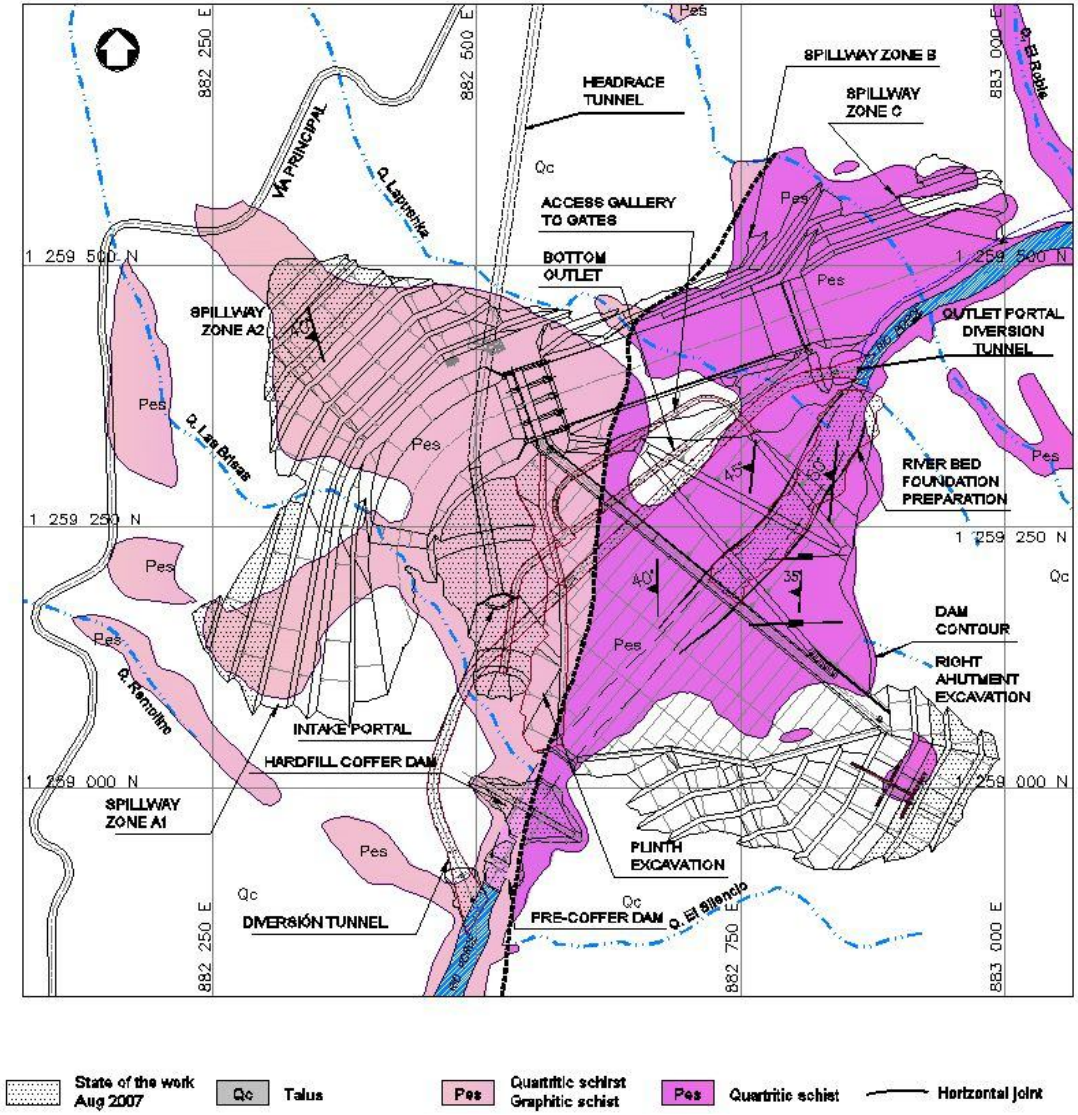
ZONES AND STAGES OF THE DAM



SECCIÓN MÁXIMA PRESA (ABSCISA 210)

INGETEC INGENIEROS CONSULTORES

PORCE III: GEOLOGY AT DAM SITE. PLAN VIEW



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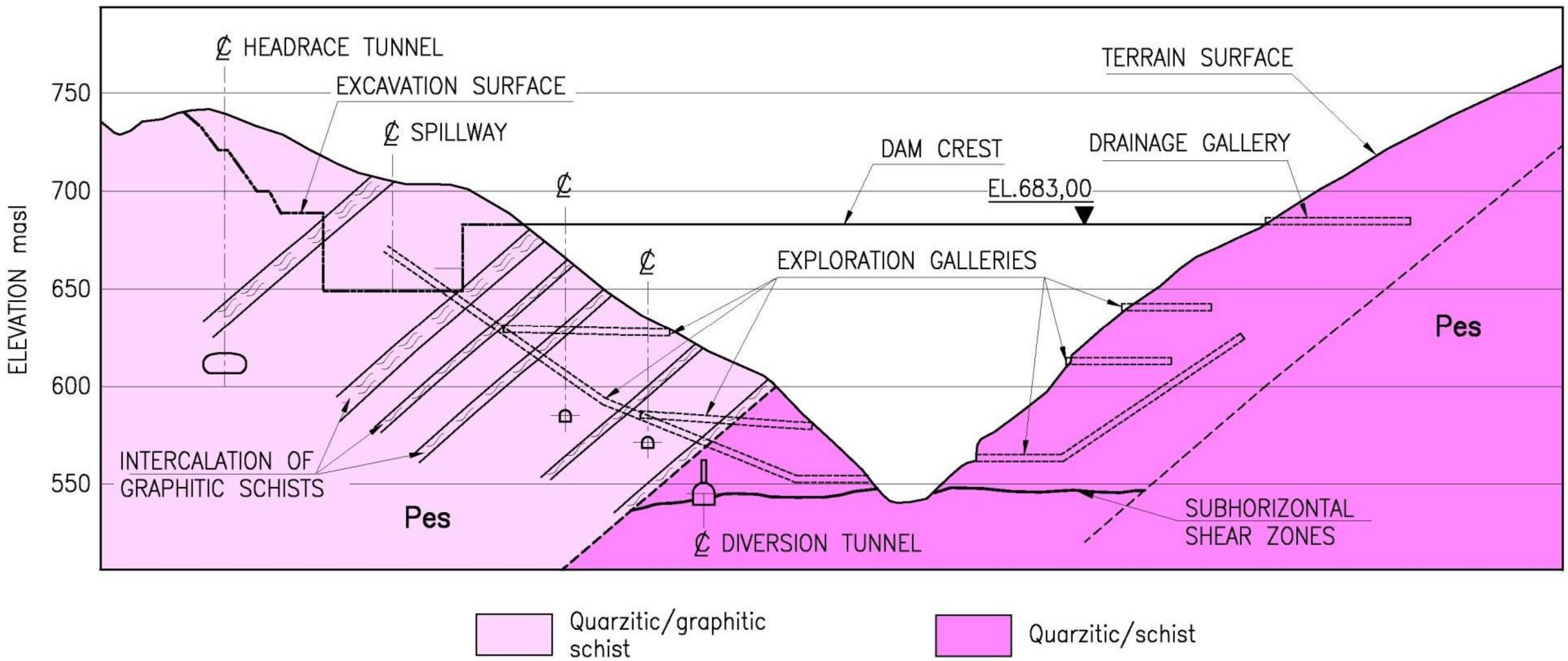
PORCE III: GEOLOGY AT DAM SITE



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PORCE III: TYPES OF SCHISTS

PORCE III: TYPES OF SCHISTS

PORCE III: TYPES OF SCHISTS

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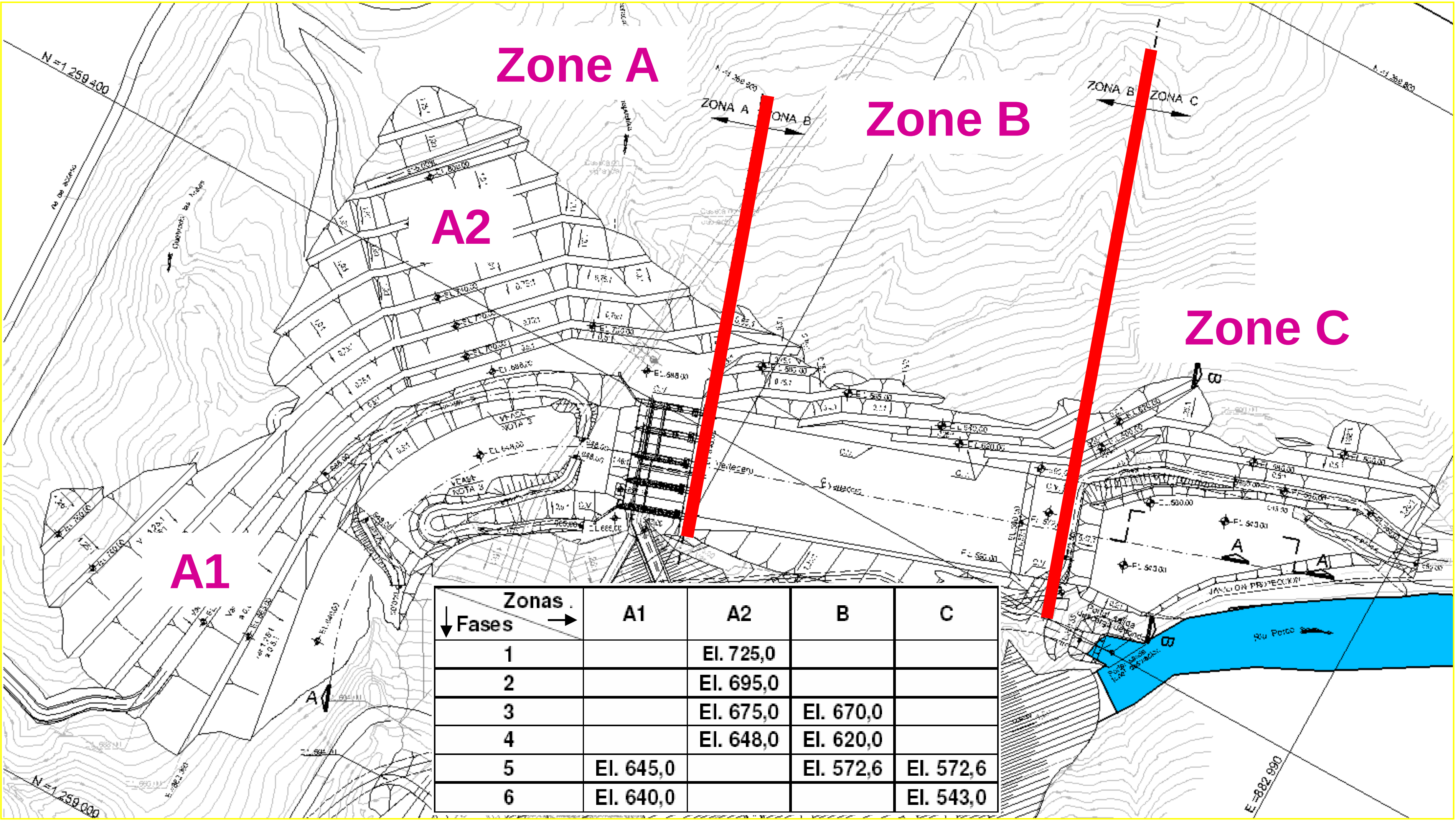
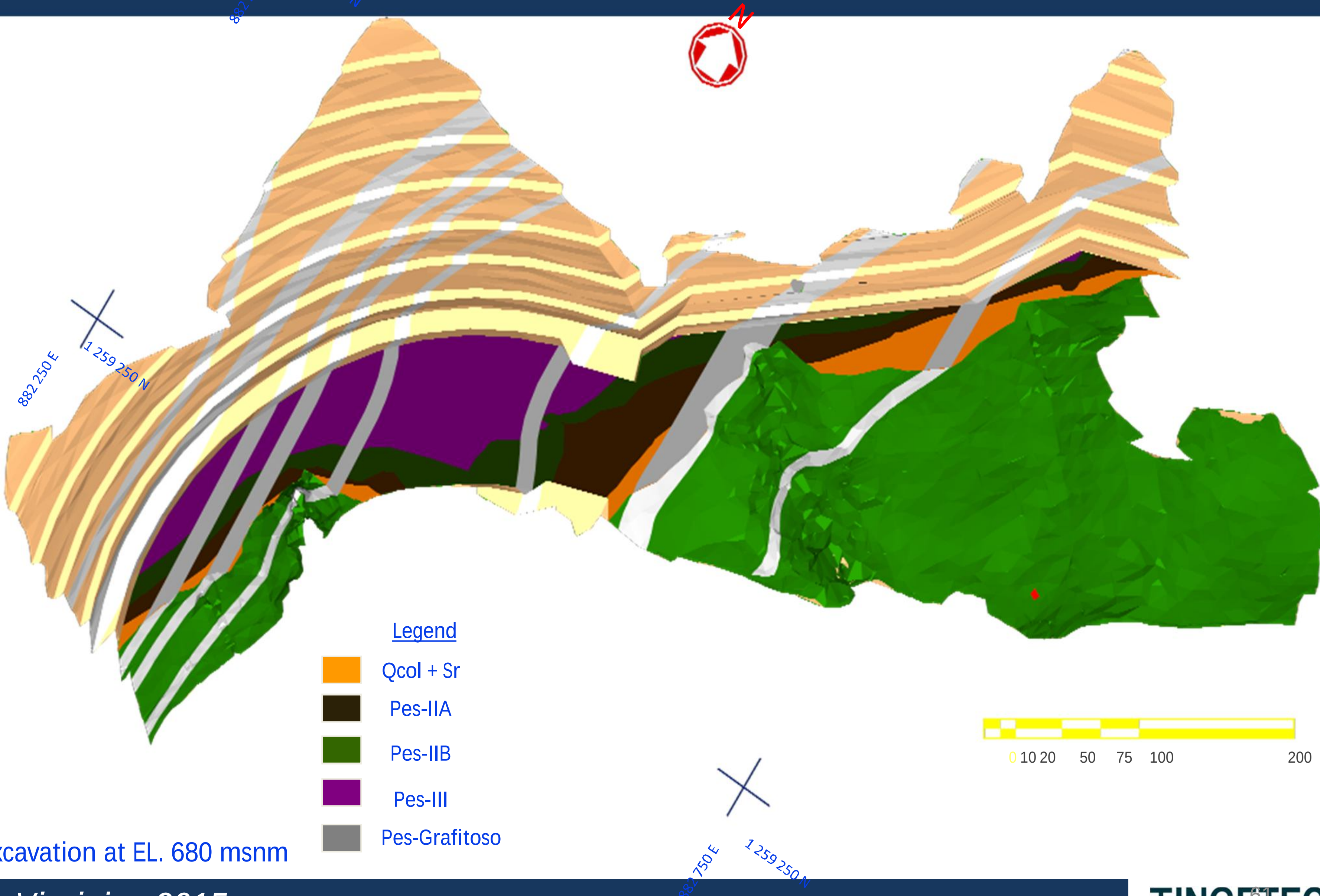
PORCE III: SOURCE – SPILLWAY ZONES



Zonas → / ↓ Fases	A1	A2	B	C
1		El. 725,0		
2		El. 695,0		
3		El. 675,0	El. 670,0	
4		El. 648,0	El. 620,0	
5	El. 645,0		El. 572,6	El. 572,6
6	El. 640,0			El. 543,0

PORCE III: SPILLWAY EXCAVATION



Excavation at EL. 680 msnm

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PORCE III: PLATE LOAD TEST ON TRIAL FILL

PORCE III: DAM FOUNDATION

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PORCE III: PLACEMENT OF 3D ZONE MATERIAL

PORCE III: PLACEMENT OF 3D ZONE MATERIAL

PORCE III: SLUICING OF 3D ZONE MATERIAL

PORCE III: PLACEMENT OF 3D ZONE MATERIAL

PORCE III: ZONES 3D & 3C. DAM FILL AT EL. 555,7

PORCE III: SLUICING OF 3D ROCKFILL

PORCE III: DOWSTREAM SLOPE. END OF STAGE 1

PORCE III: PLINTH EXCAVATION LEFT ABUTMENT

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PORCE III: STAGE 2 ZONE C. SPREADING PROCESS 0.6M LAYER

PORCE III: STAGE 2 ZONES 3D & 3C

PORCE III: DAM FILLS. CONSTRUCTION OF STAGE 2

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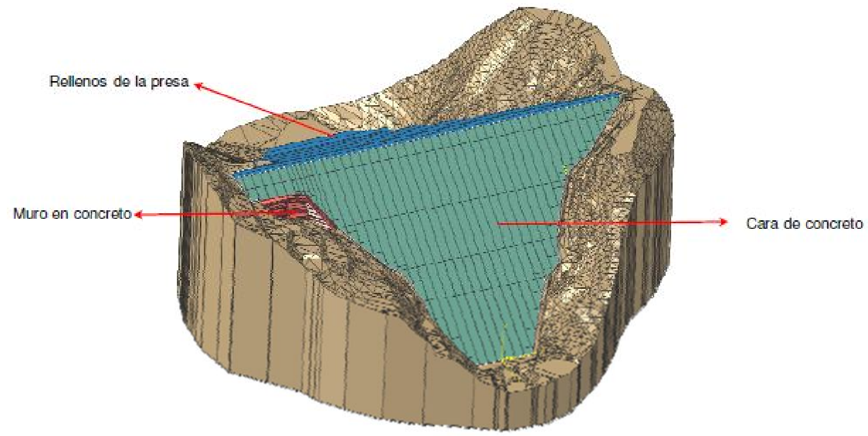
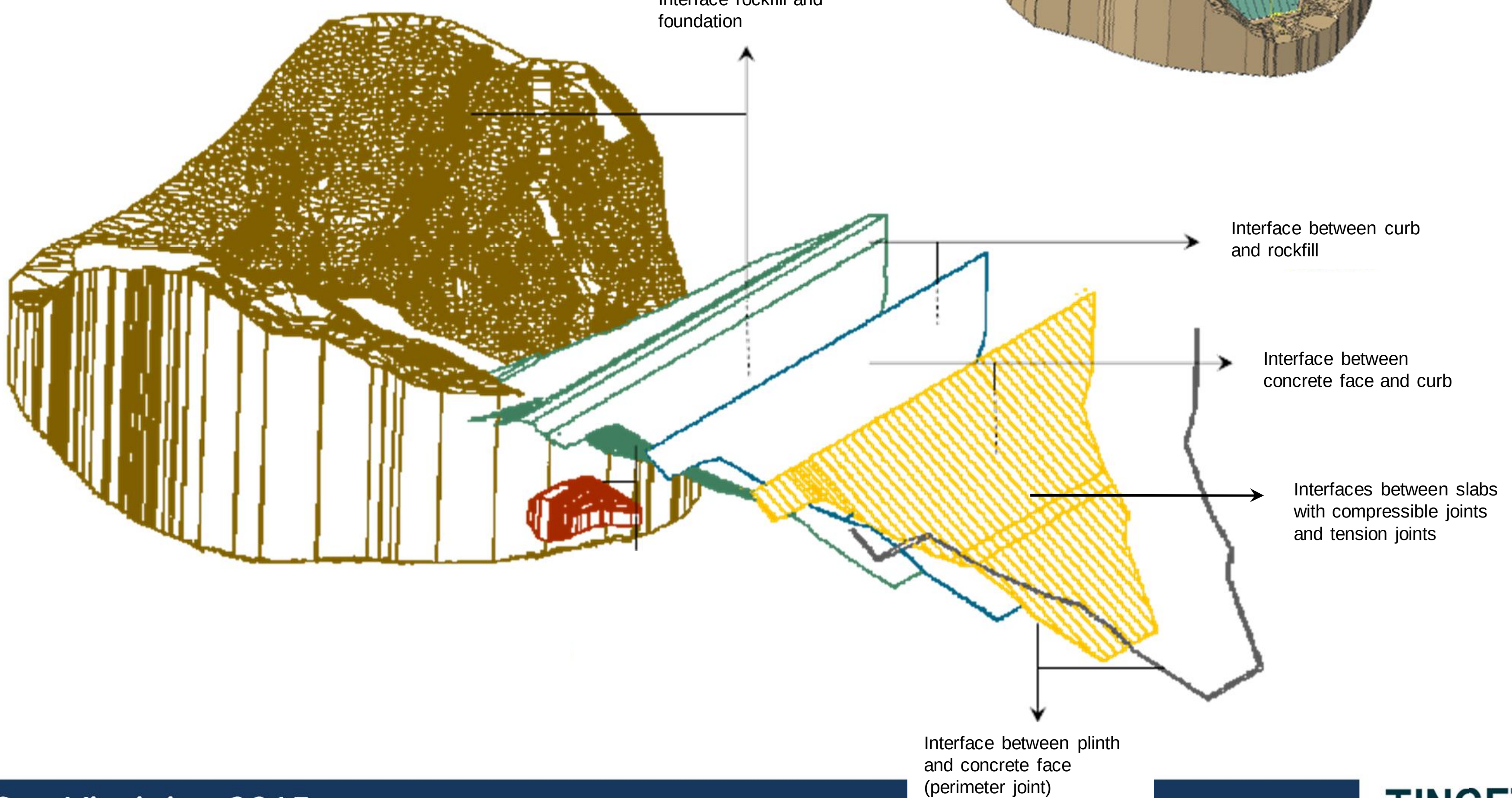
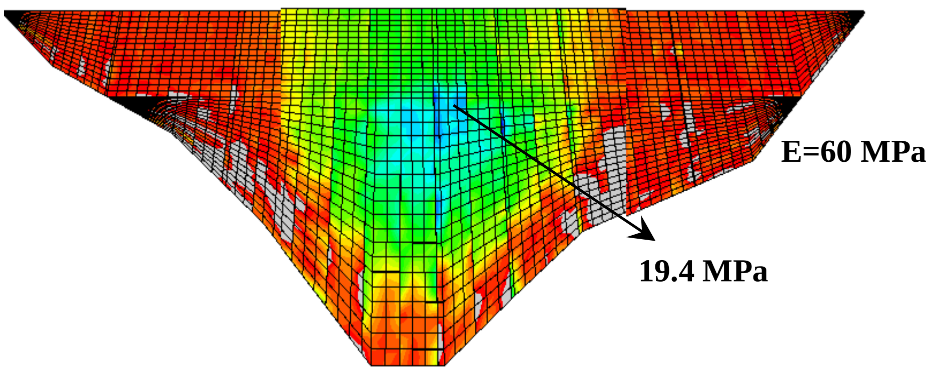
PORCE III

Interfaces


Rellenos de la presa
Muro en concreto
Cara de concreto

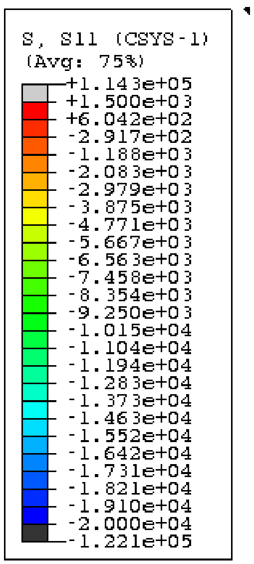

Interface rockfill and foundation
Interface between curb and rockfill
Interface between concrete face and curb
Interfaces between slabs with compressible joints and tension joints
Interface between plinth and concrete face (perimeter joint)

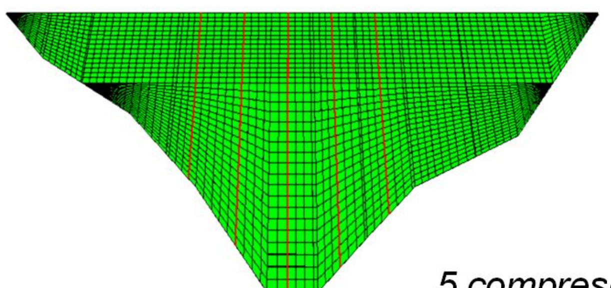
PORCE III

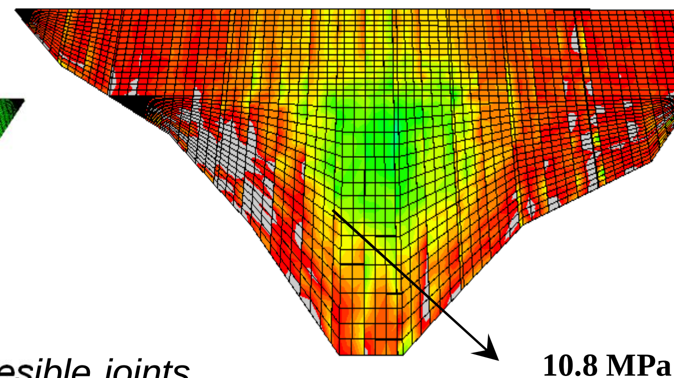


No compresible joints

- Project: Porce III (Col)
- Dam height: 150m
- Crest length: 330m
- A/H²: 2.4

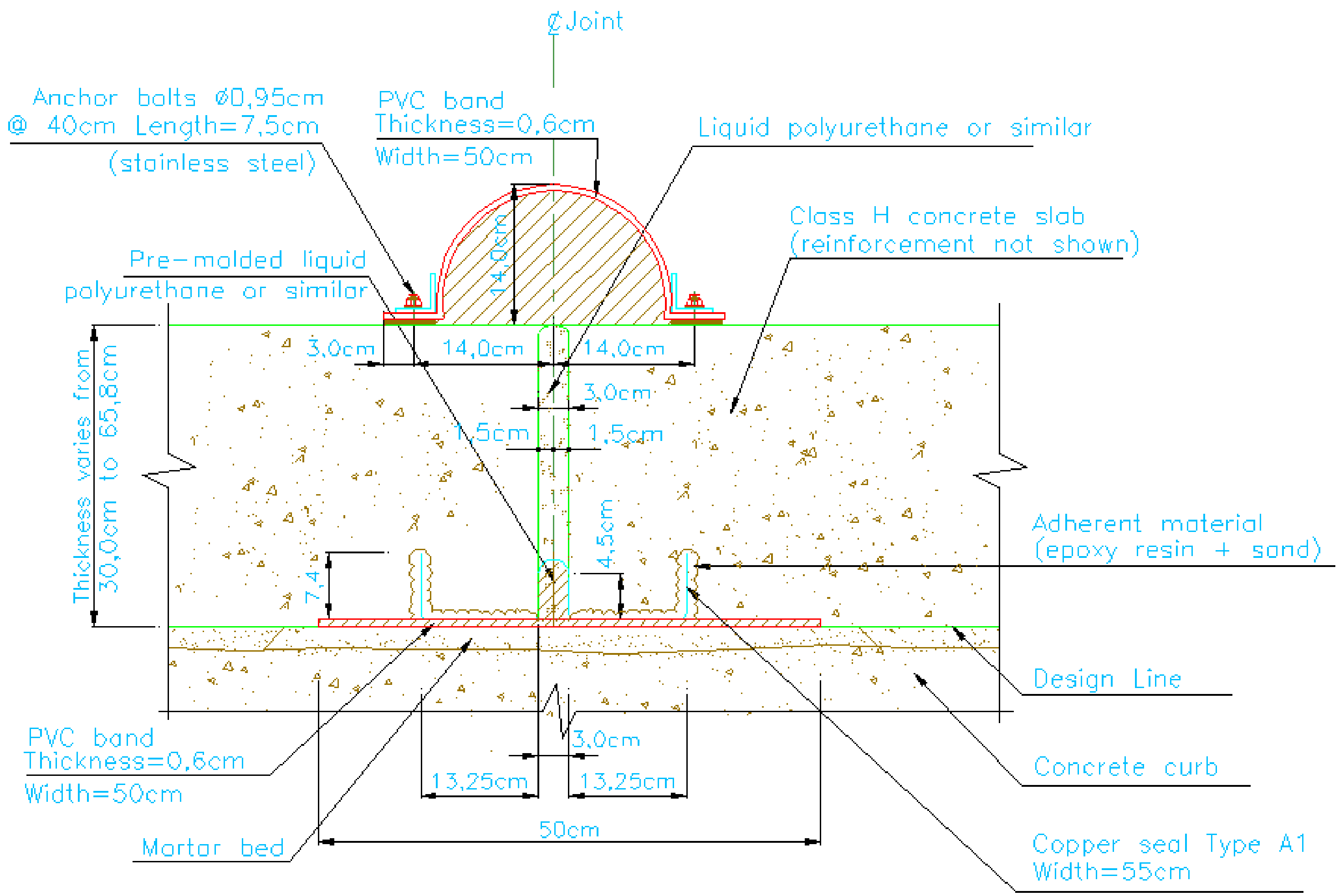






5 compresible joints

COMPRESSIBLE VERTICAL JOINT

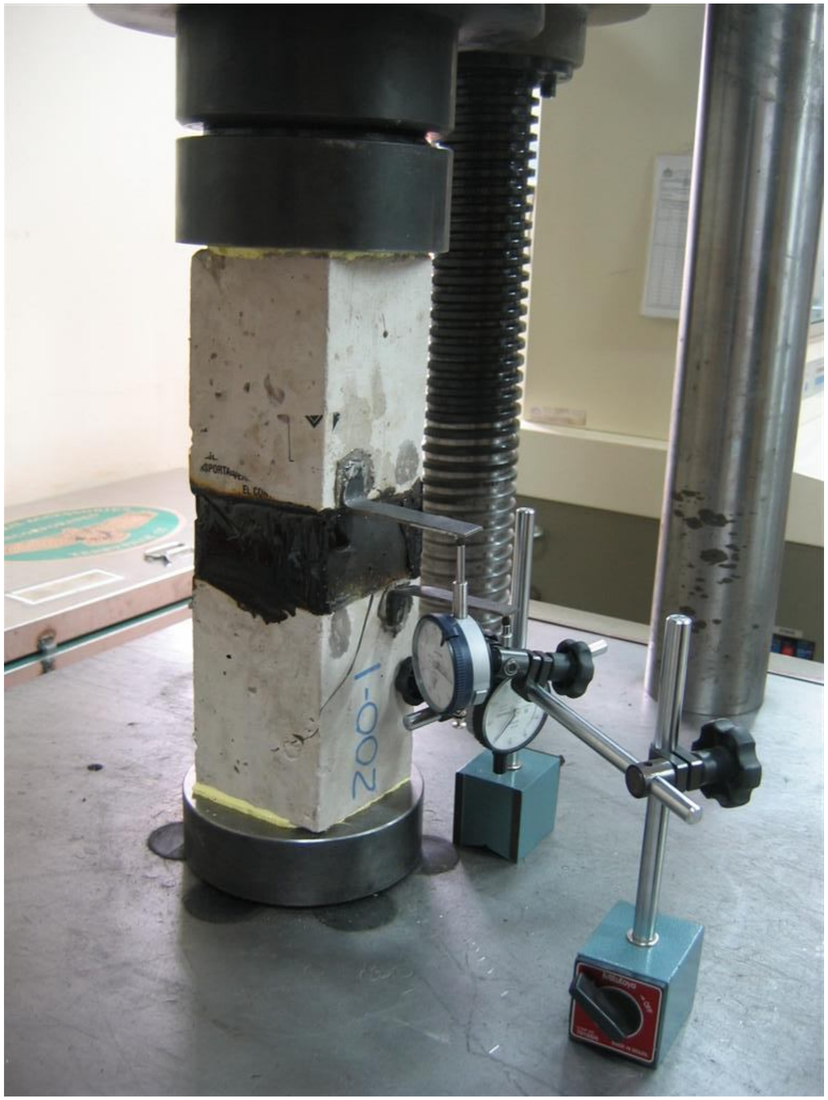


COMPRESSIBLE JOINT DETAIL
IN INTERIOR SLABS

PORCE III

LABORATORY TEST ON COMPRESSIBLE MATERIAL

LABORATORY TEST ON COMPRESSIBLE MATERIAL



PROGRESS WORKS OF CONCRETE FACE

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SLIDING SLAB PROCESS BETWEEN EL. 640 AND EL. 683

VIEW OF CONCRETE FACE FROM LEFT MARGIN

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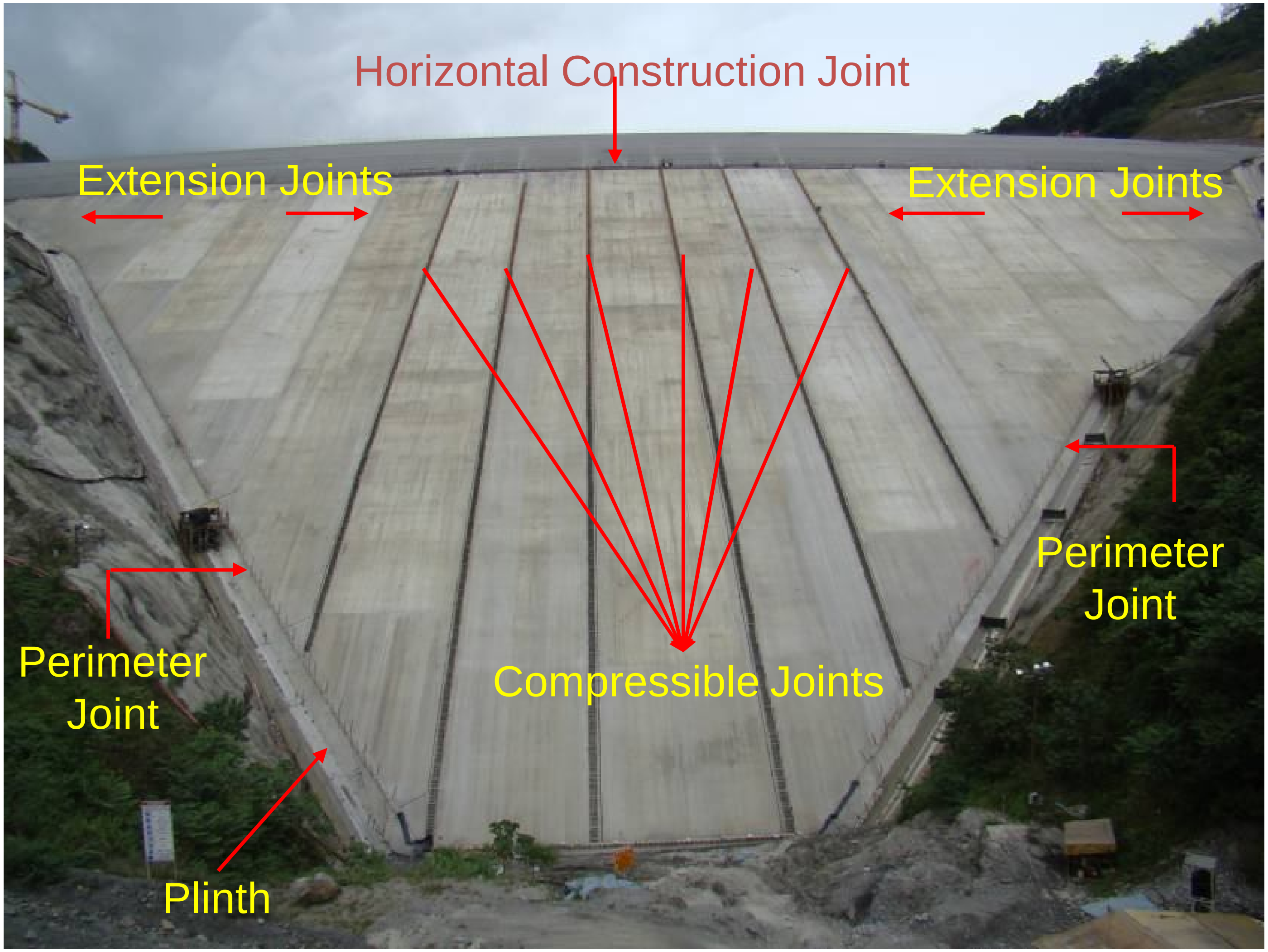
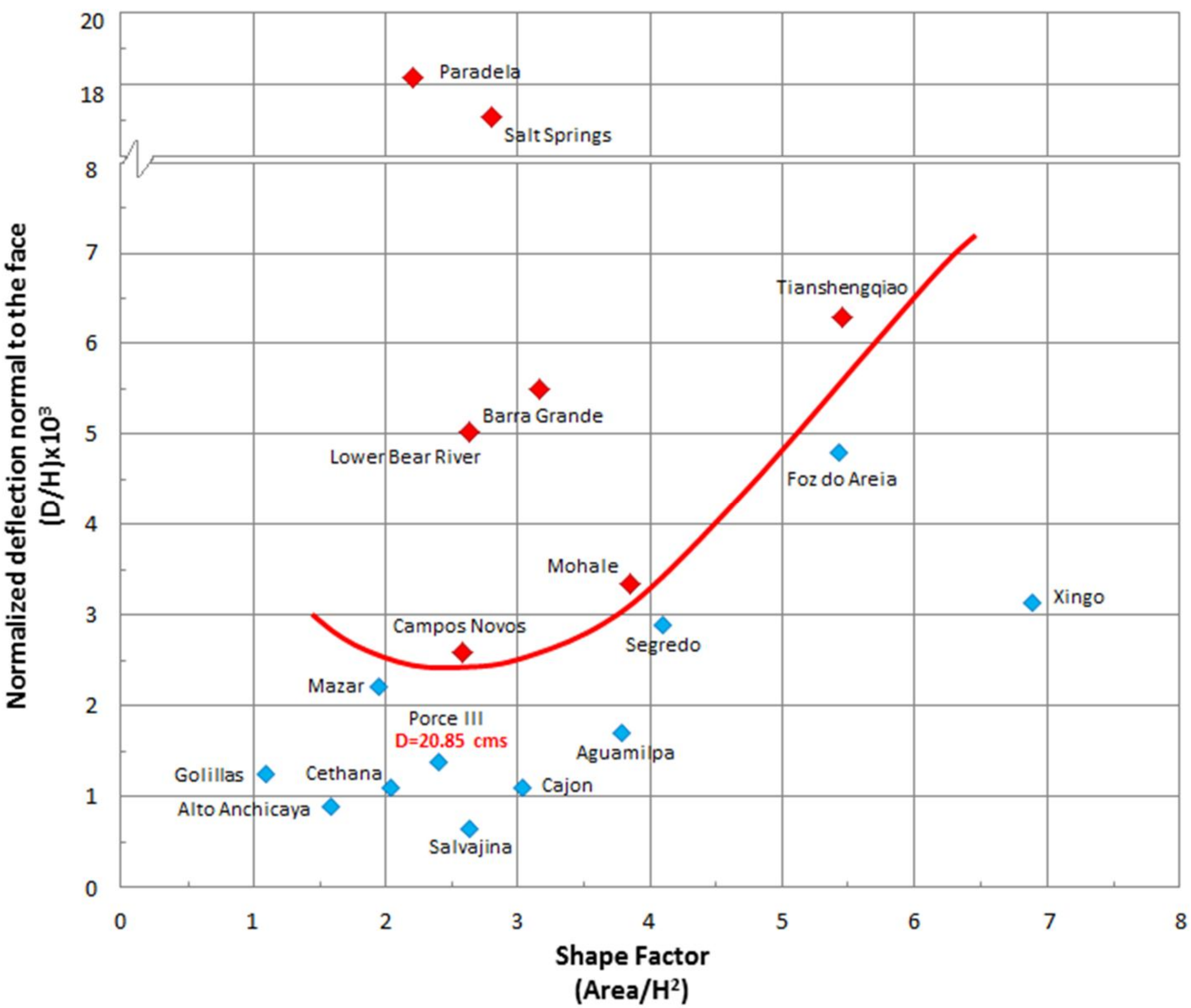
CONCRETE FACE



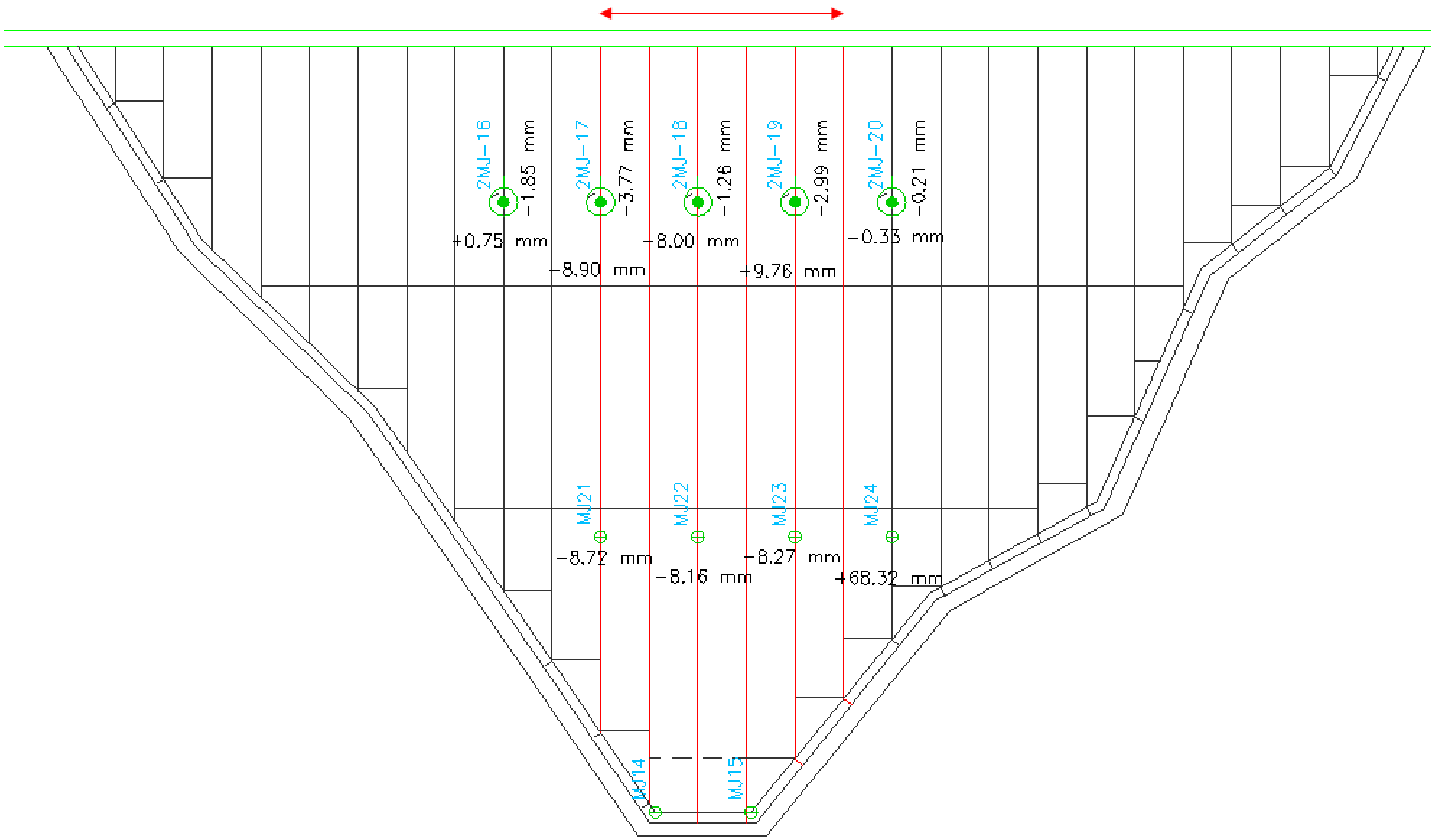
CONCRETE FACE – COMPRESSIBLE JOINTS

CONCRETE FACE – COMPRESSIBLE JOINTS



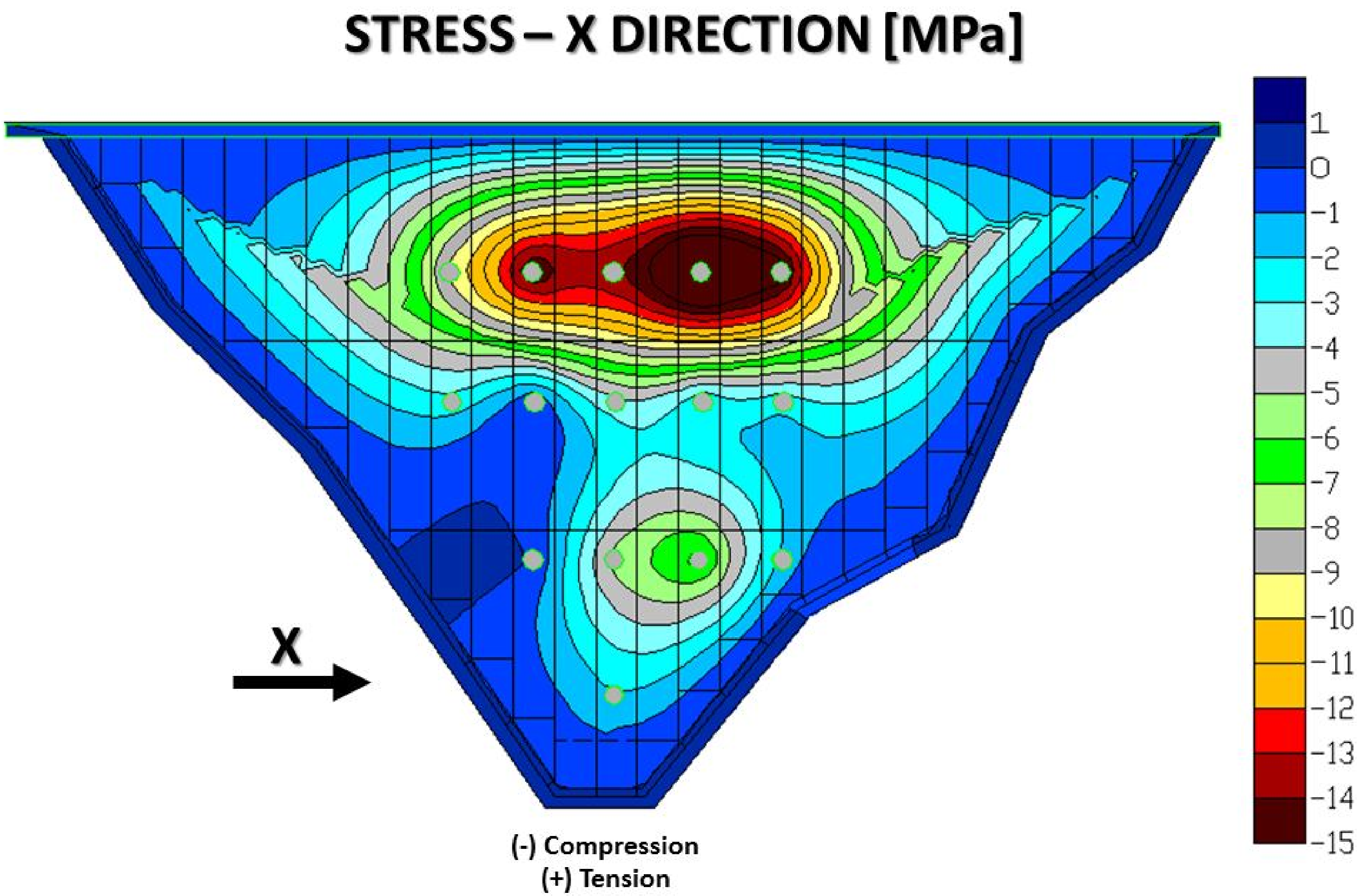
Paradela
Salt Springs
Tianshengqiao
Barra Grande
Lower Bear River
Foz do Areia
Mohale
Xingo
Campos Novos
Segredo
Mazar
Porce III
D=20.85 cms
Aguamilpa
Golillas
Cethana
Cajon
Alto Anchicaya
Salvajina
Normalized deflection normal to the face
(D/H)x10³
Shape Factor
(Area/H²)
0
1
2
3
4
5
6
7
8
18
20

JOINT MOVEMENTS

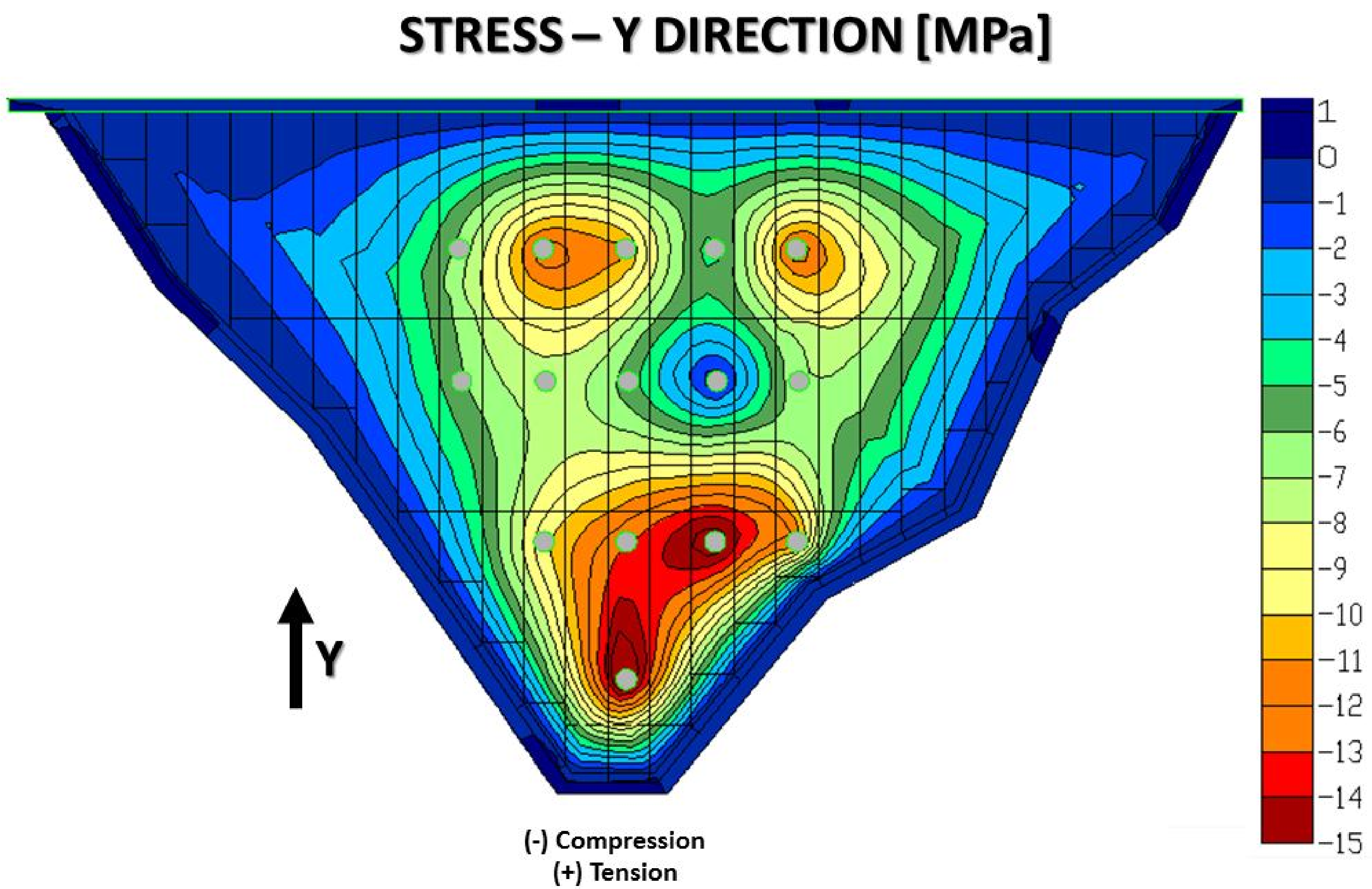


(+): Opening of the Join
(-): Closing of the Join

INSTRUMENTATION RESULTS



INSTRUMENTATION RESULTS

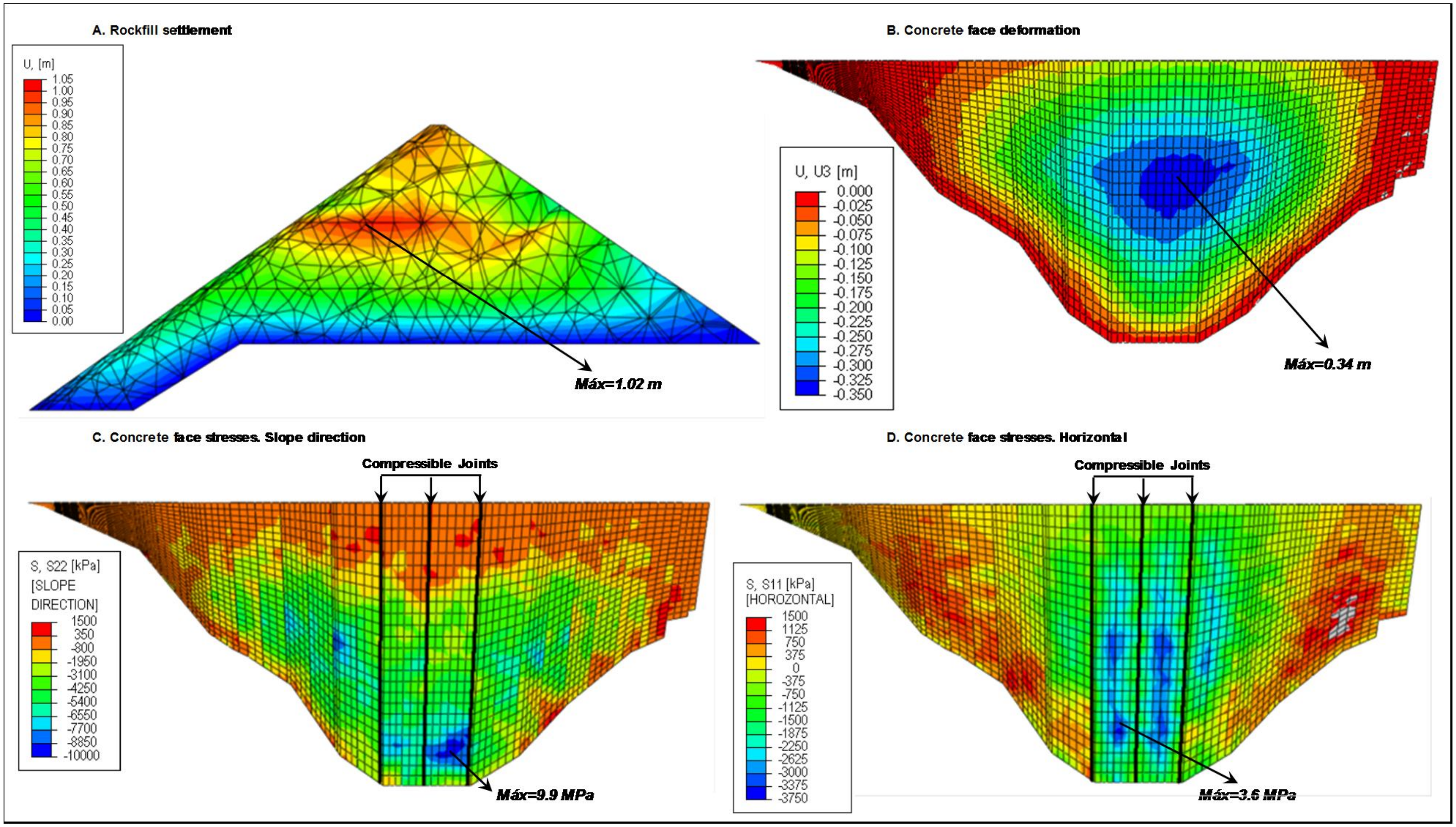


INGETEC
INGENIEROS CONSULTORES

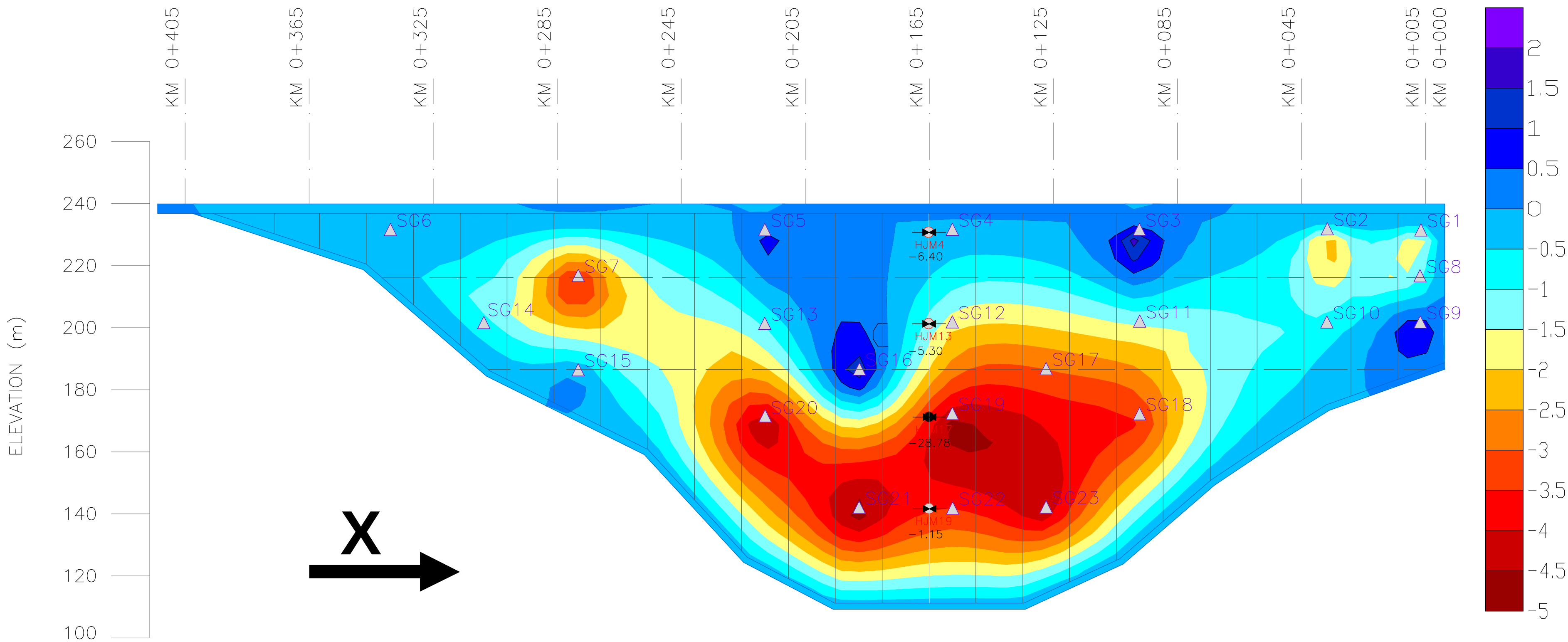
PORCE III

PORCE III

NUMERICAL ANALYSIS

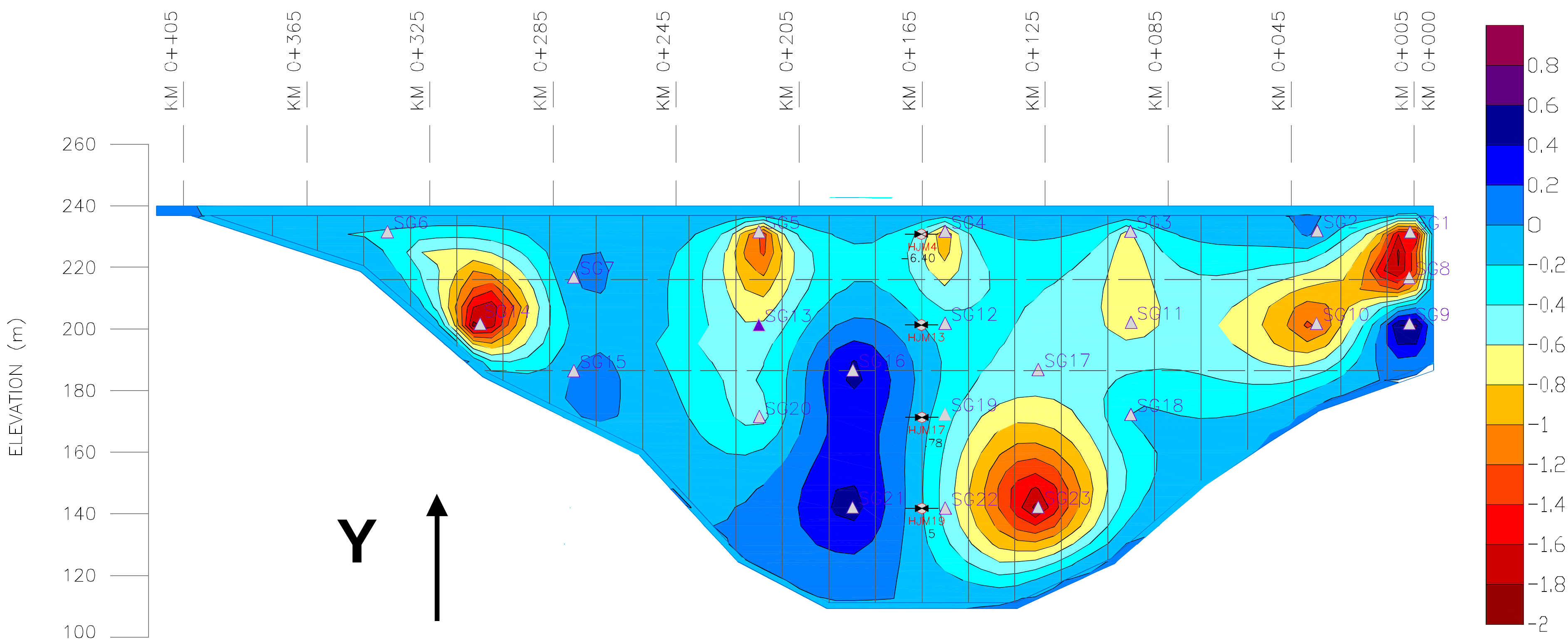


STRESS – X DIRECTION [MPa]



INGETEC
INGENIEROS CONSULTORES

STRESS – Y DIRECTION [MPa]



INGETEC INGENIEROS CONSULTORES

YEDIGOZE

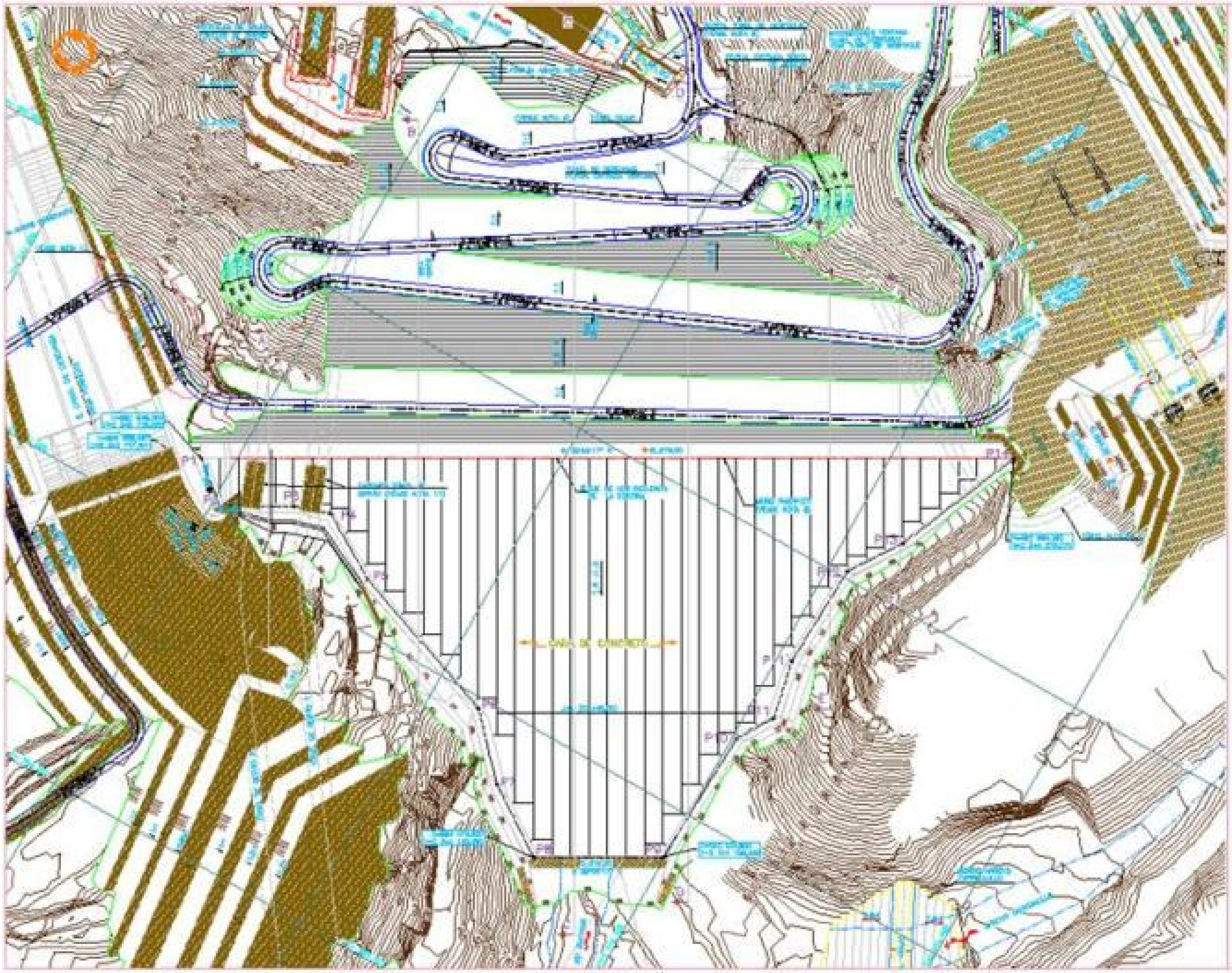
MAZAR DAM -ECUADOR

MAZAR DAM

MAZAR COMPRESSIBLE JOINT

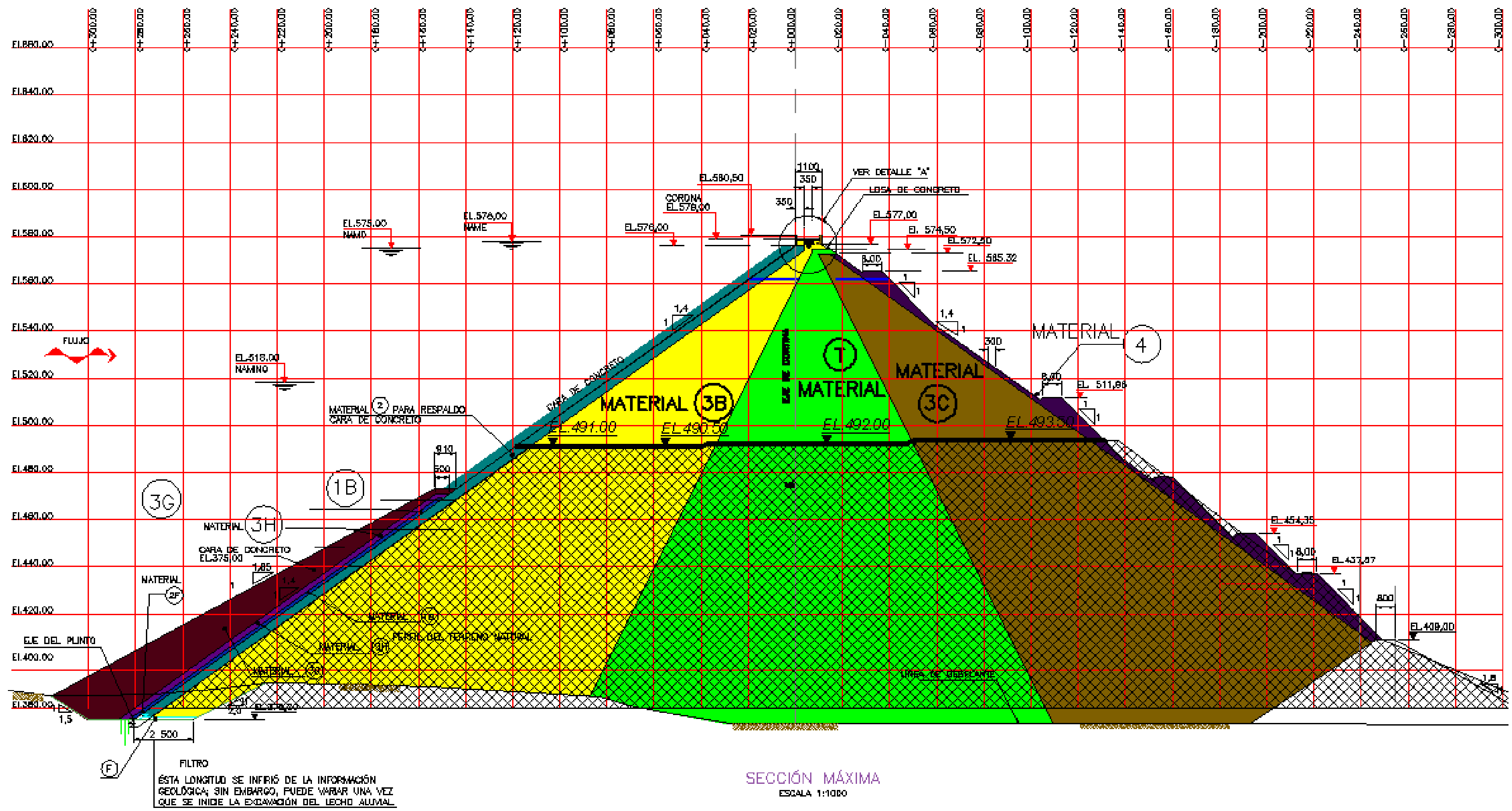
MAZAR COMPRESSIBLE JOINT

LA YESCA DAM



INGETEC INGENIEROS CONSULTORES

LA YESCA DAM



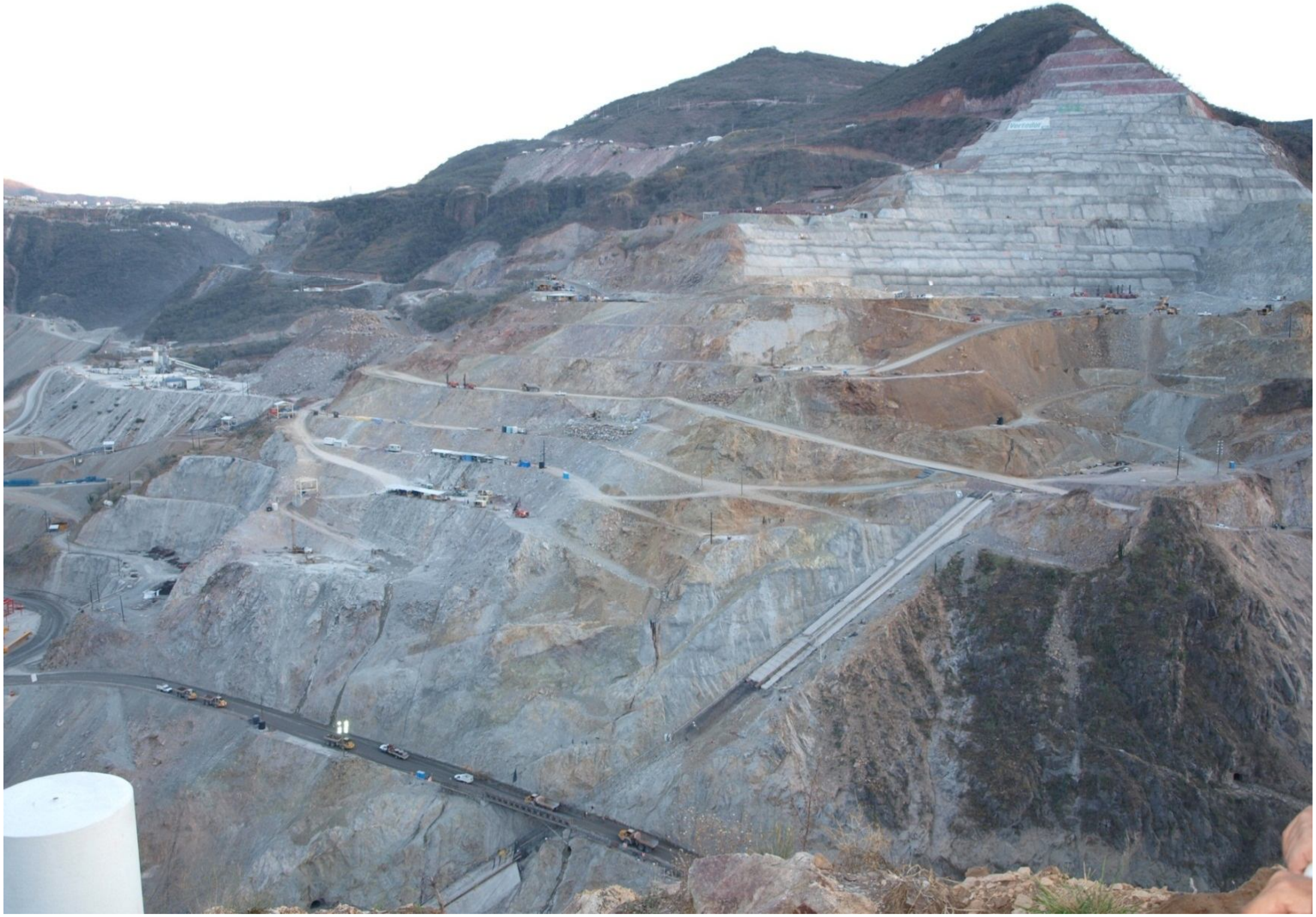
INGETEC INGENIEROS CONSULTORES

LA YESCA - DAM

LA YESCA - DAM

LA YESCA - DAM

LA YESCA - DAM



LA YESCA - DAM

INGETEC INGENIEROS CONSULTORES

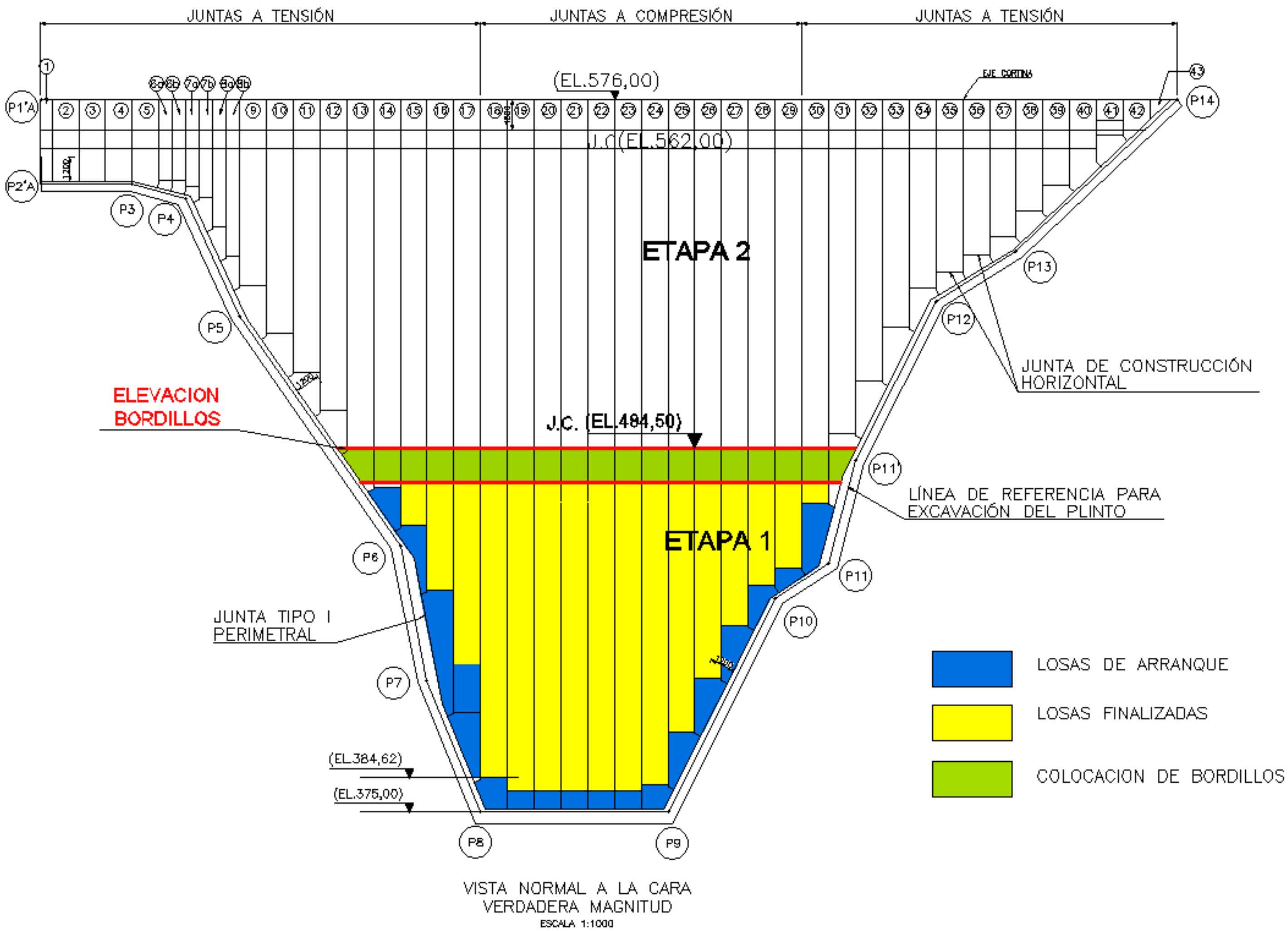
LA YESCA DAM

LA YESCA - DAM

GENERAL VIEW OF THE DAM MAY-2012

LA YESCA - DAM

LA YESCA - DAM



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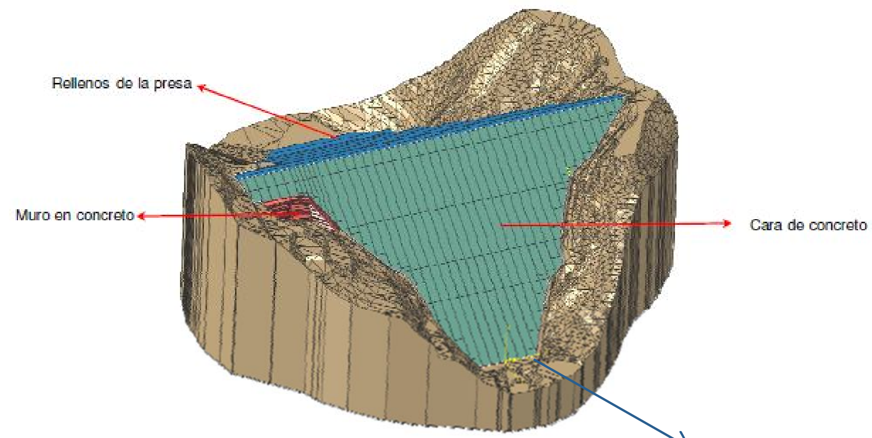
LA YESCA
STATIC AND DYNAMIC ANALYSIS

Objective of the analysis:

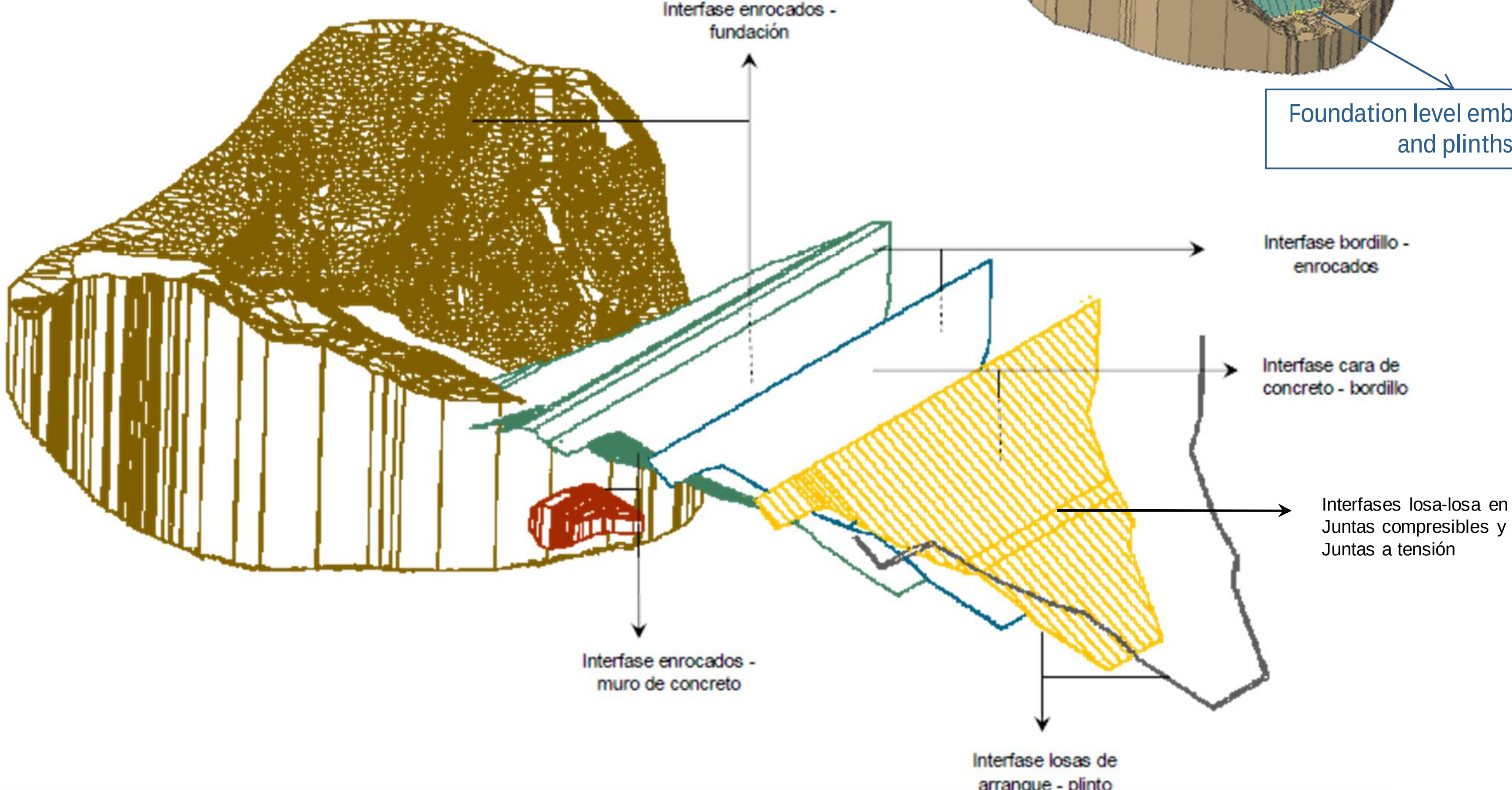
- Estimate Stress – strain – deformations behavior of the concrete face
- Deformation of the fill
- Joint displacements of the joints
- Maximum deformations at the crest of the dam
- Dynamic behavior of the dam

1. Geometry of the model

1.1. Interfaces

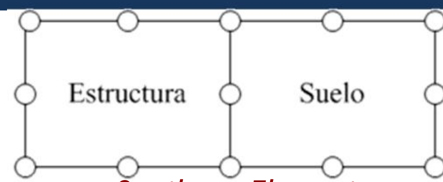


Foundation level embedded fills and plinths

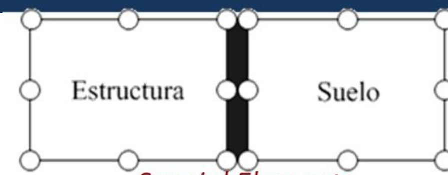


2. Constitutive Models

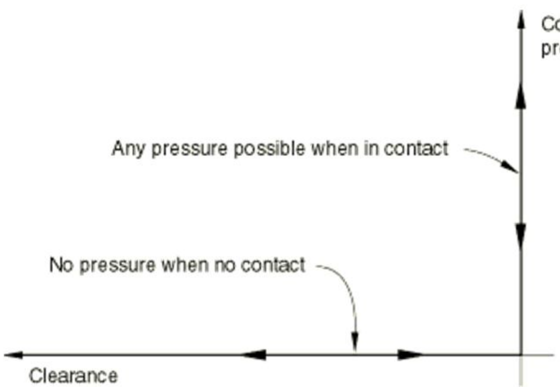
2.3. Interface behavior



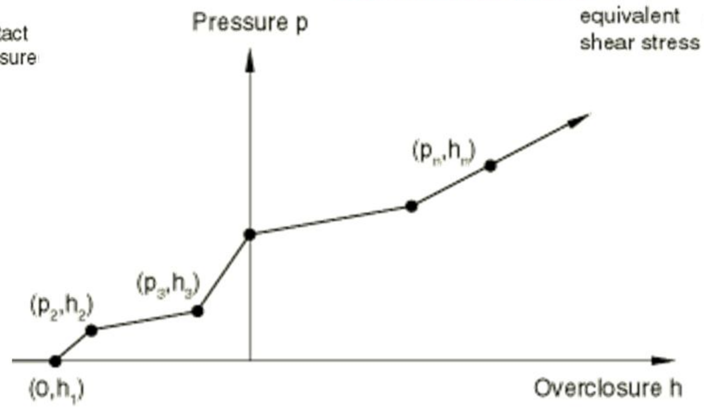
Continuos Elements



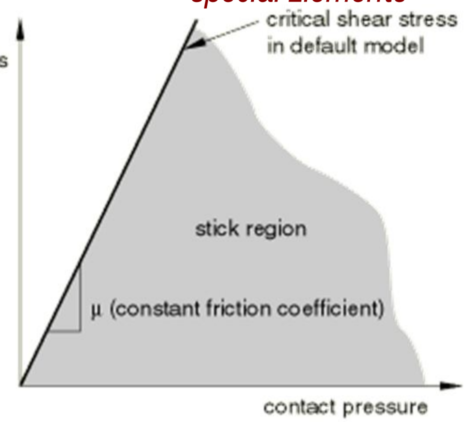
Special Elements



Normal Compression - Rigid



Normal Compression - Constant Rigidity



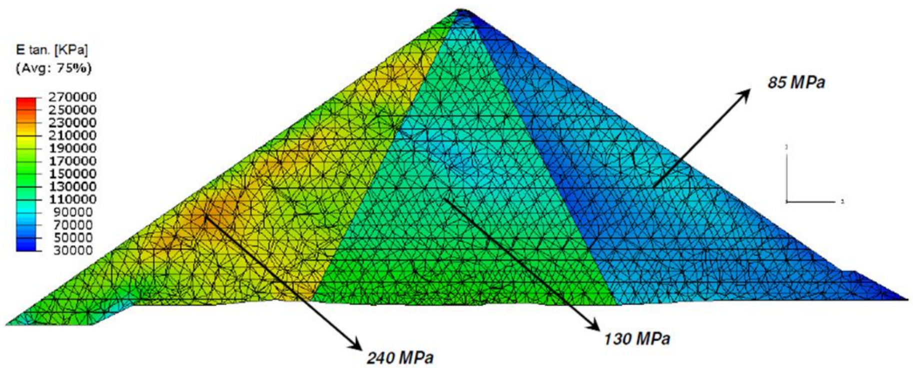
Tangential Compression – Friction Model

Interfase	Tipo de interfase	Comportamiento normal	Comportamiento tangencial
Losa-Losa Juntas compresibles	Elementos especiales de contacto	Rigidez constante de 100MPa hasta una deformación unitaria del 60%	Coeficiente de fricción: 0.4
Losa-Losa Juntas a tensión	Elementos especiales de contacto	Comportamiento rígido	Coeficiente de fricción: 0.85
Losa-Plinto	Elementos especiales de contacto	Comportamiento rígido	Coeficiente de fricción: 0.85
Losas-Bordillo	Elementos especiales de contacto	Comportamiento rígido	Coeficiente de fricción: 0.85
Bordillo-Enrocado	Elementos continuos	Compatibilidad de deformaciones	
Enrocado-Muro	Elementos especiales de contacto	Comportamiento rígido	Coeficiente de fricción: 1.0

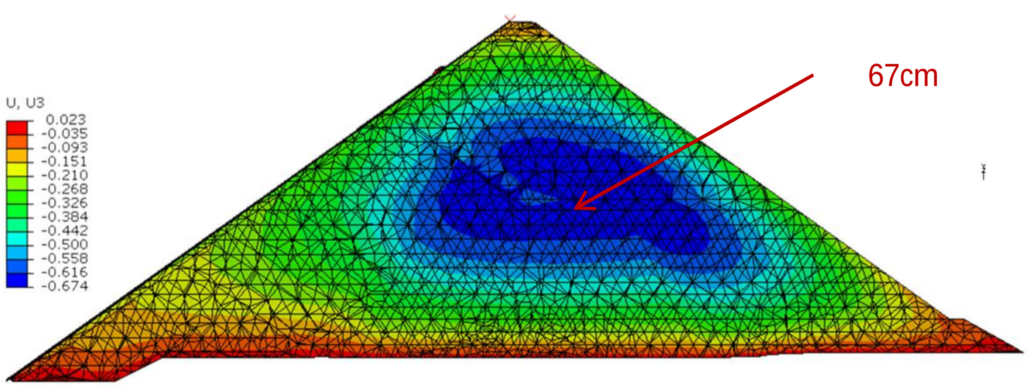
3. Results

3.1. Analysis under static conditions

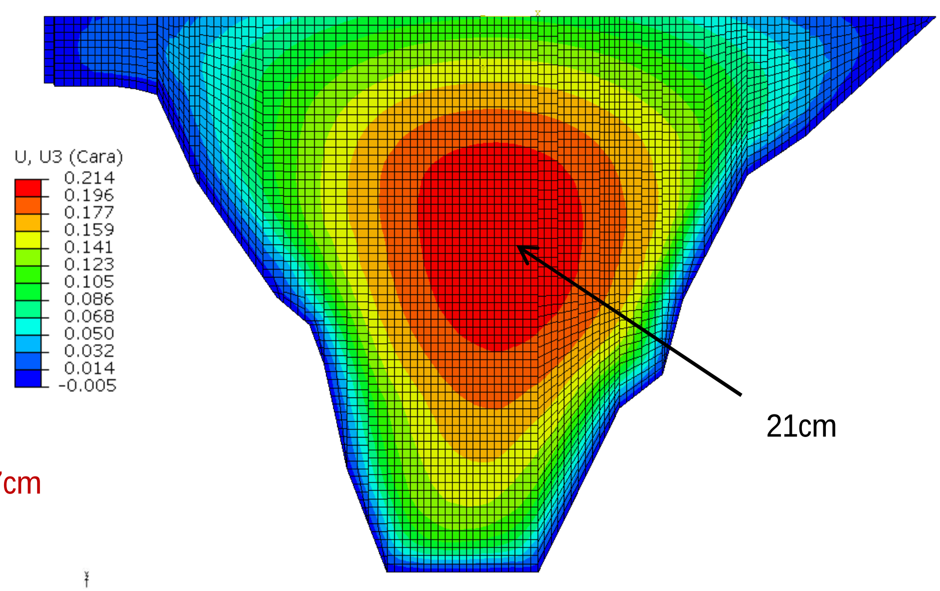
a. Deformation Modulus at the end of construction



b. Settlement of the fill



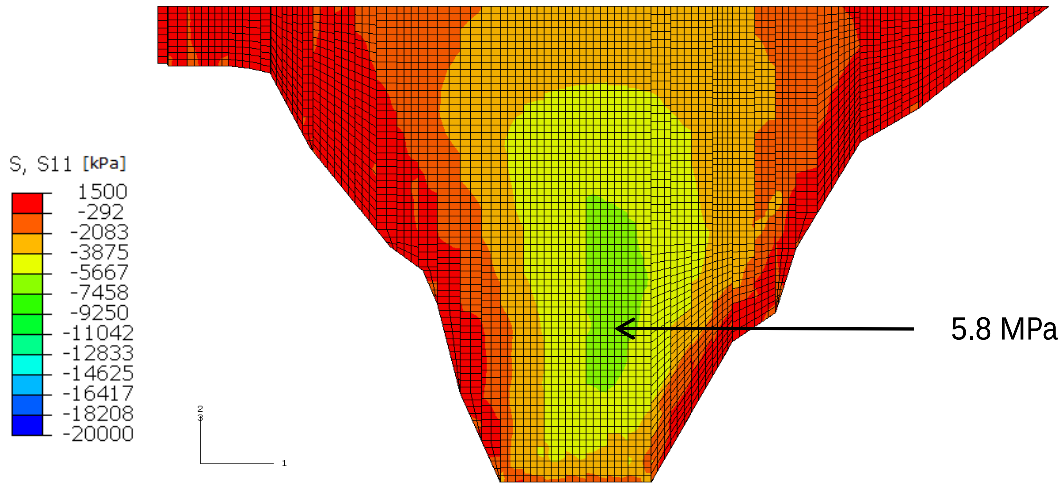
c. Displacement normal to the face



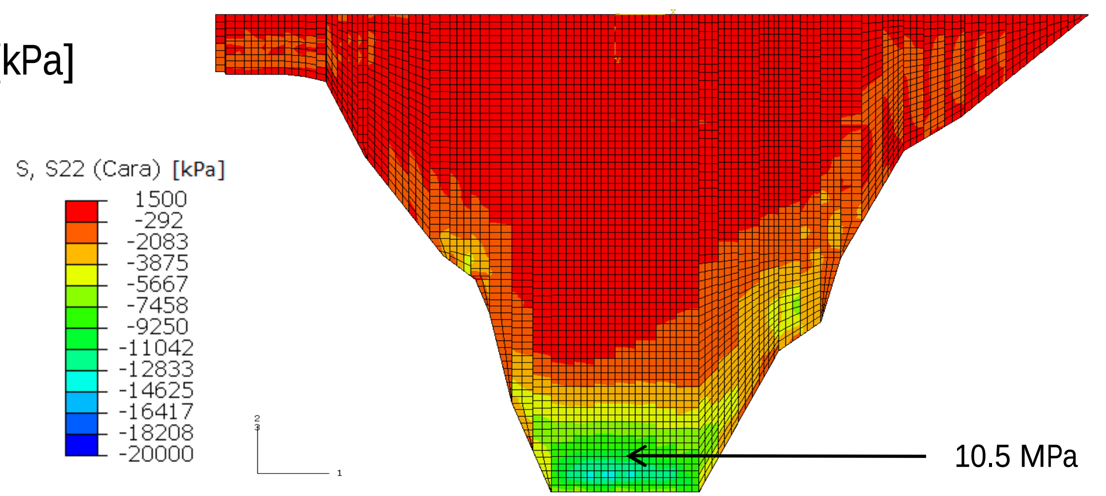
3. Results

3.1. Analysis under static conditions

c. Stresses in the horizontal direction (S11) [kPa]



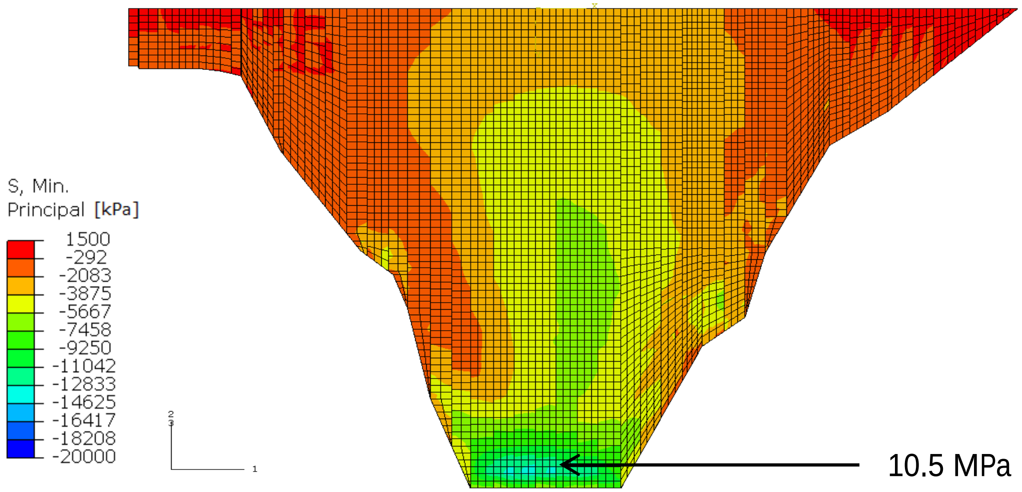
d. Stresses along the slope [kPa]



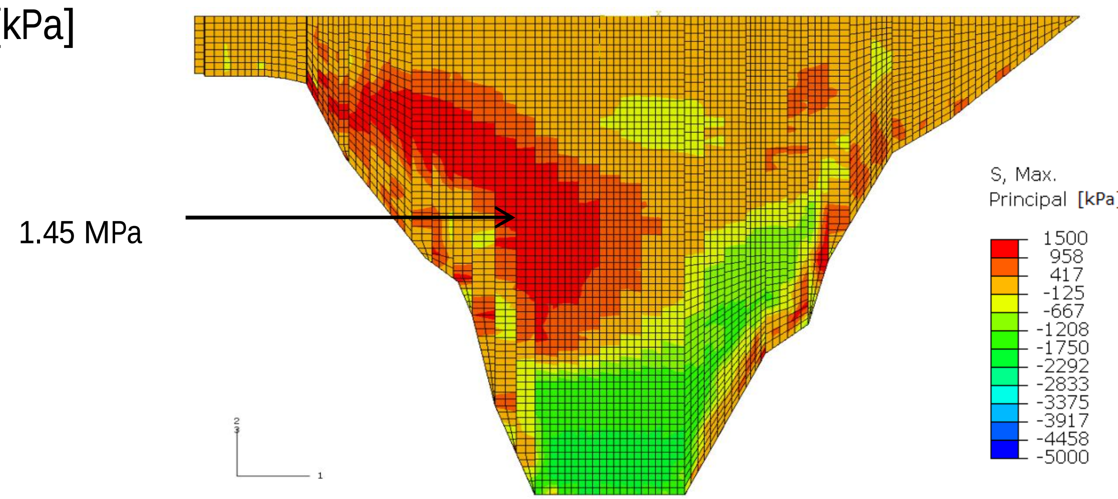
3. Results

3.1. Analysis under static conditions

e. Minor principal stresses (compression) [kPa]



f. Mayor principal stresses [kPa]



LA YESCA - DAM

LA YESCA - DAM

LA YESCA - DAM

LA YESCA - DAM

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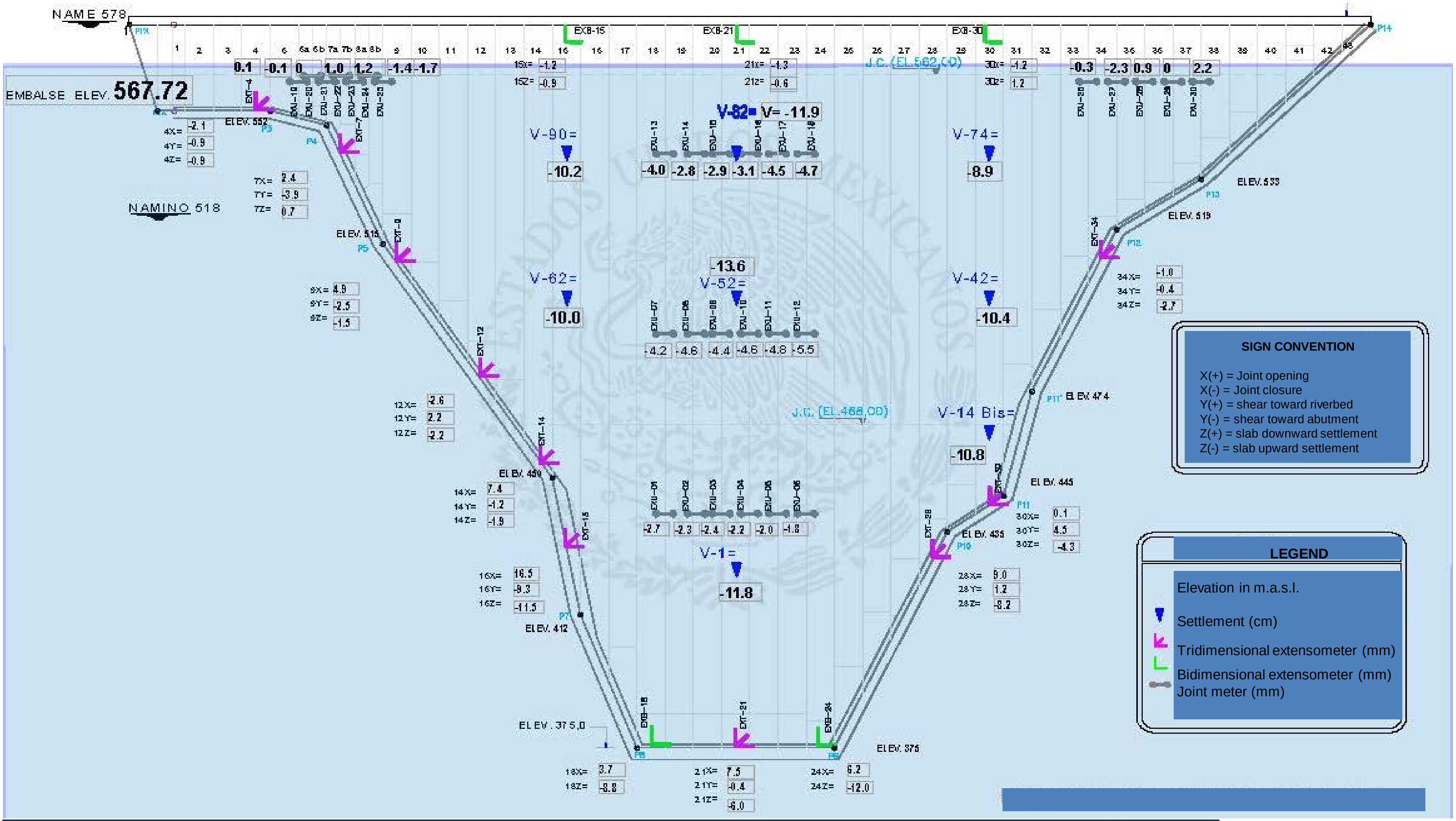
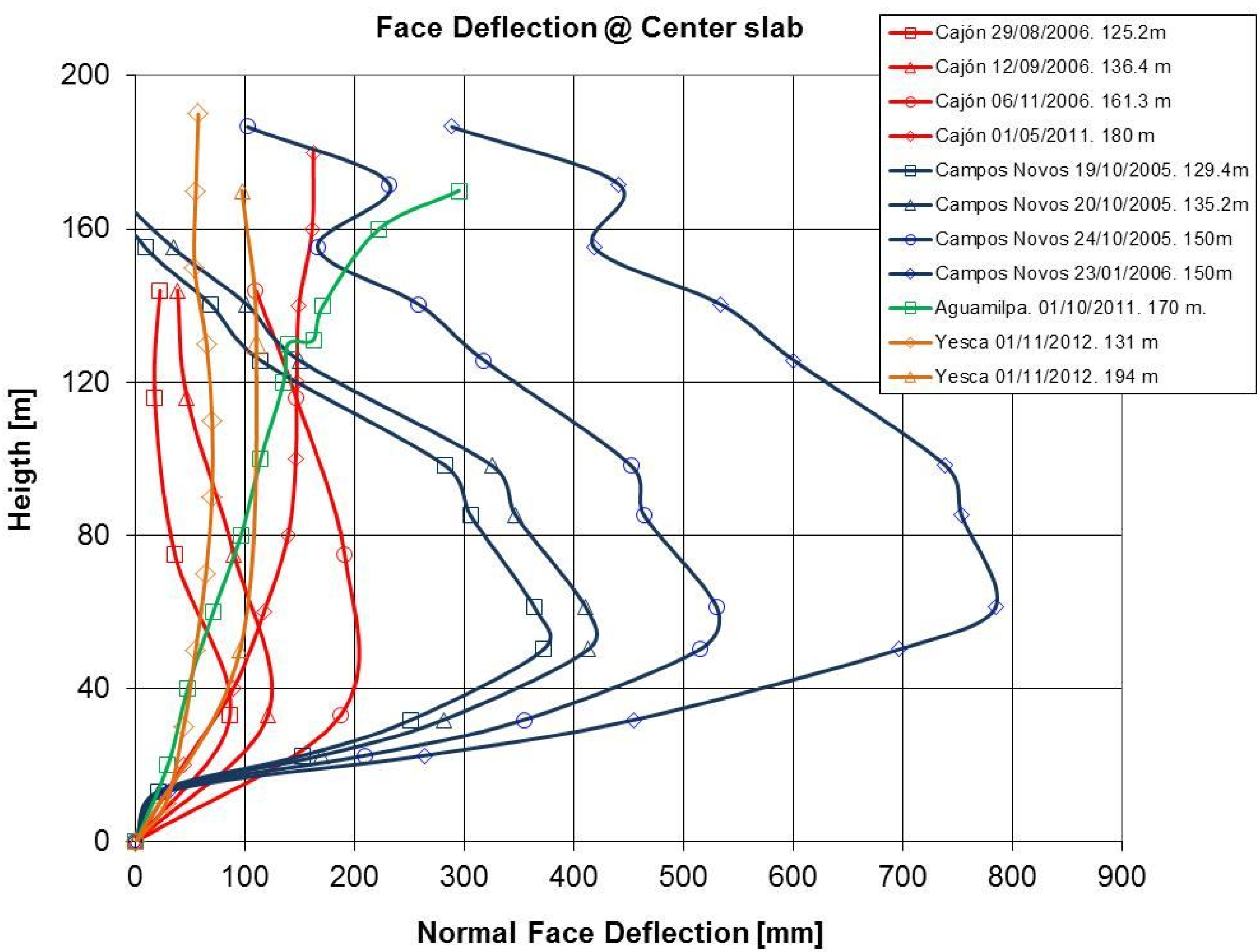
La Yesca Dam: Face Slab deformation (Sep 30/2013)





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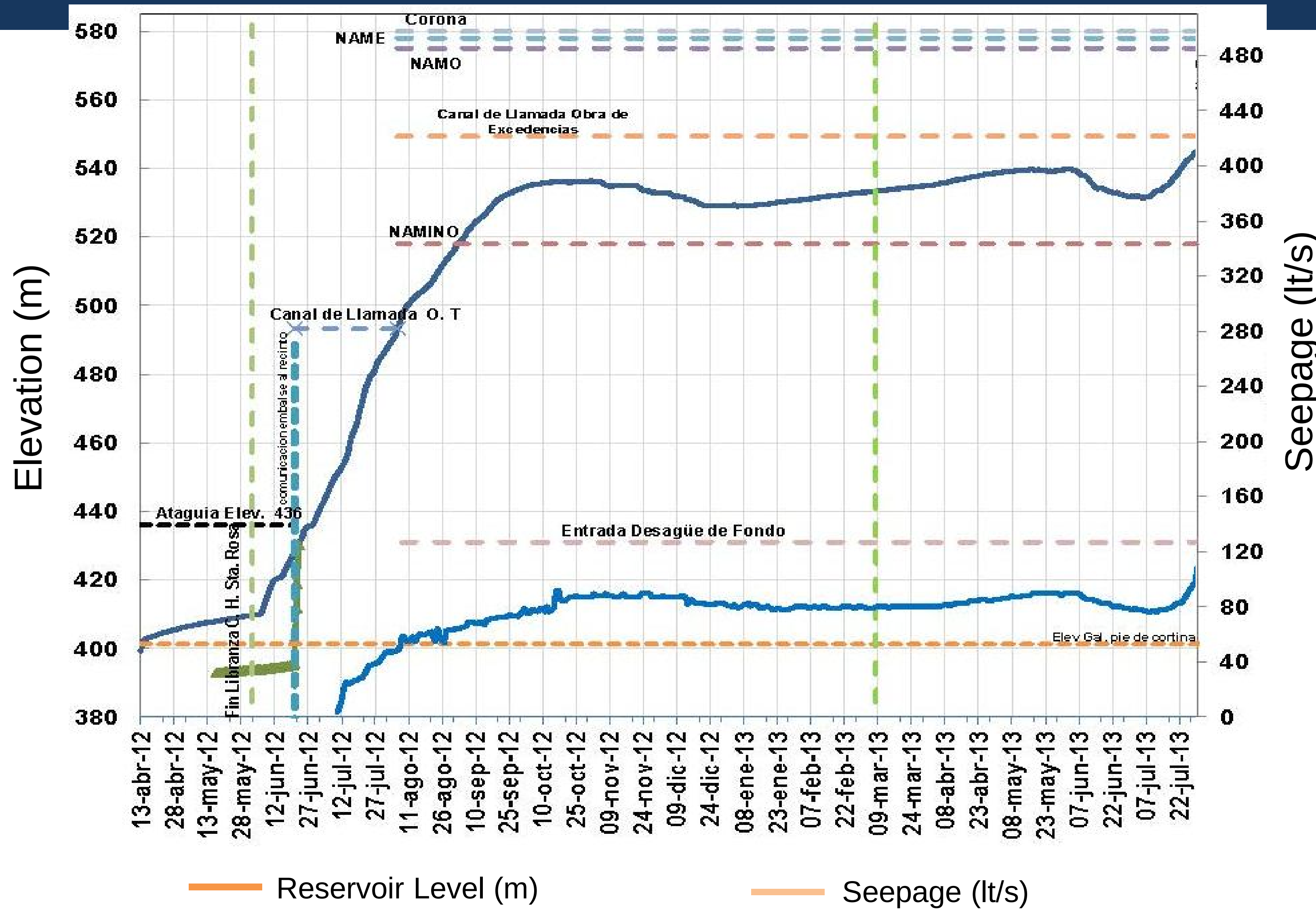
LA YESCA DAM



Corona
NAME
NAMO
Canal de Llamada Obra de Excedencias
NAMINO
Canal de Llamada O. T
comunicacion embalse al recinto
Ataguia Elev. 436
Fin Libranza C. H. Sta. Rosa
Entrada Desagüe de Fondo
Elev Gal. pie de cortina
Elevation (m)
Seepage (lt/s)
580
560
540
520
500
480
460
440
420
400
380
480
440
400
360
320
280
240
200
160
120
80
40
0
13-abr-12
28-abr-12
13-may-12
28-may-12
12-jun-12
27-jun-12
12-jul-12
27-jul-12
11-ago-12
26-ago-12
10-sep-12
25-sep-12
10-oct-12
25-oct-12
09-nov-12
24-nov-12
09-dic-12
24-dic-12
08-ene-13
23-ene-13
07-feb-13
22-feb-13
09-mar-13
24-mar-13
08-abr-13
23-abr-13
08-may-13
23-may-13
07-jun-13
22-jun-13
07-jul-13
22-jul-13
Reservoir Level (m)
Seepage (lt/s)

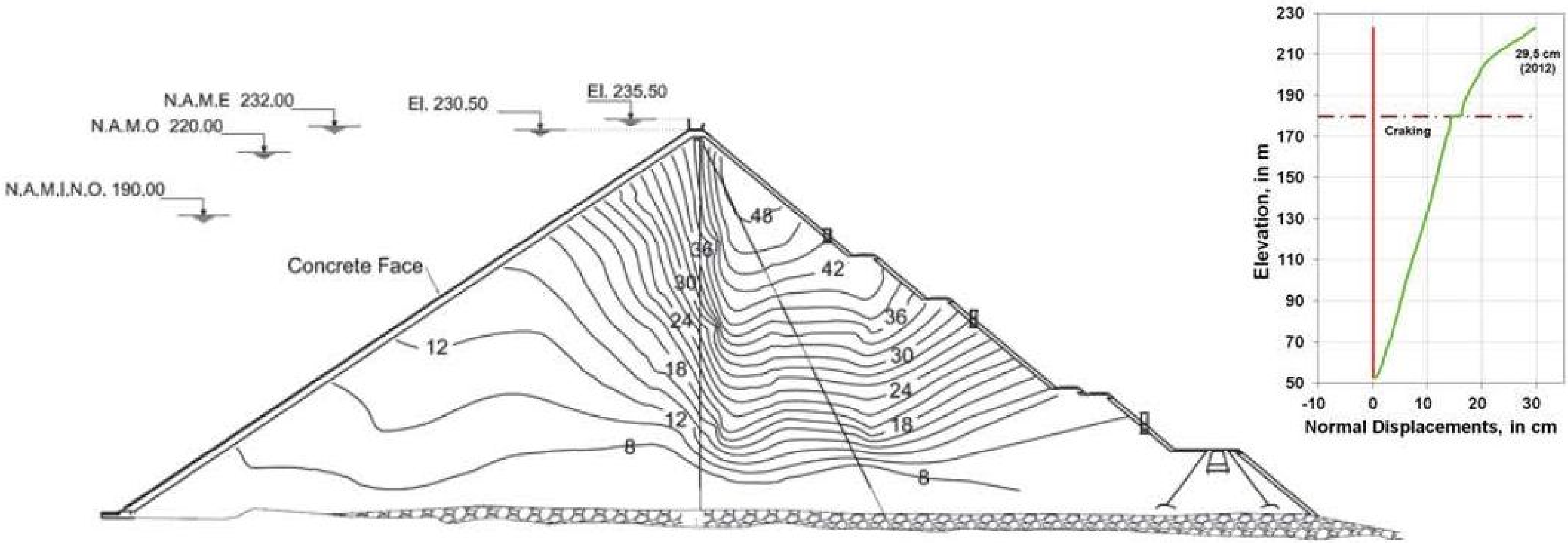


Figure 6. Settlement contours at Aguamilpa Dam as of October 2011 (first filling is included)

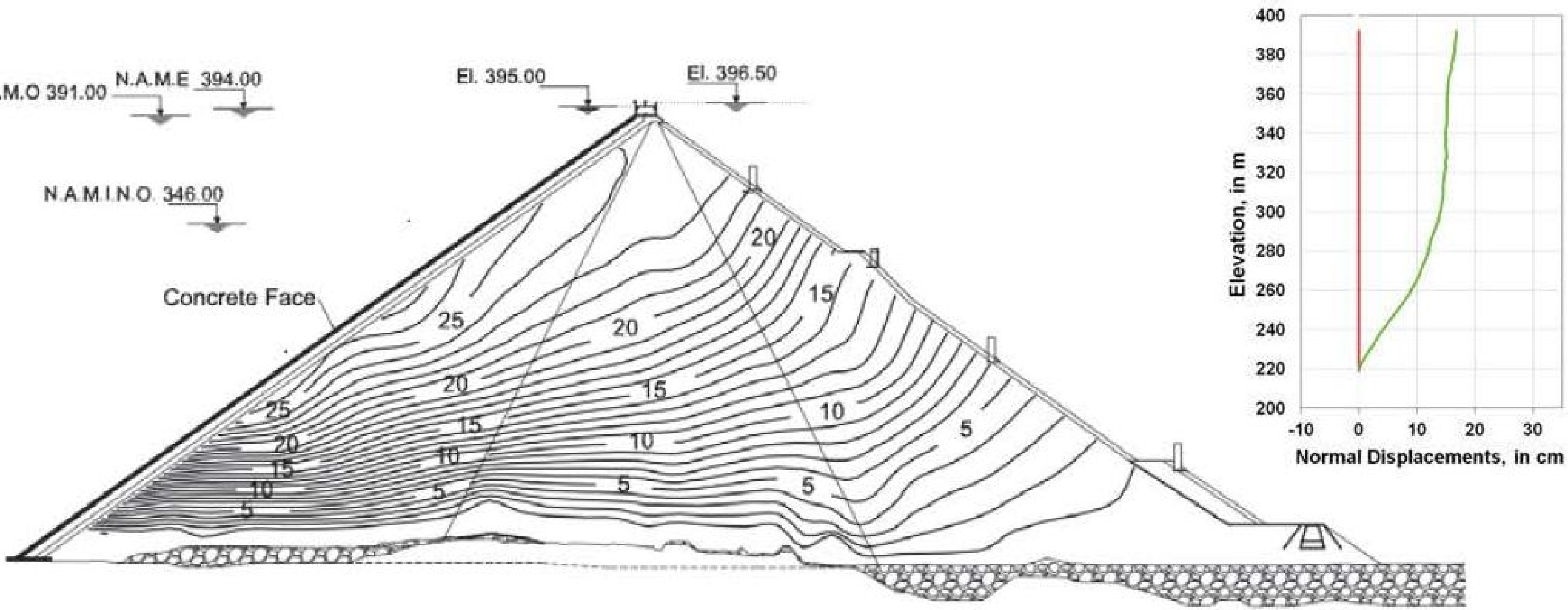
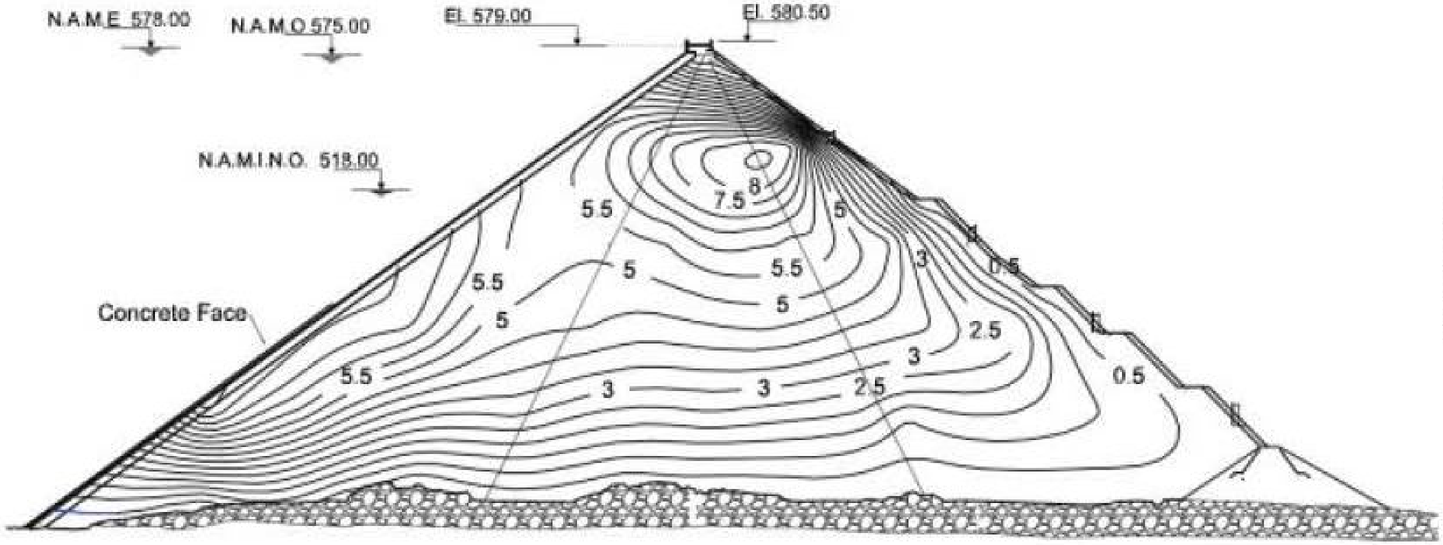


Figure 7. Settlement contours as of may 2011 measured at El Cajón Dam (first filling is included).



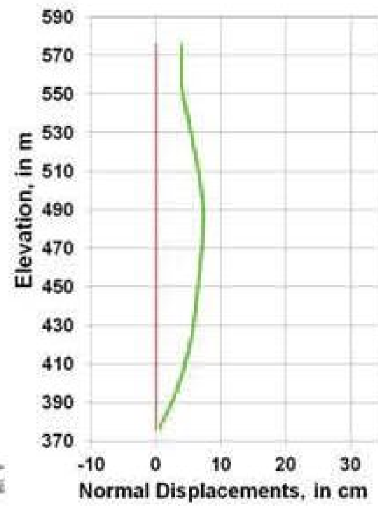


Figure 8. Settlement contours as of November 2012 measured at La Yesca Dam (first filling).

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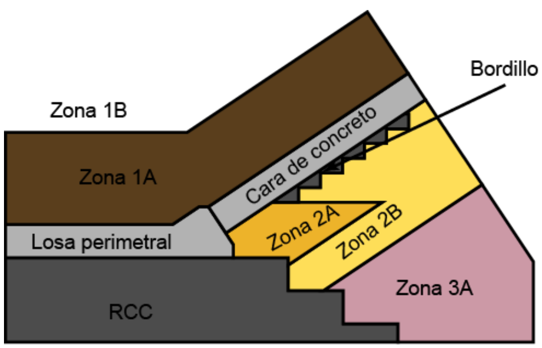
SOGAMOSO DAM

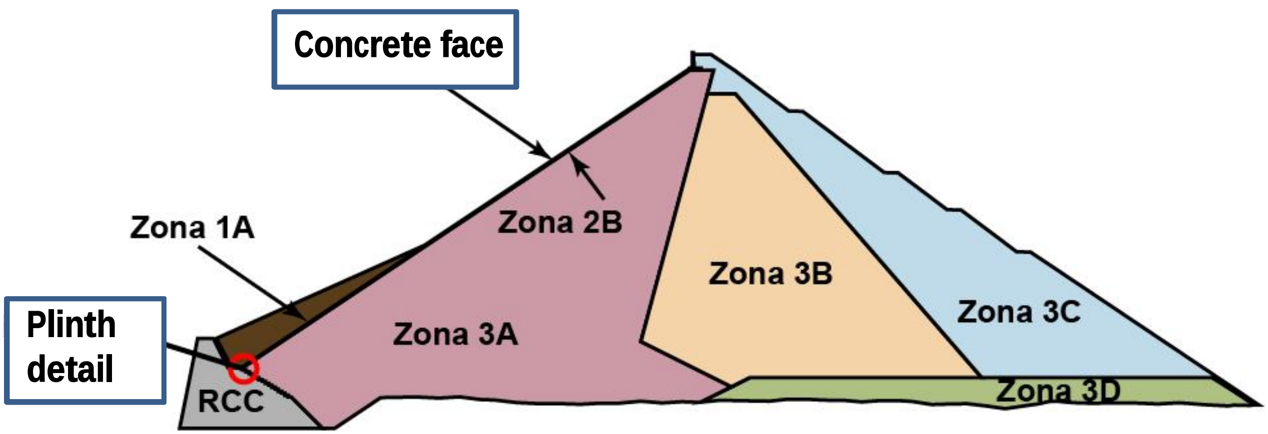
- 190 meters tall.
- 345 meters wide.
- 8,5 million cubic meters.

DAM ZONES

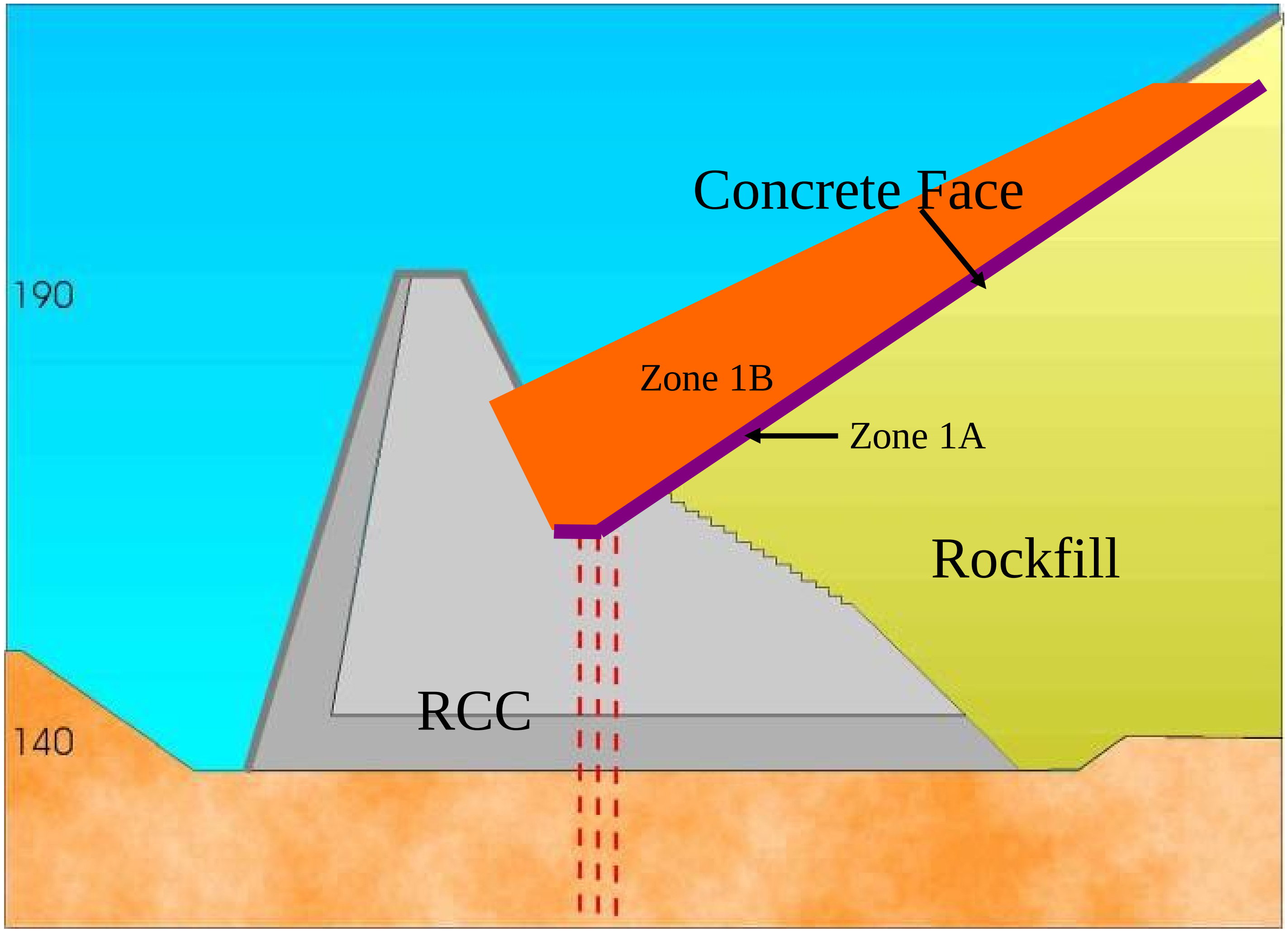
Zone	Volume [m³]	Description
2A	31.000	Processed gravel
2B	289.600	Processed gravel
3A	4’103.600	Natural gravel
3B	2’307.300	Spillway rockfill
3C	1’293.500	Spillway rockfill
3D	128.000	Filter material



Plinth Detail



CONCRETE FACE AND COFFERDAM

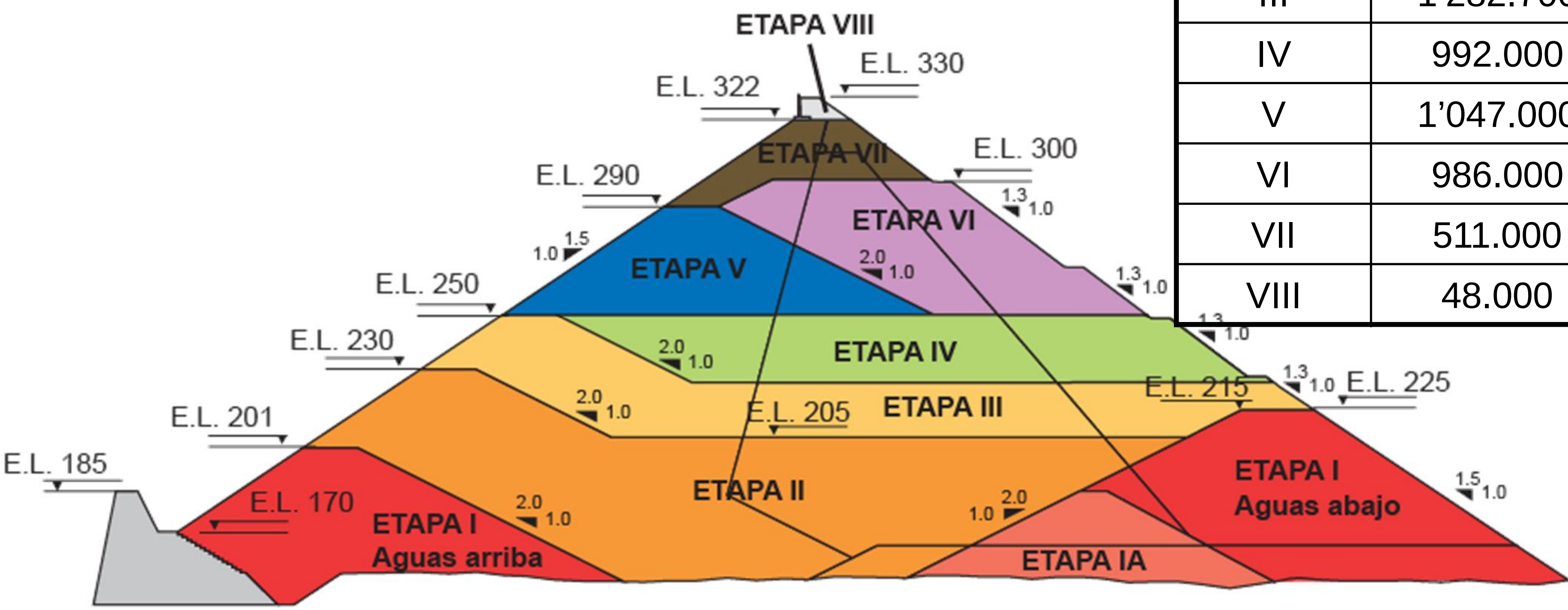


SOGAMOSO DAM

CONSTRUCTION STAGES OF THE DAM

Duration: 22.5 months
Total Volume: 8’153.000 m^3
Average efficiency: 362.360 m^3/mes

Stage	Volume [m^3]
I - Down stream	568.300
I - Up stream	793.000
II	1’925.000
III	1’282.700
IV	992.000
V	1’047.000
VI	986.000
VII	511.000
VIII	48.000



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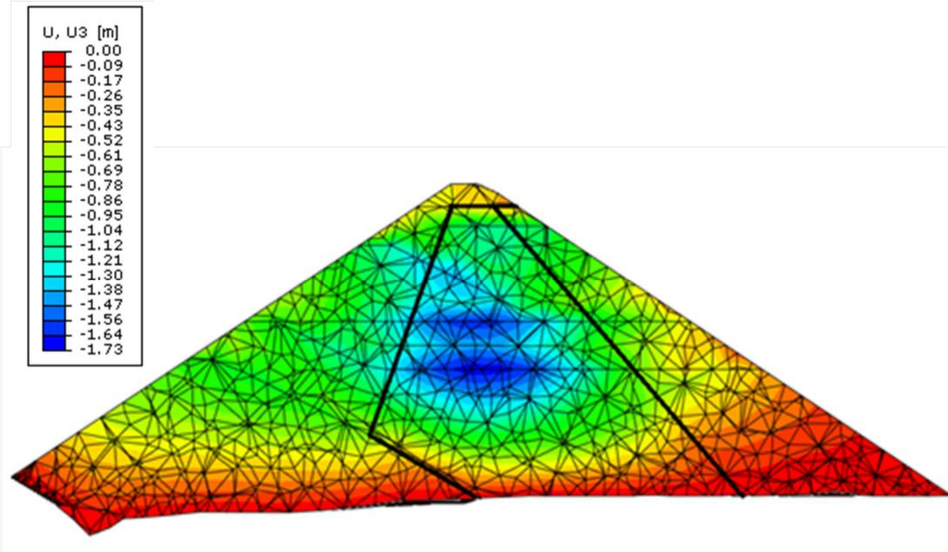
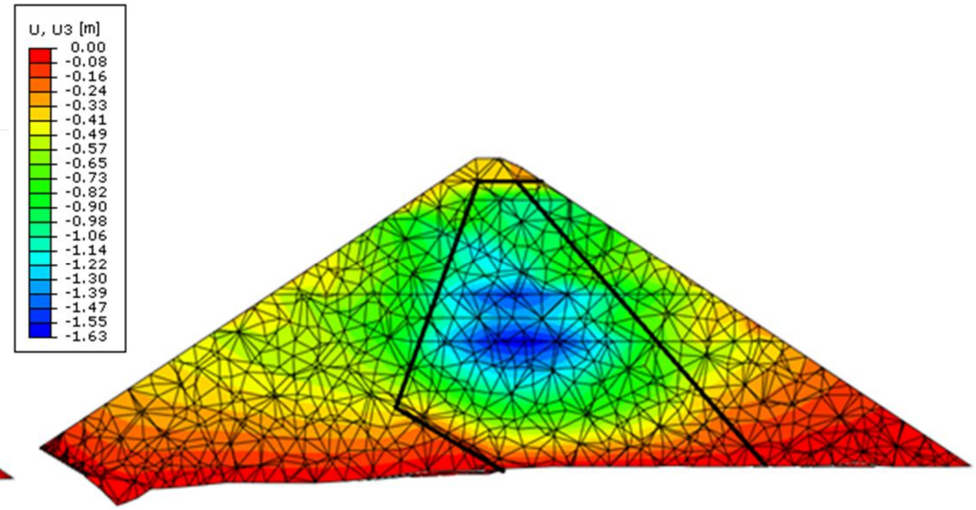
FILL DISPLACEMENTS

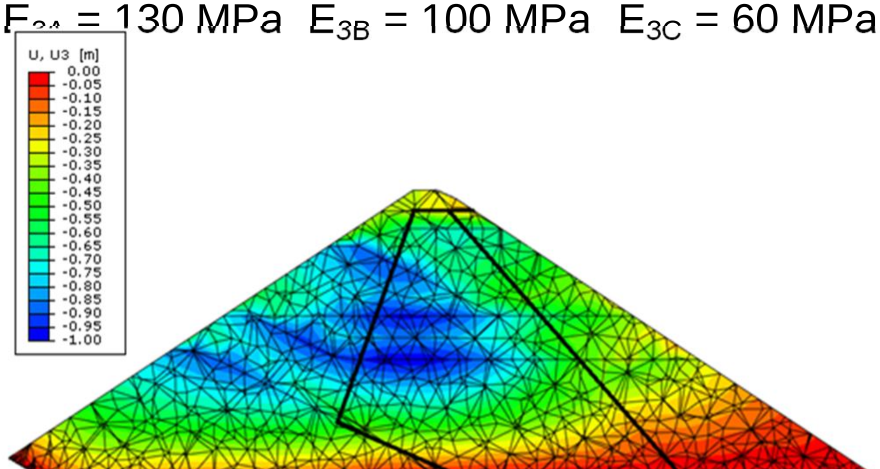
E_{3A} = 130 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



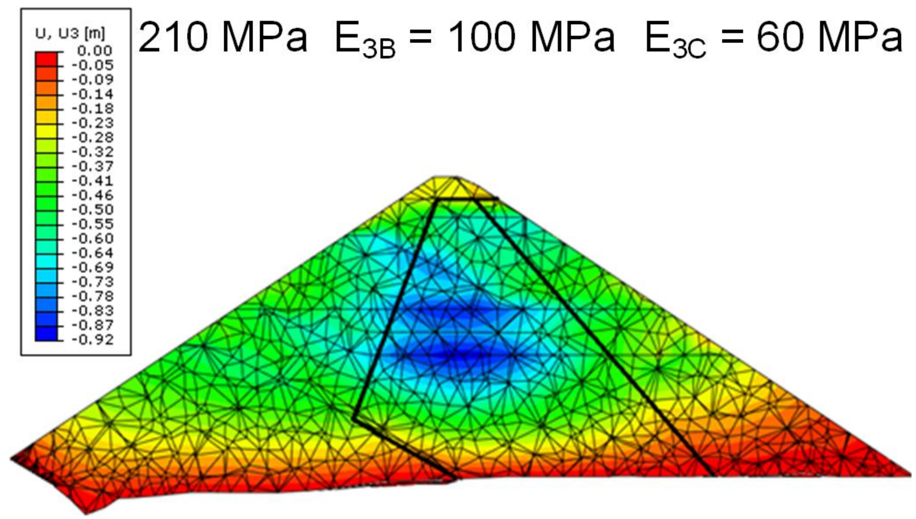
E_{3A} = 210 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



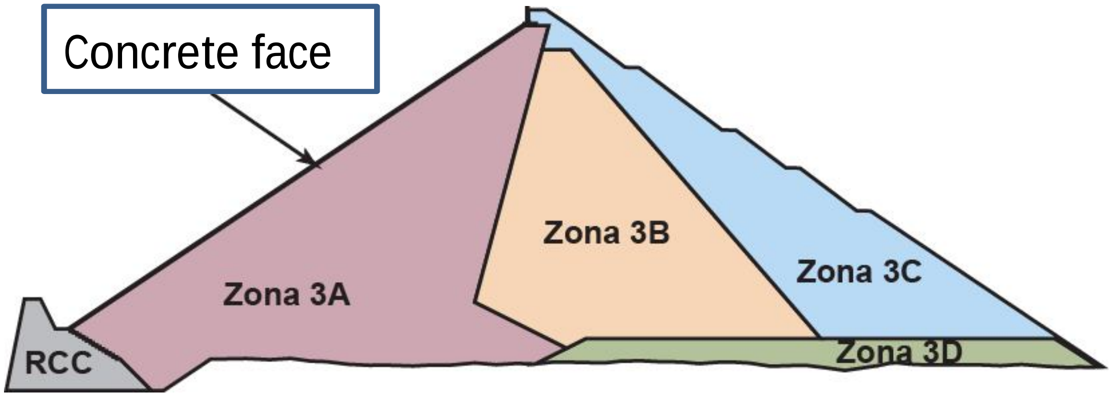
E_{3A} = 130 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa



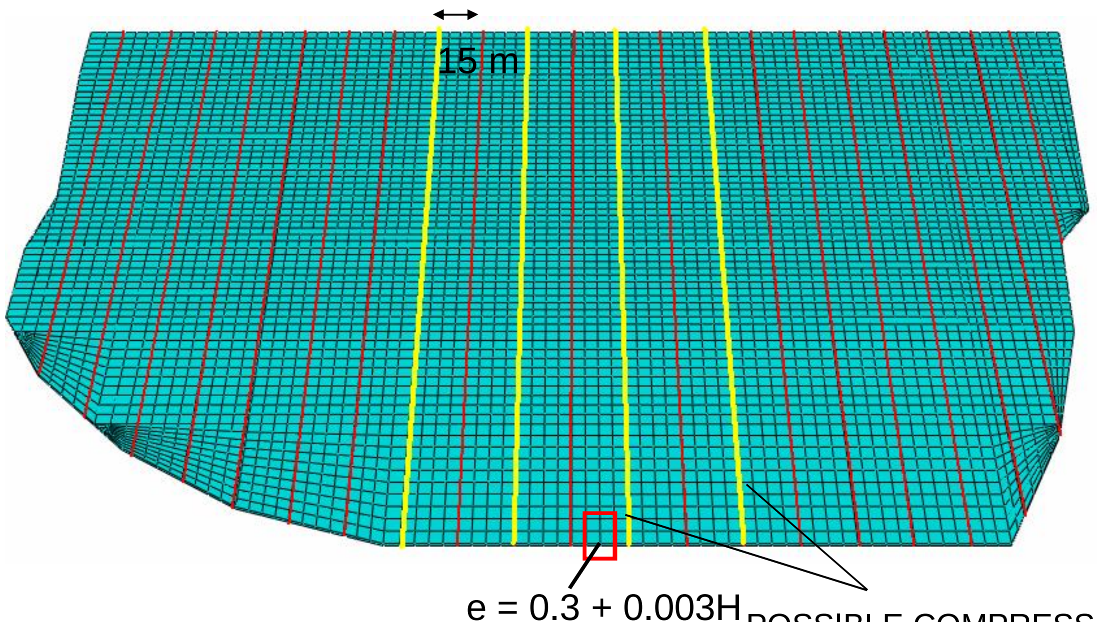
210 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa



FILL ZONES

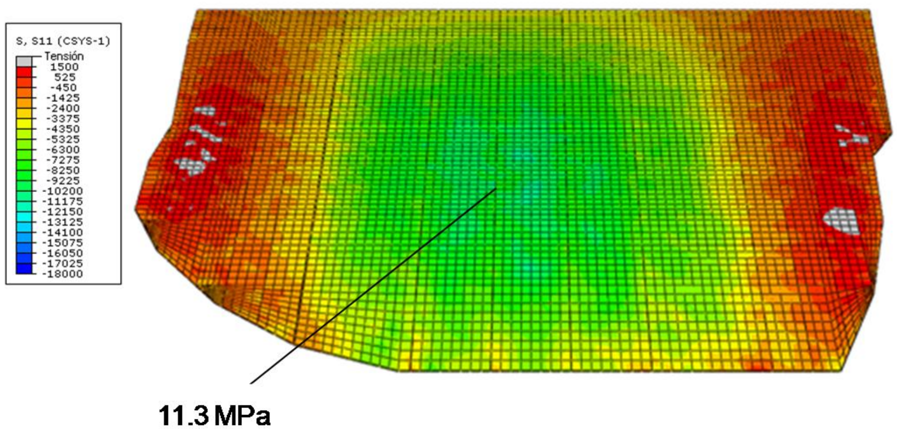


CONCRETE FACE JOINT DISTRIBUTION

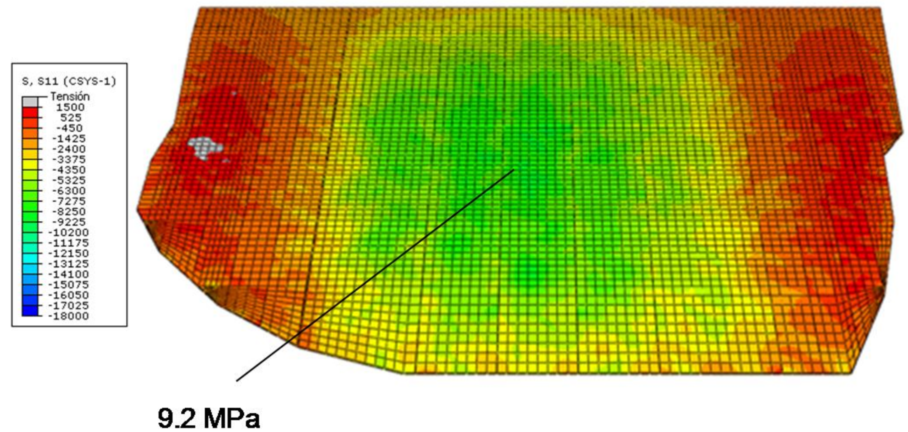


MAXIMUM HORIZONTAL STRESS

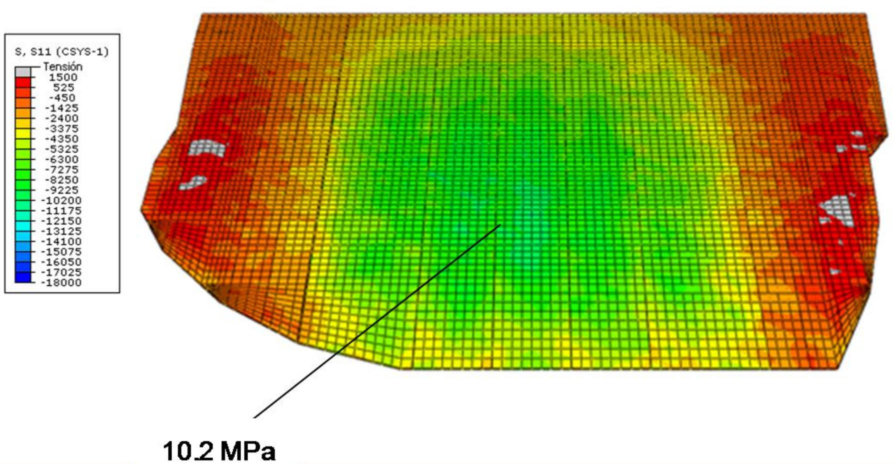
E_{3A} = 130 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



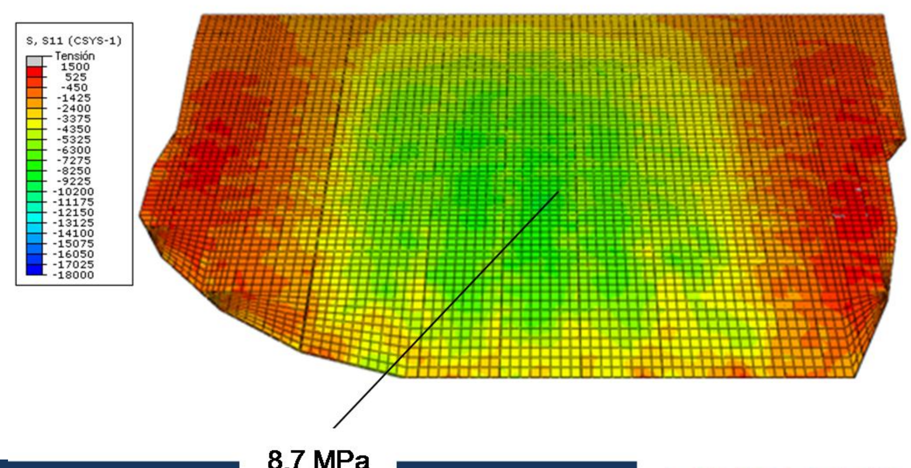
E_{3A} = 210 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



E_{3A} = 130 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa

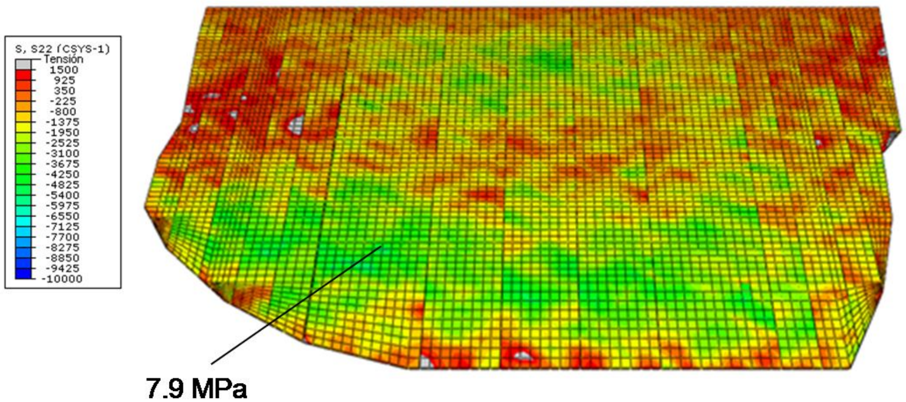


E_{3A} = 210 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa

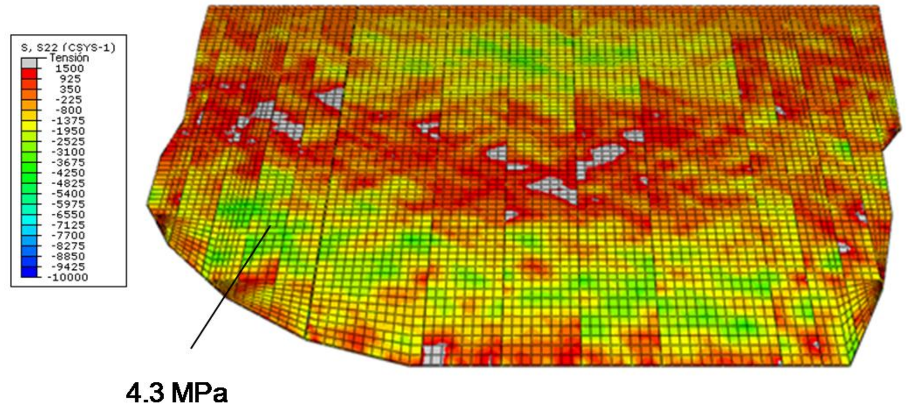


MAXIMUM STRESS ALONG THE CONCRETE FACE (DOWNWARD DIRECTION)

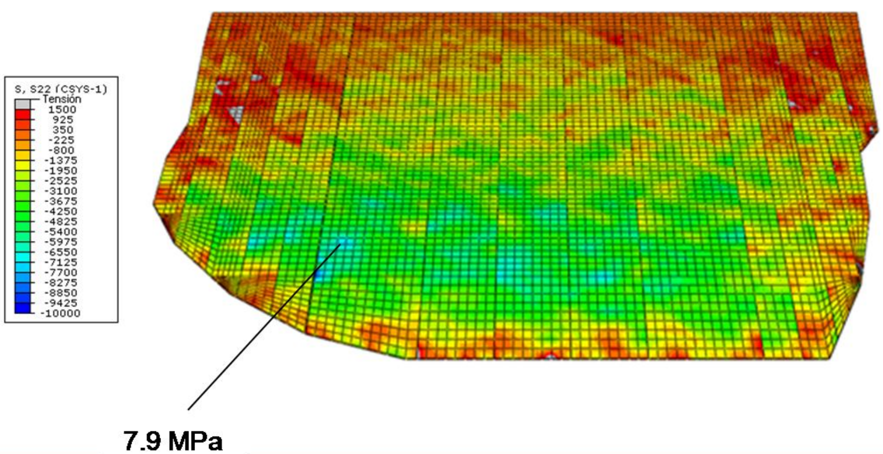
E_{3A} = 130 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



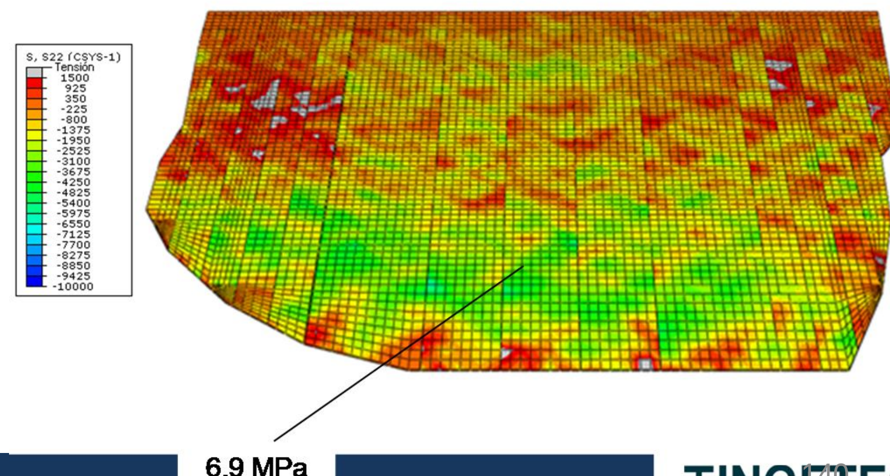
E_{3A} = 210 MPa E_{3B} = 50 MPa E_{3C} = 60 MPa



E_{3A} = 130 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa



E_{3A} = 210 MPa E_{3B} = 100 MPa E_{3C} = 60 MPa



SOGAMOSO DAM

SOGAMOSO DAM

SOGAMOSO DAM

SOGAMOSO DAM

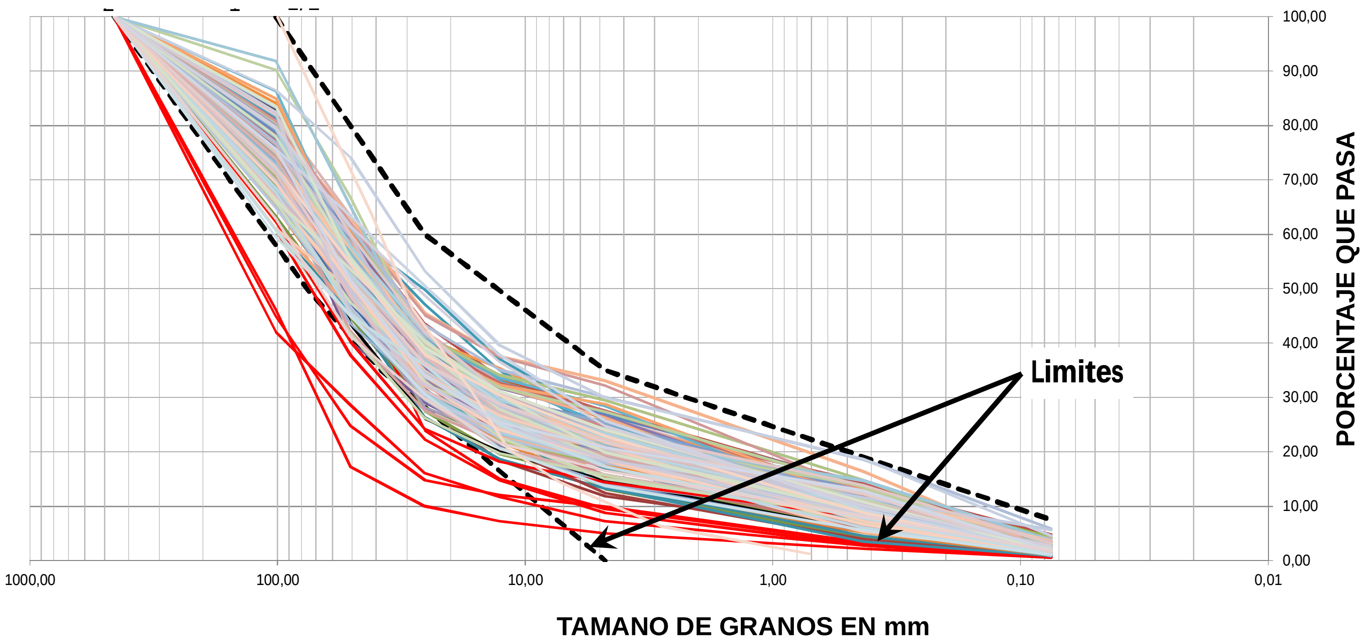
RIGHT PLINTH EXCAVATIONS

INGETEC
INGENIEROS CONSULTORES

CONCRETE

Final pouring – Section 16

ZONE 3 A GRADATION



Cu = 108

K = 9 x 10^{-04} m/s

modulus of compressibility BY IN SITU LOAD PLATE TESTS

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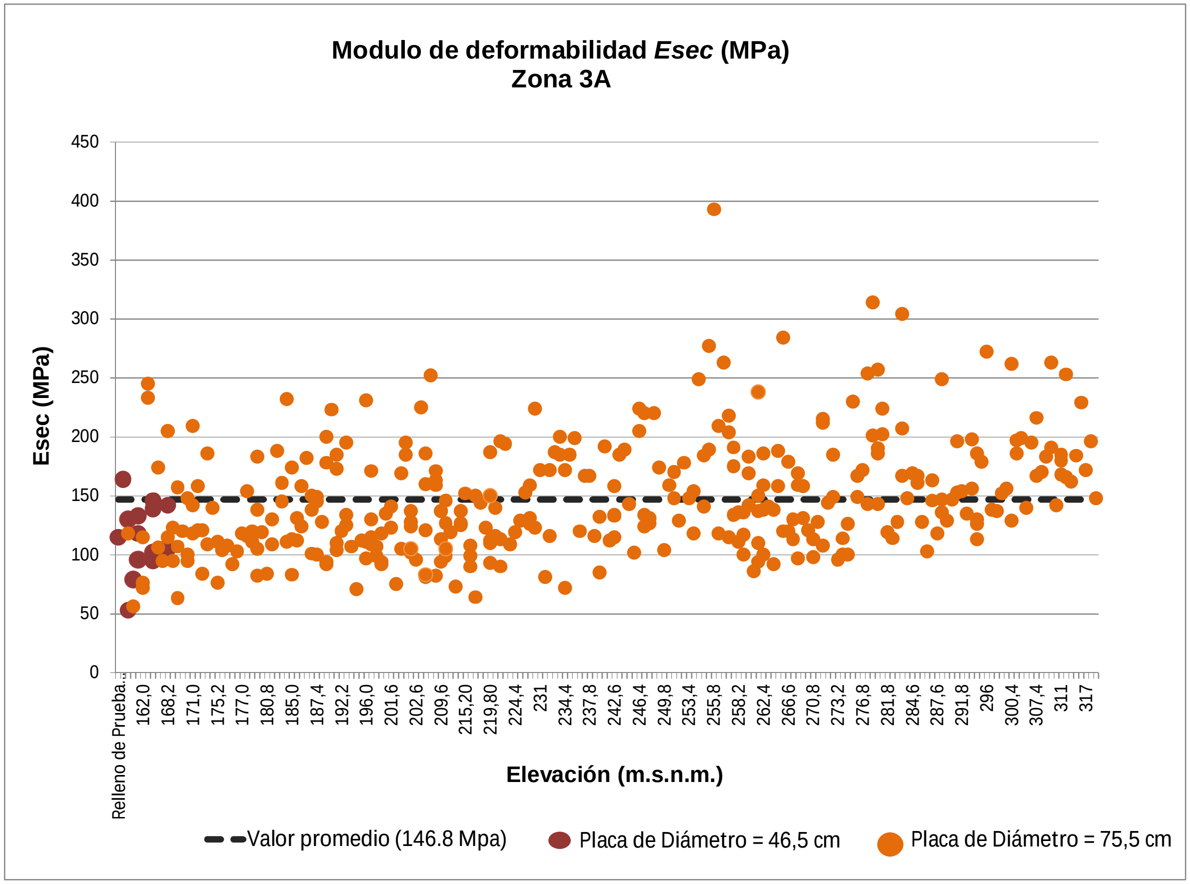
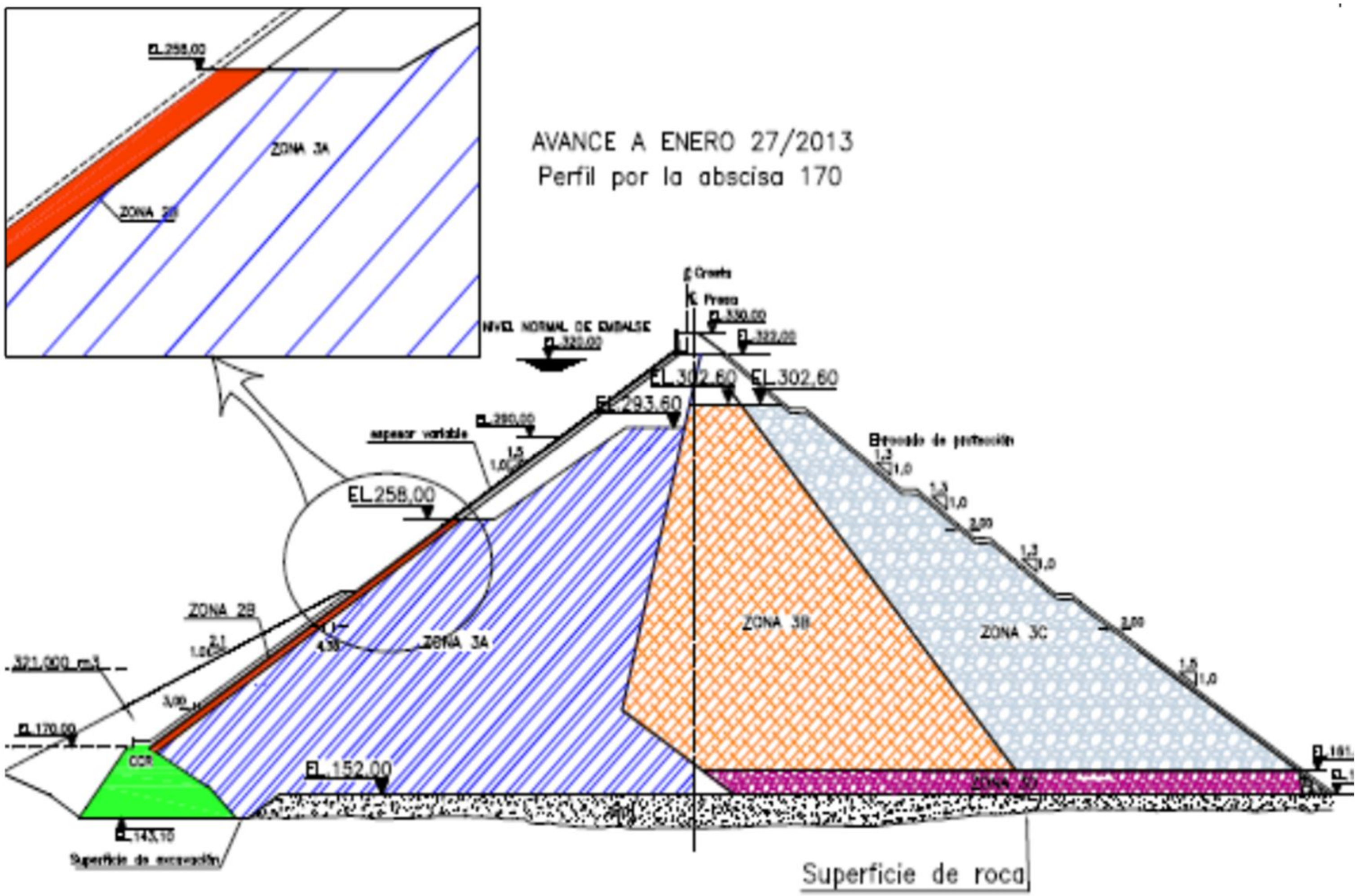
ZONE 3 A (Eav = 147 Mpa)




AVANCE A ENERO 27/2013
Perfil por la abscisa 170
EL.258,00
ZONA 3A
ZONA 2B
NIVEL NORMAL DE EMBALSE
EL.302,60
EL.302,60
EL.293,60
espesor variable
EL.258,00
ZONA 2B
ZONA 3A
ZONA 3B
ZONA 3C
EL.152,00
Superficie de excavación
Superficie de roca

SOGAMOSO DAM

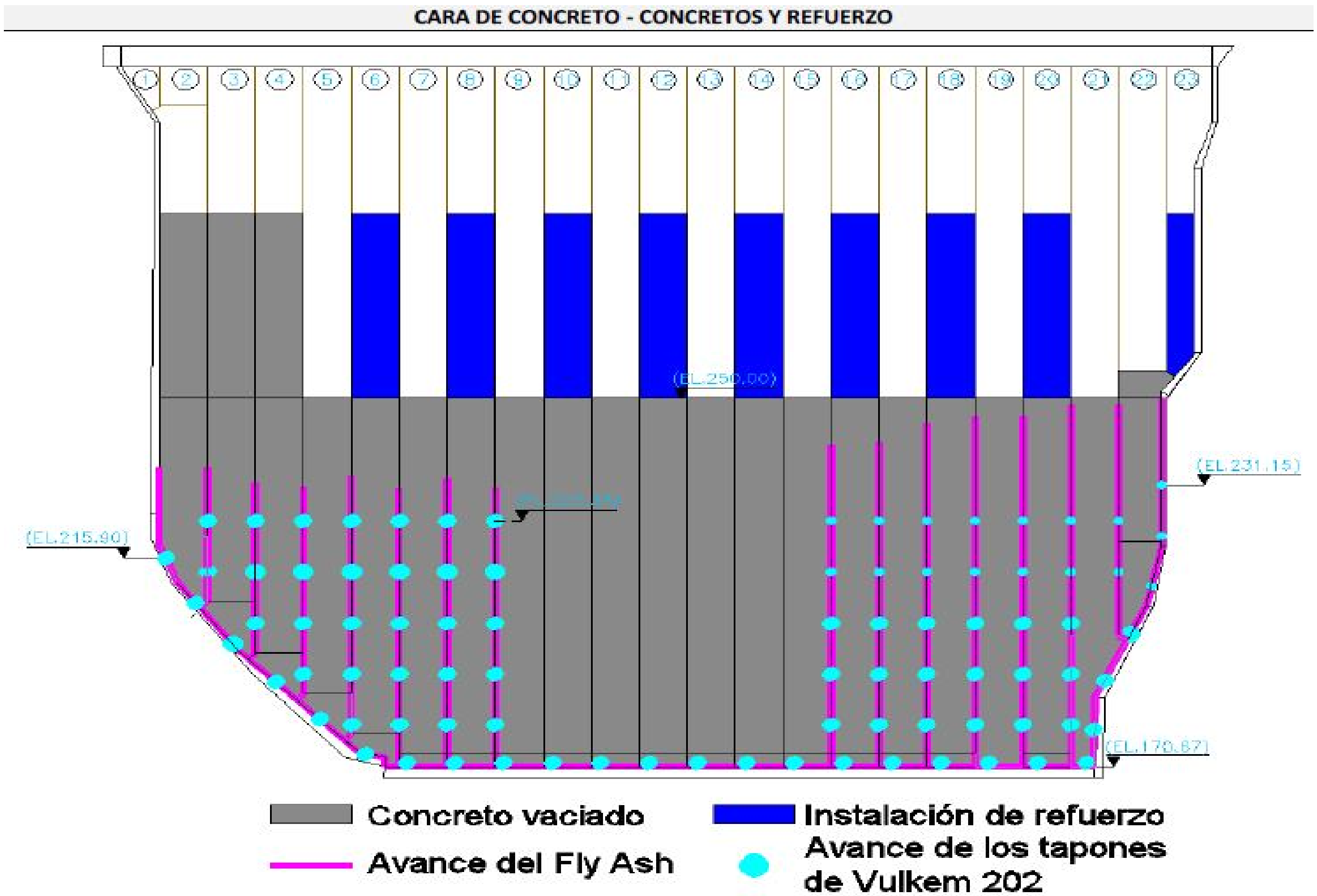
SOGAMOSO DAM

JUNE 2013

SOGAMOSO DAM

Panoramic view of the Dam. September 2013

CONSTRUCTION PROGRESS – CONCRETE FACE



AERIAL VIEW OF THE DAM

AERIAL VIEW OF THE DAM

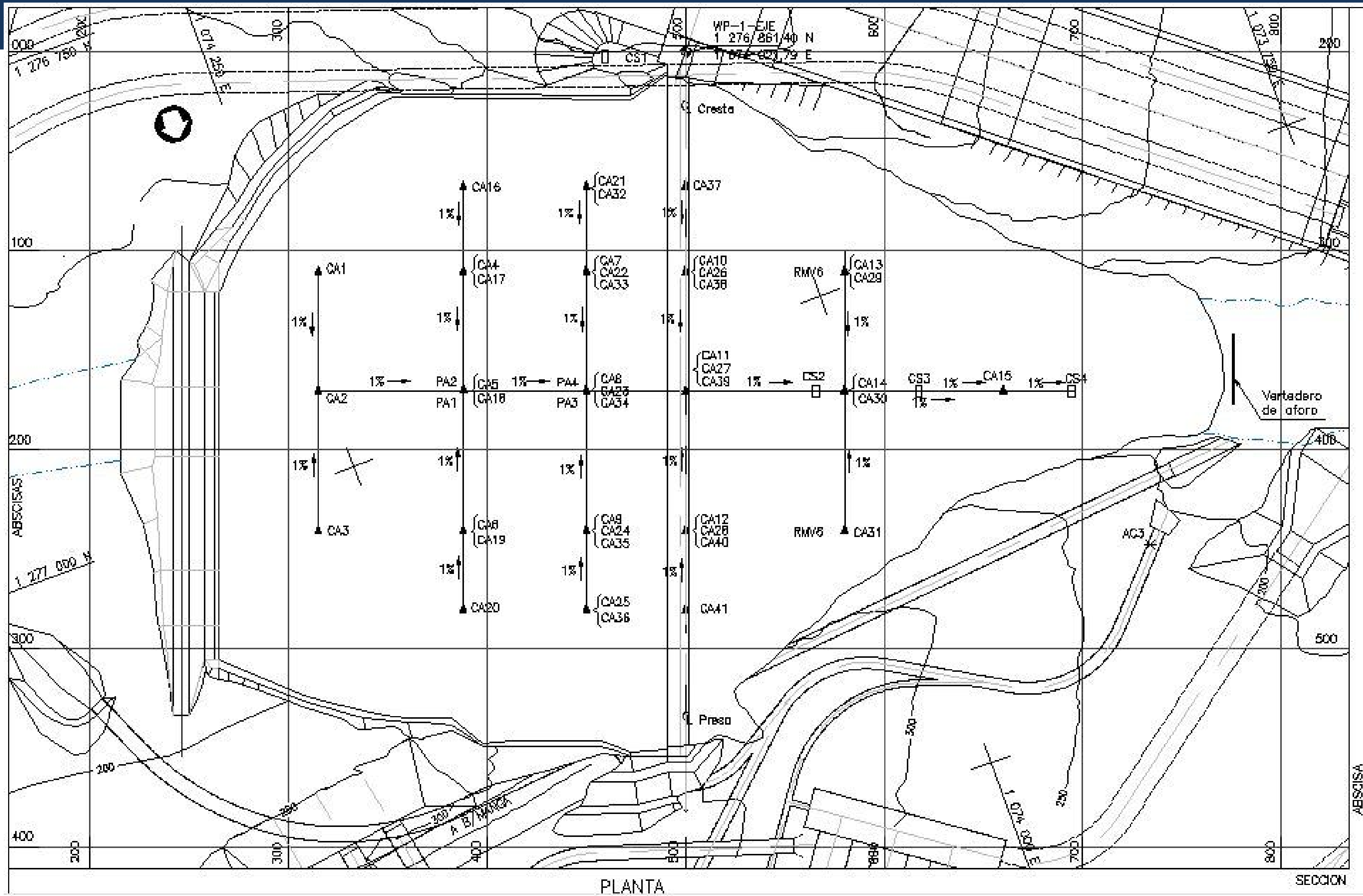
AERIAL VIEW OF THE DAM

AERIAL VIEW OF THE DAM

SETTLEMENTS

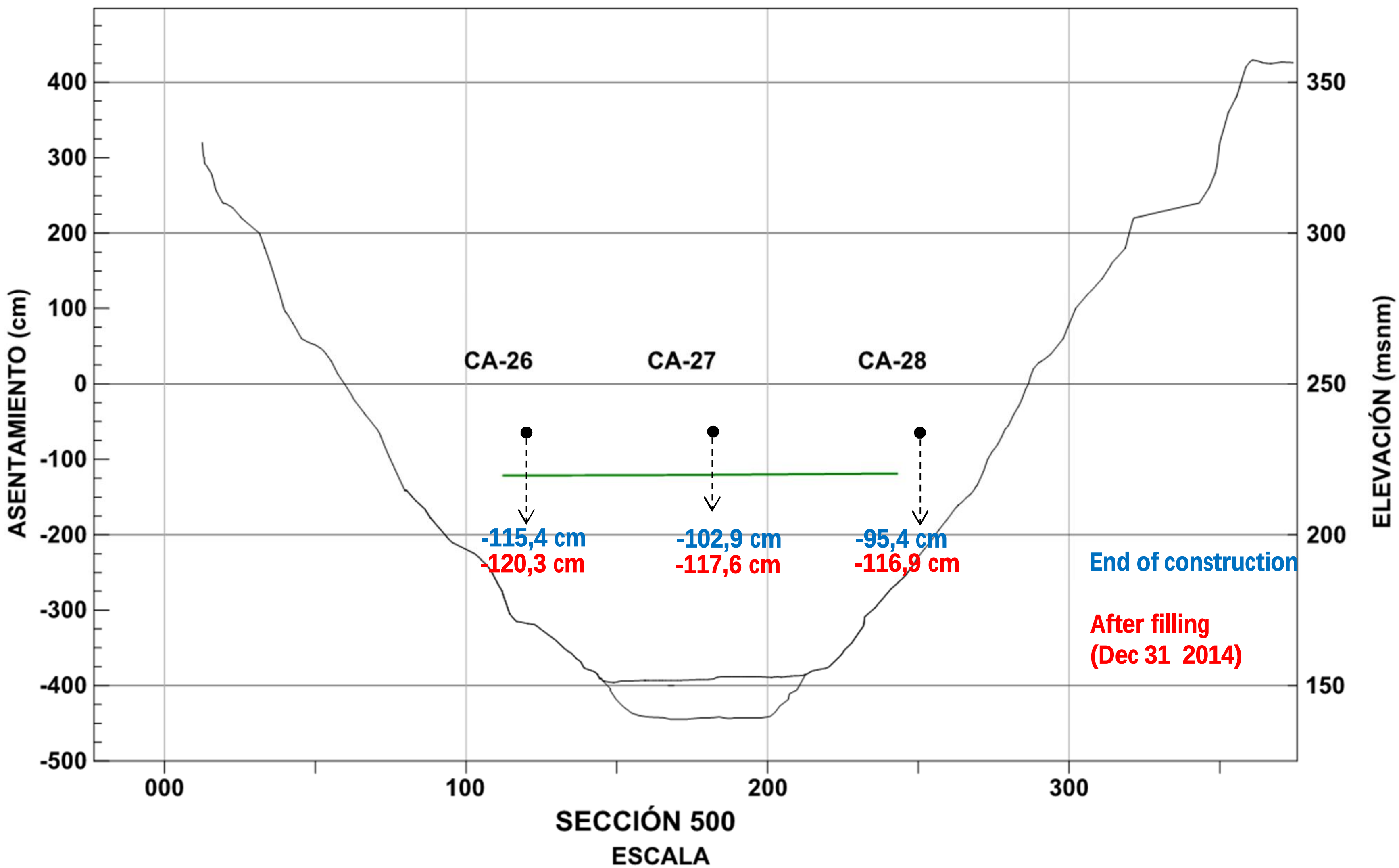
Fill settlements were measured by hydraulic cells and magnetic ring settlement gauges

DAM PLAN VIEW – SETTLEMENT CELLS



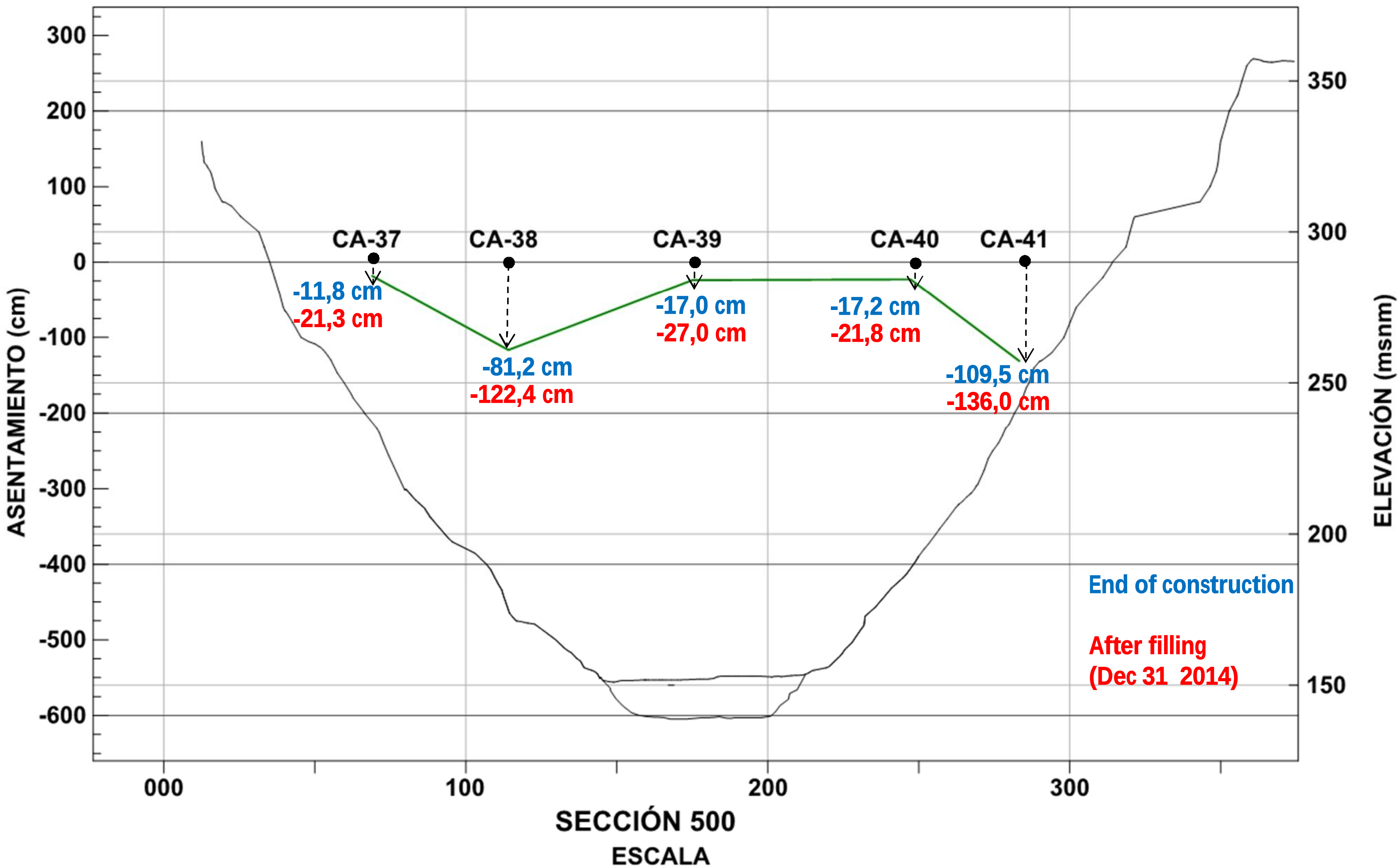
INGETEC
INGENIEROS CONSULTORES

CROSS SECTION - SETTLEMENT AT EL. 250,00 m.a.s.l



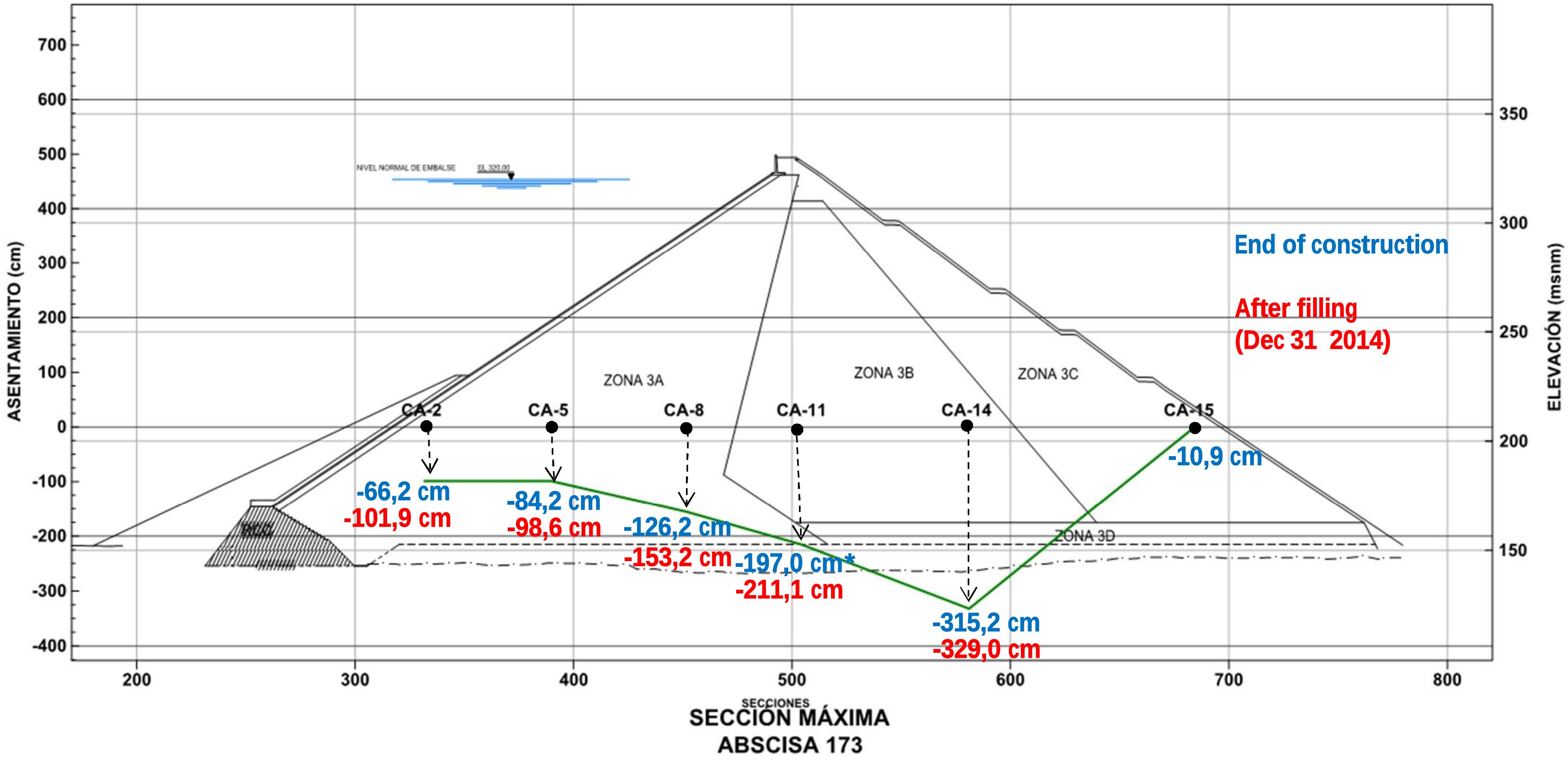
INGETEC INGENIEROS CONSULTORES

CROSS SECTION - SETTLEMENT AT EL. 290,00



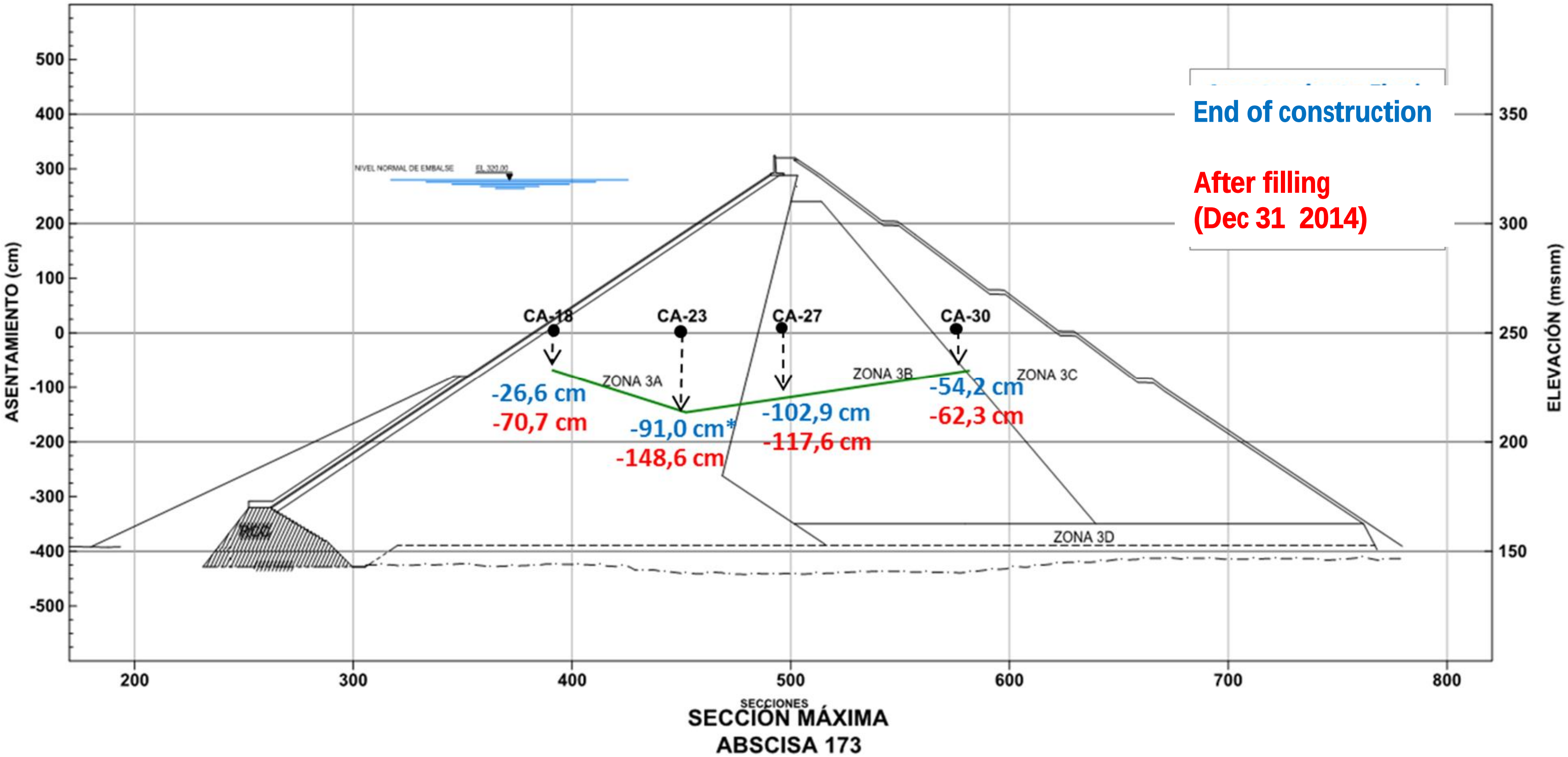
INGETEC INGENIEROS CONSULTORES

MAXIMUM SECTION - SETTLEMENT AT EL. 206,50 m.a.s.l



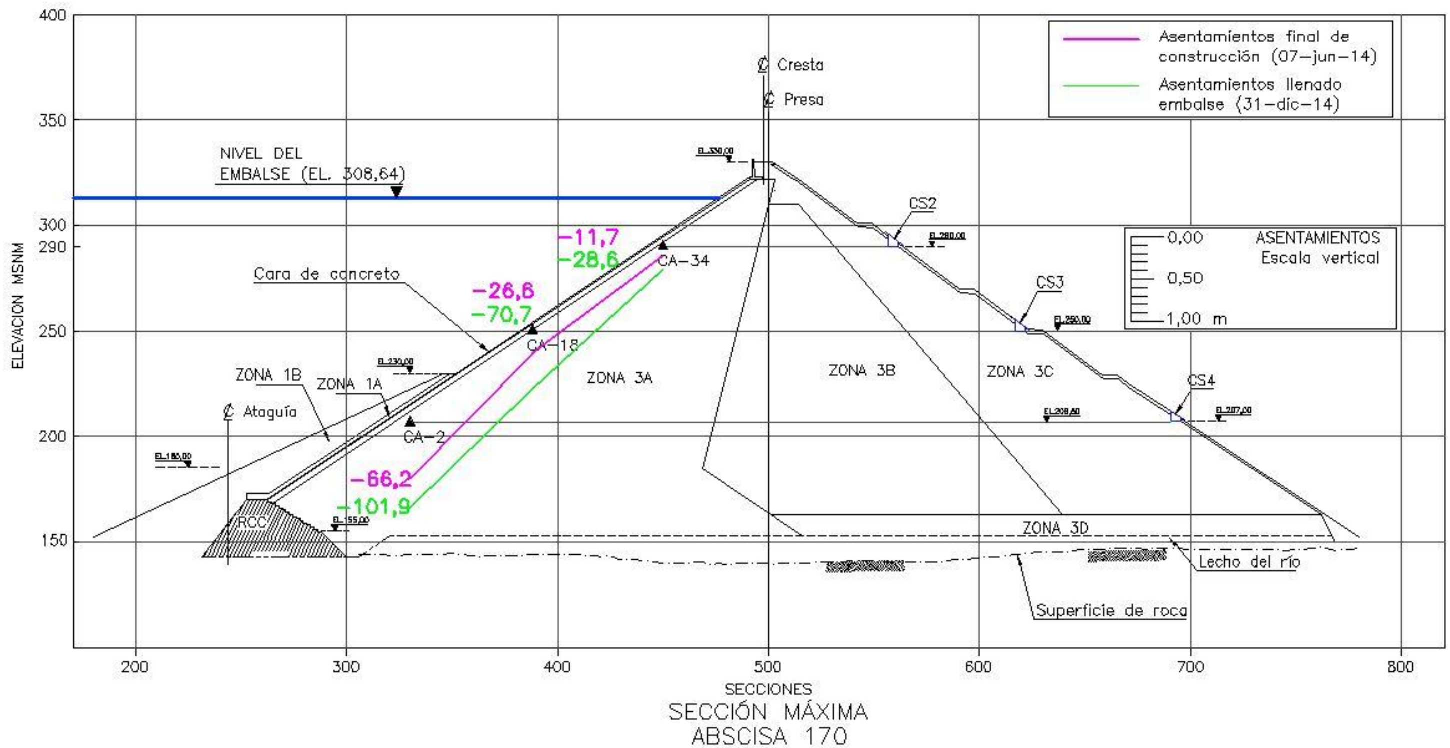
INGETEC
INGENIEROS CONSULTORES

MAXIMUM SECTION - SETTLEMENT AT EL. 250



INGETEC INGENIEROS CONSULTORES

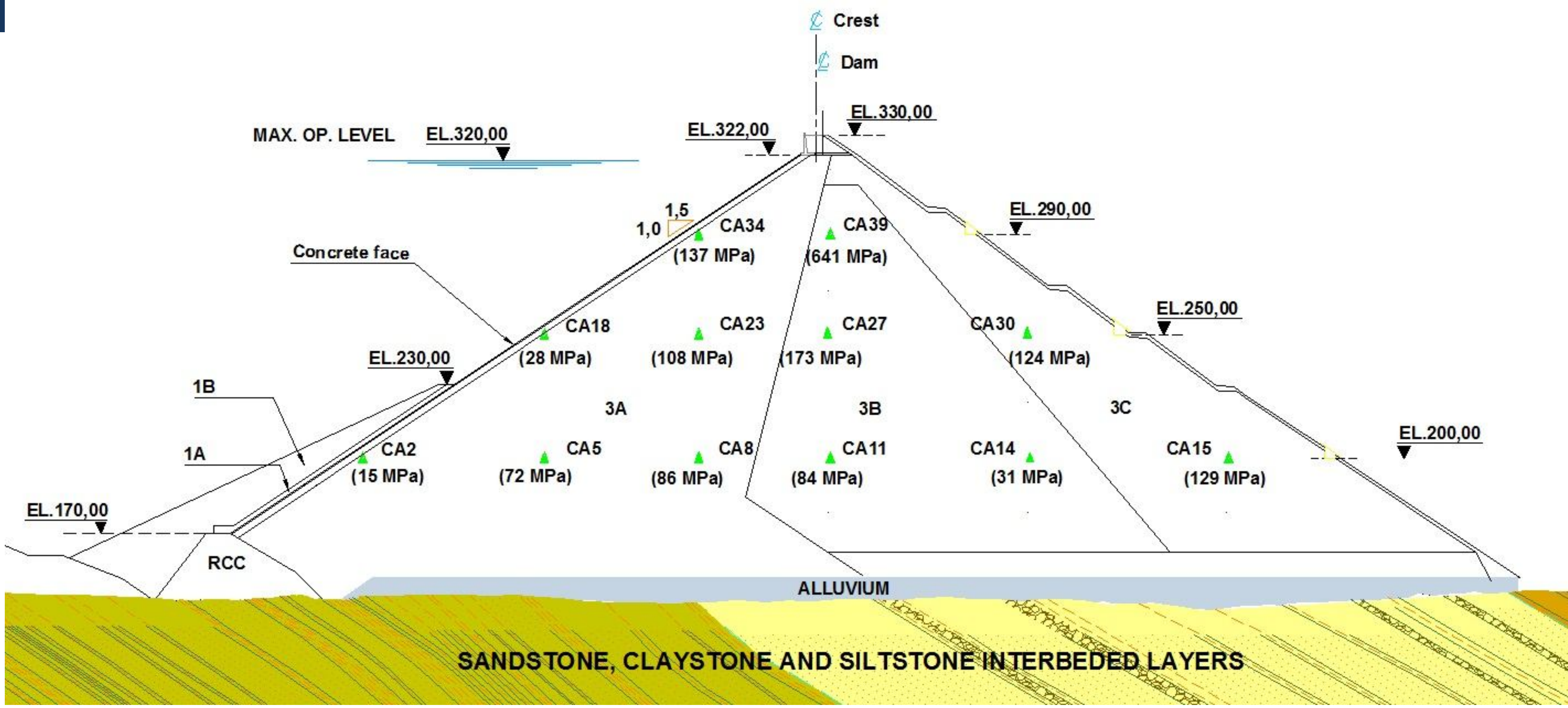
SETTLEMENT OF NEAREST CELLS TO THE CONCRETE FACE AT END OF CONSTRUCTION AND AFTER FILLING

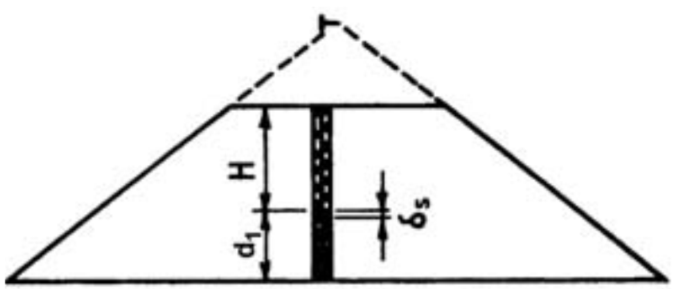


End of construction

After filling

MODULUS DURING CONSTRUTION



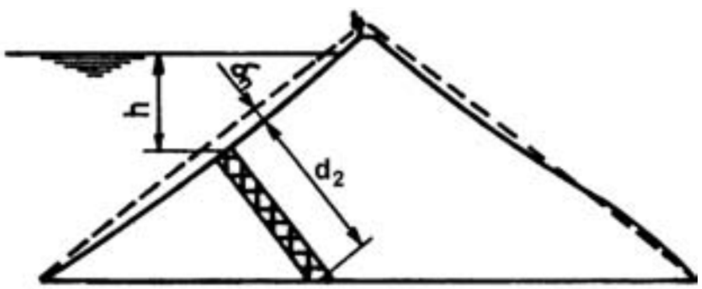


$$E_{rC} = \gamma \, H \, d_1/\delta_S$$

INGETEC INGENIEROS CONSULTORES

MODULUS DURING RESERVOIR FILLING E_{rf} (Fitzpatrick et al, 1985)

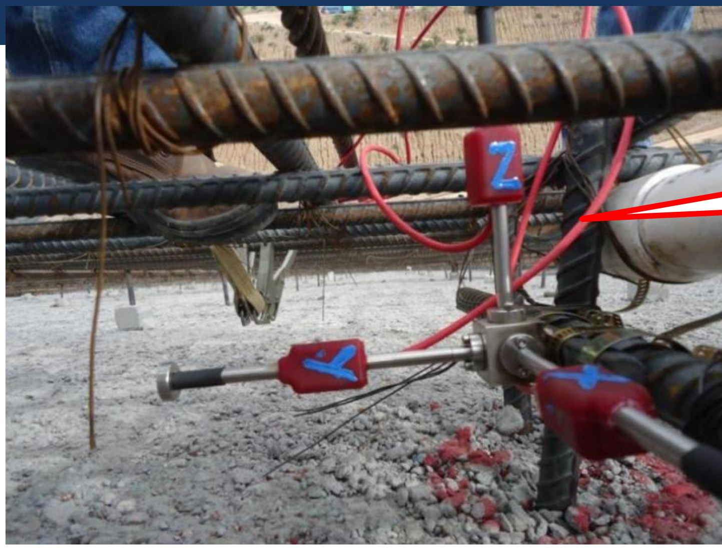
CELL	Elevation (m.a.s.l)	Normal displacement δ_n (cm)	Modulus E_{rf} MPa
CA - 02	206,5	35,7	228
CA - 18	250	44,1	184
CA - 34	290	16,9	180



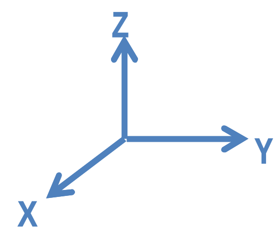
MODULUS DURING RESERVOIR FILLING

$$E_{rf} = \gamma_w \, h \, d_2/\delta_n$$

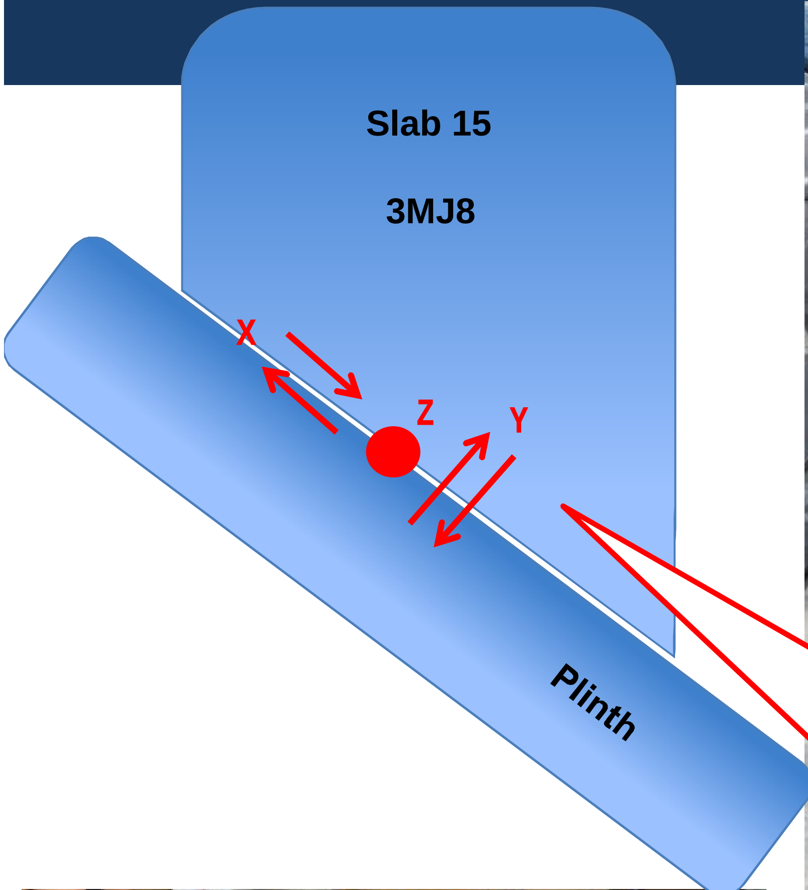
CONCRETE FACE
STRAIN GAUGE



Strain Gauge
Eje **X**: Along the concrete face
Eje **Y**: Horizontal to the concrete face,
Eje **Z**: Perpenticular to the concrete face



INGETEC
INGENIEROS CONSULTORES

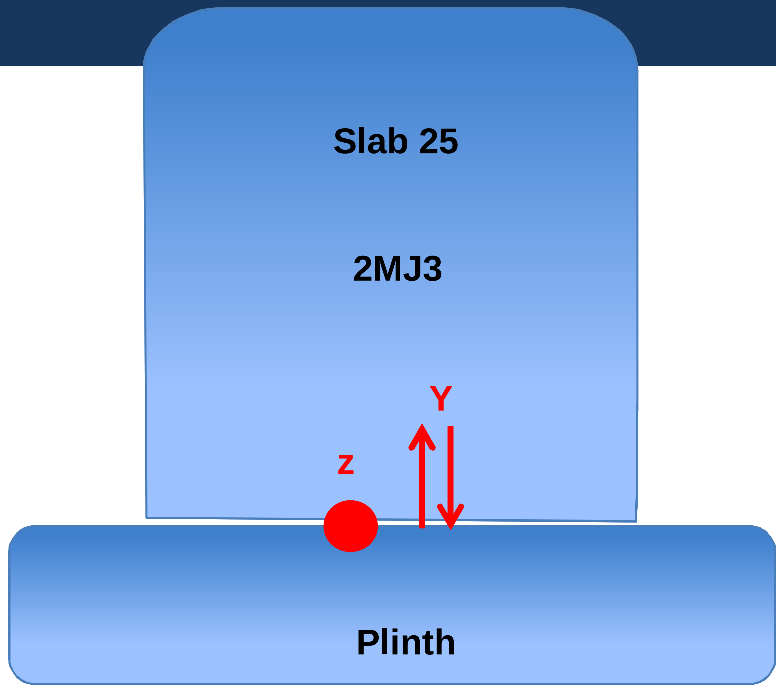


Jointmeter 3MJ – in perimeter joint

Sensor in **X axis** : Measures the displacement of the joint between the slab and the plinth (shear)

Sensor in **Y axis** : Measures the opening or closing of the joint with the plinth

Sensor in **Z axis** : Measures if the slabs goes up or down with respect to the plinth

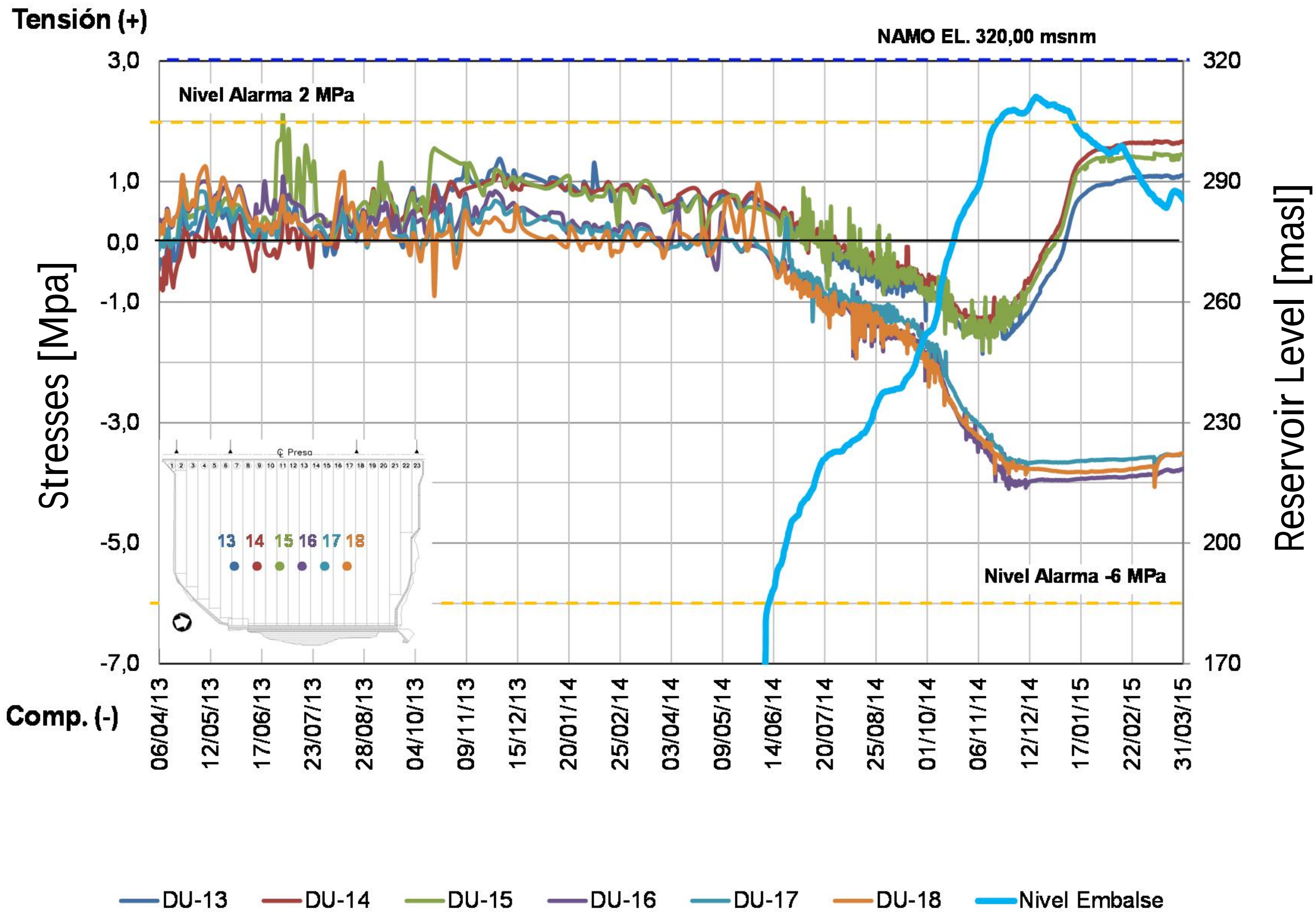


Medidor de junta 2MJ – en talón de la junta perimetral

Sensor in **Y axis** : measures the opening or closing of the slab with respect to the plinth

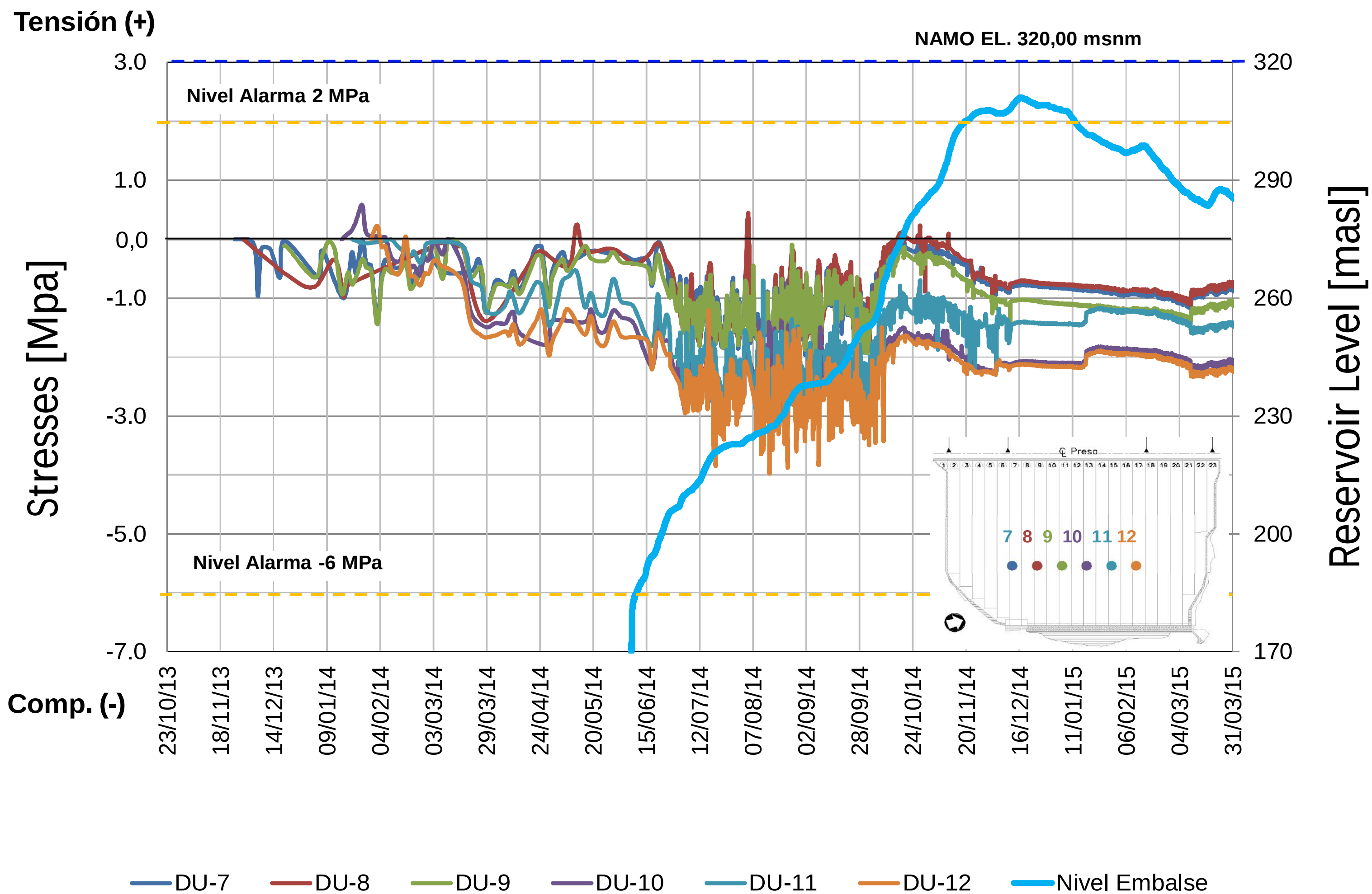
Sensor in **Z axis** : Measures if the slabs goes up or down with respect to the plinth

Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at EL. 226,04 masl



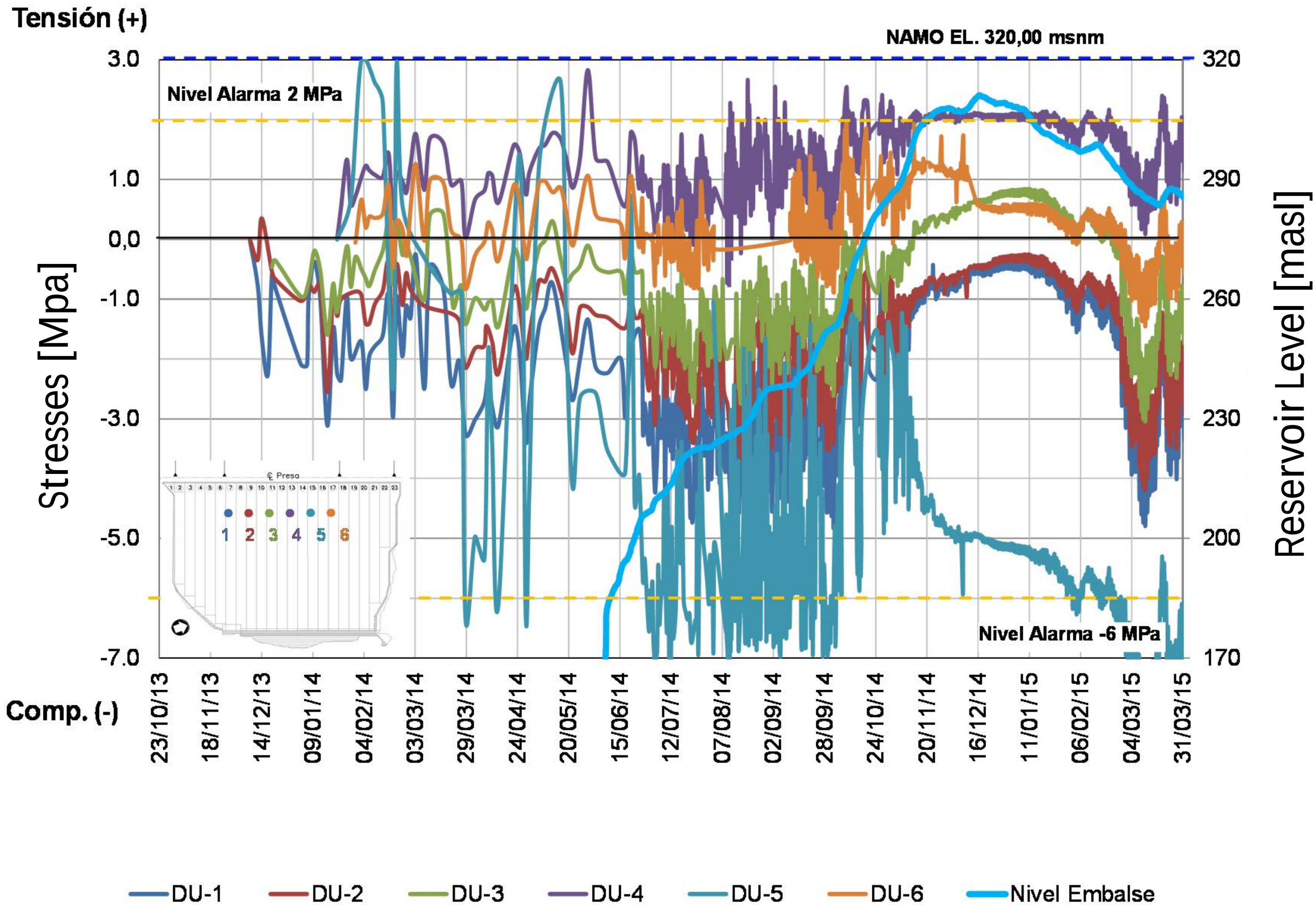
INGETEC INGENIEROS CONSULTORES

Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at EL. 259,25 masl



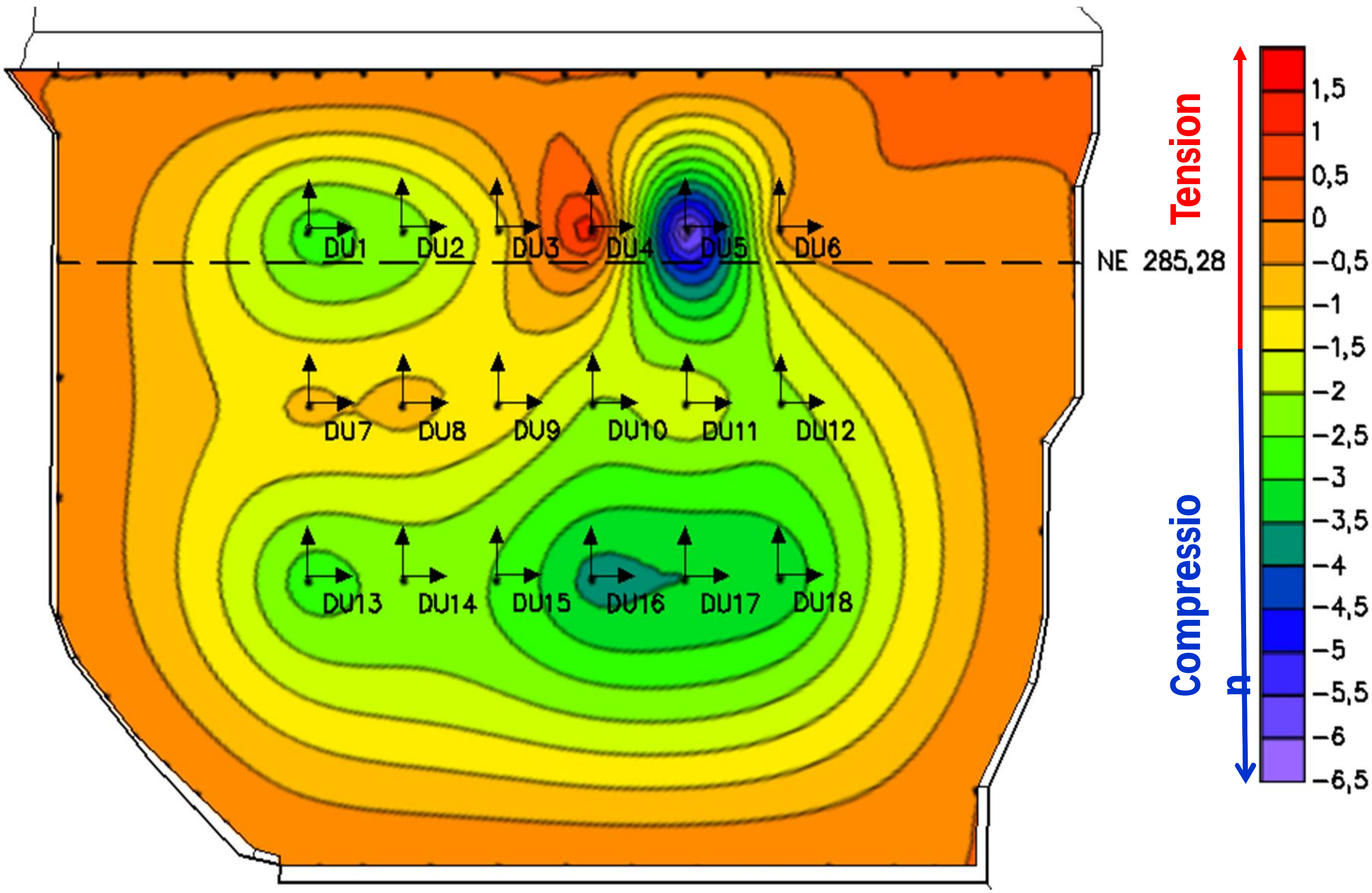
INGETEC INGENIEROS CONSULTORES

Horizontal Stresses (parallel to the dam axis) obtained from strain gauges placed at a EL. 292,47 masl



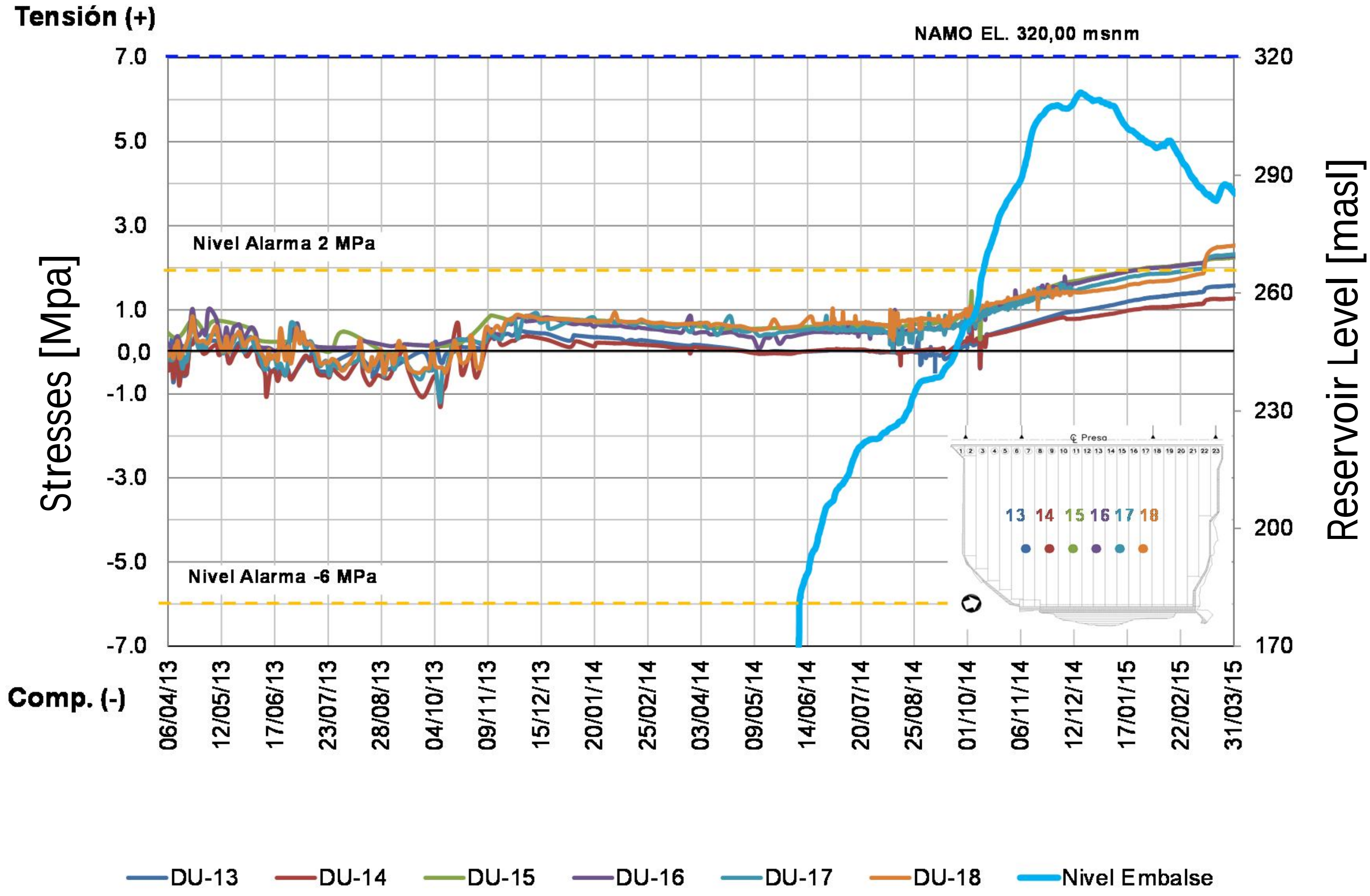
INGETEC INGENIEROS CONSULTORES

Horizontal stress contours 31 de marzo de 2015 [MPa].

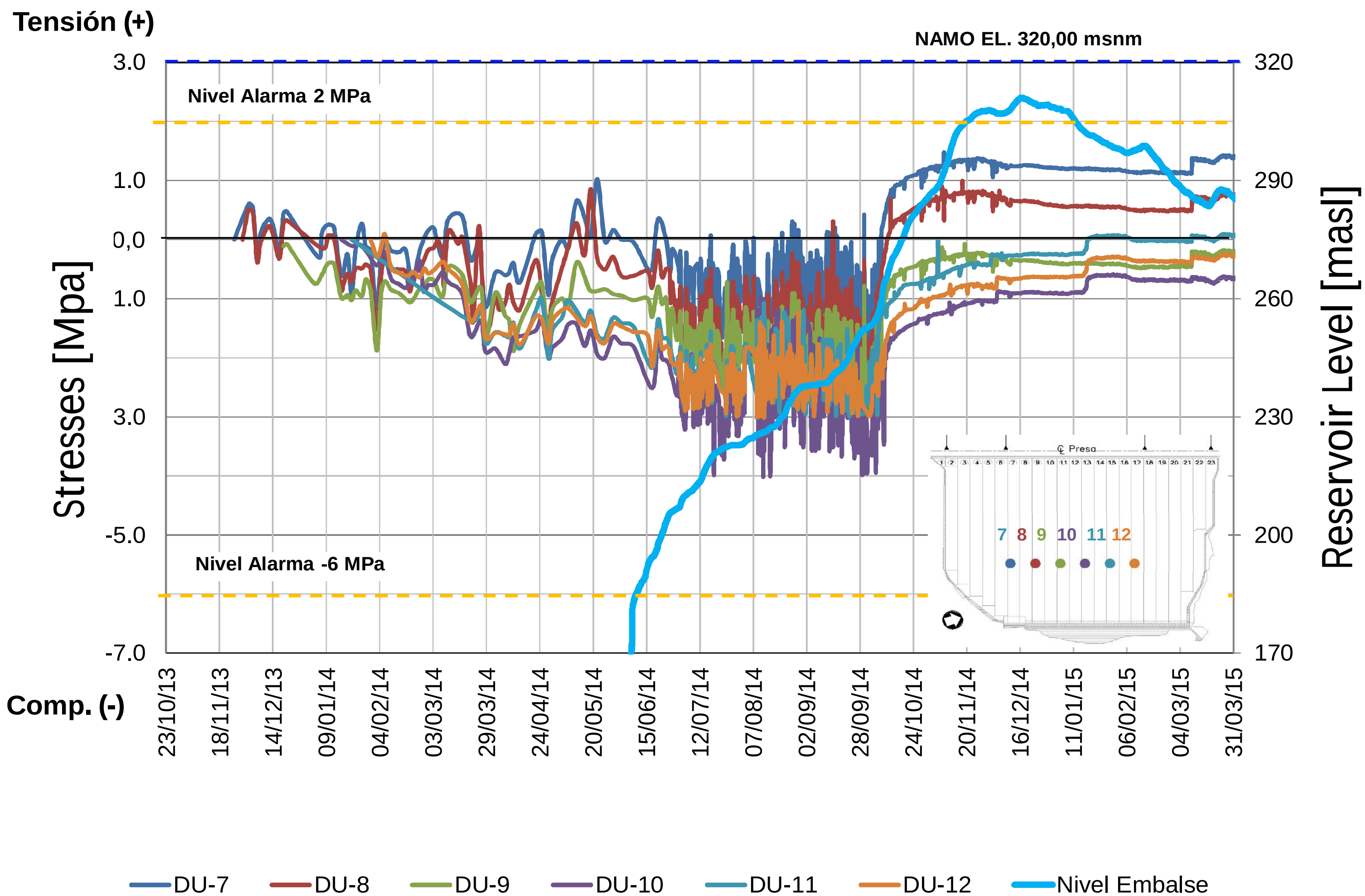


INGETEC INGENIEROS CONSULTORES

Stresses along the concrete face plane at EL. 226,04 msnm

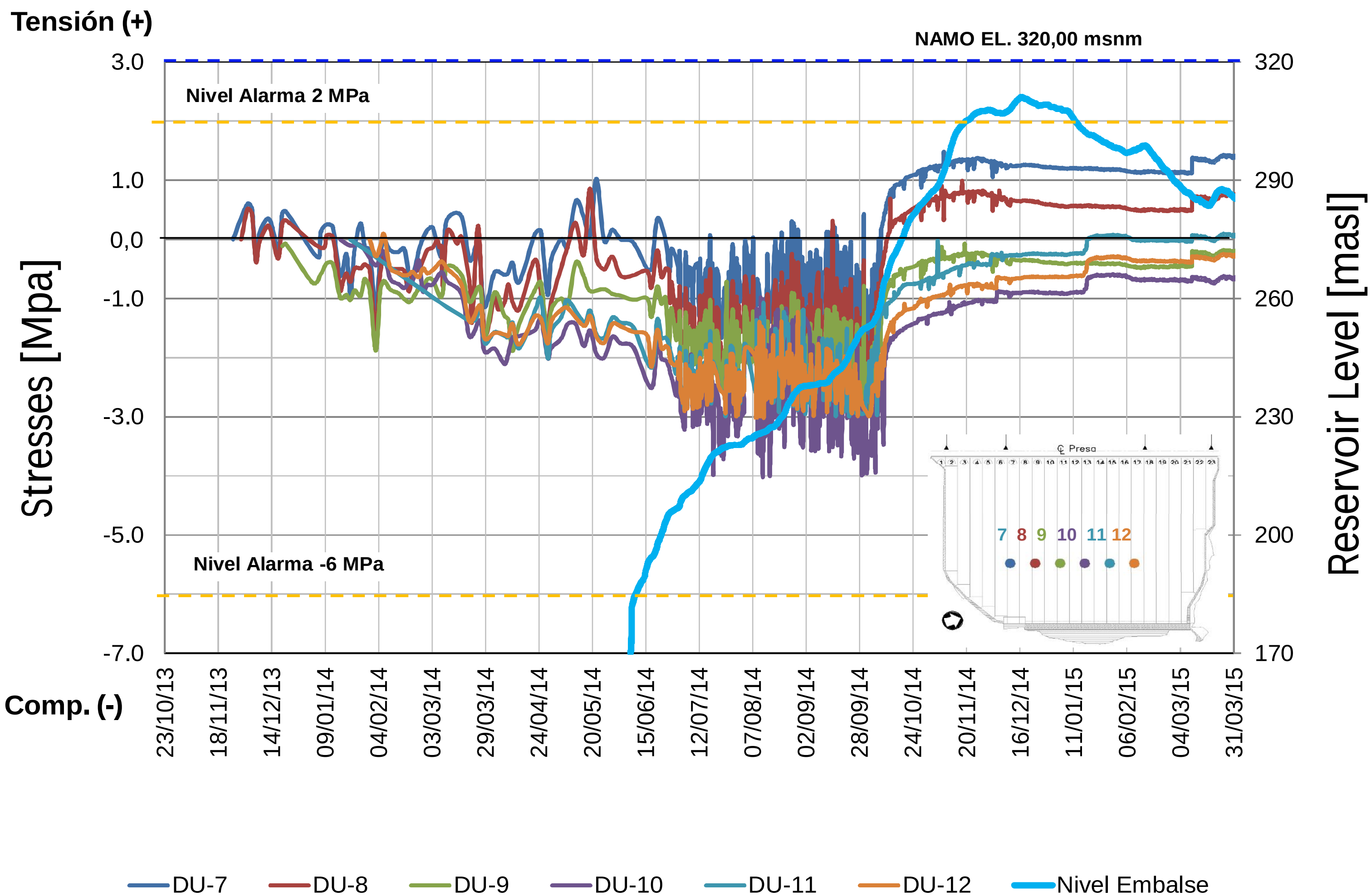


Stresses along the concrete face plane at 259,25 msnm



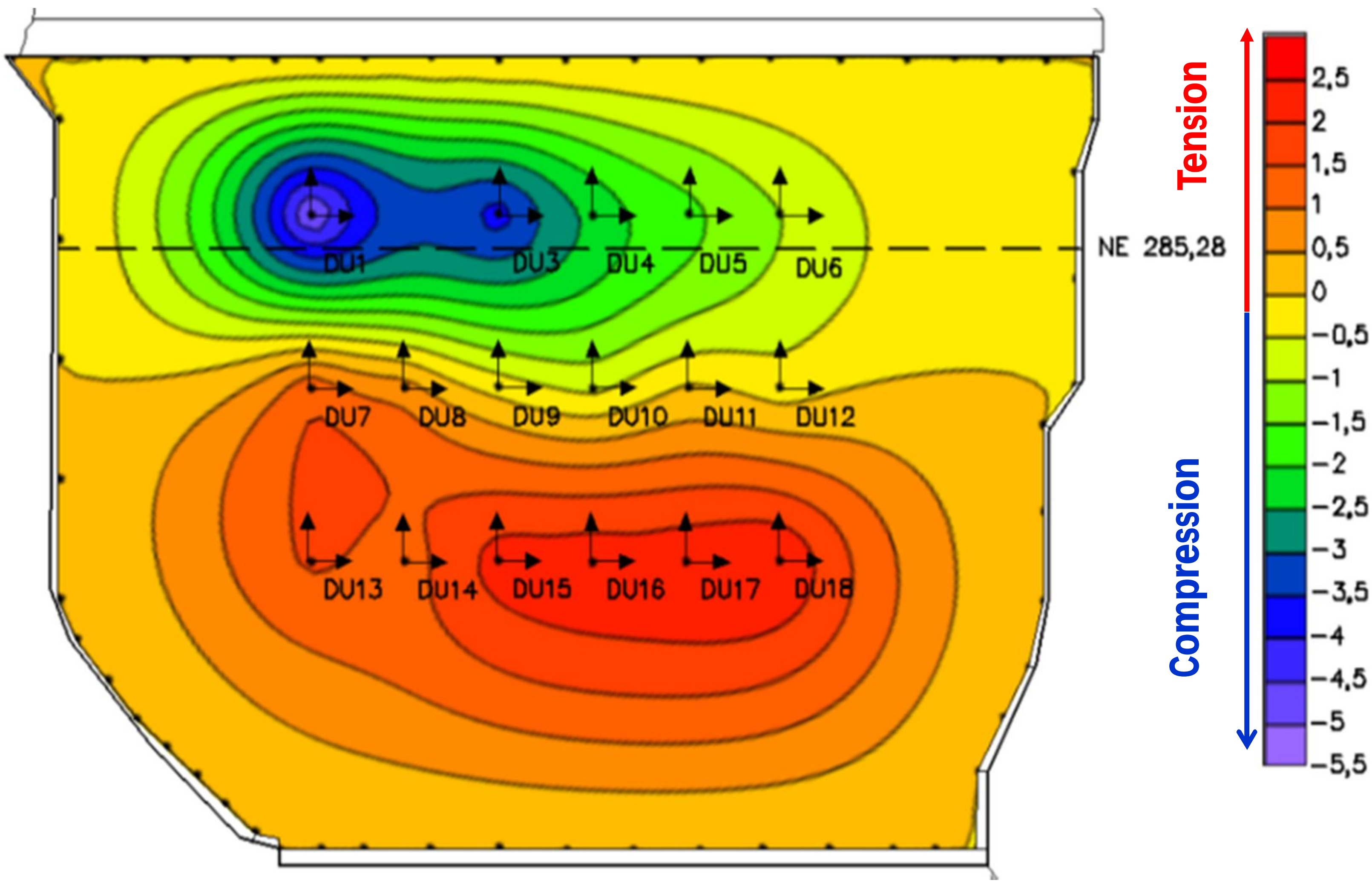
INGETEC INGENIEROS CONSULTORES

Stresses along the concrete face plane at EL. 292,47 msnm



INGETEC INGENIEROS CONSULTORES

Perpendicular stress contours 31 de marzo de 2015 [MPa].



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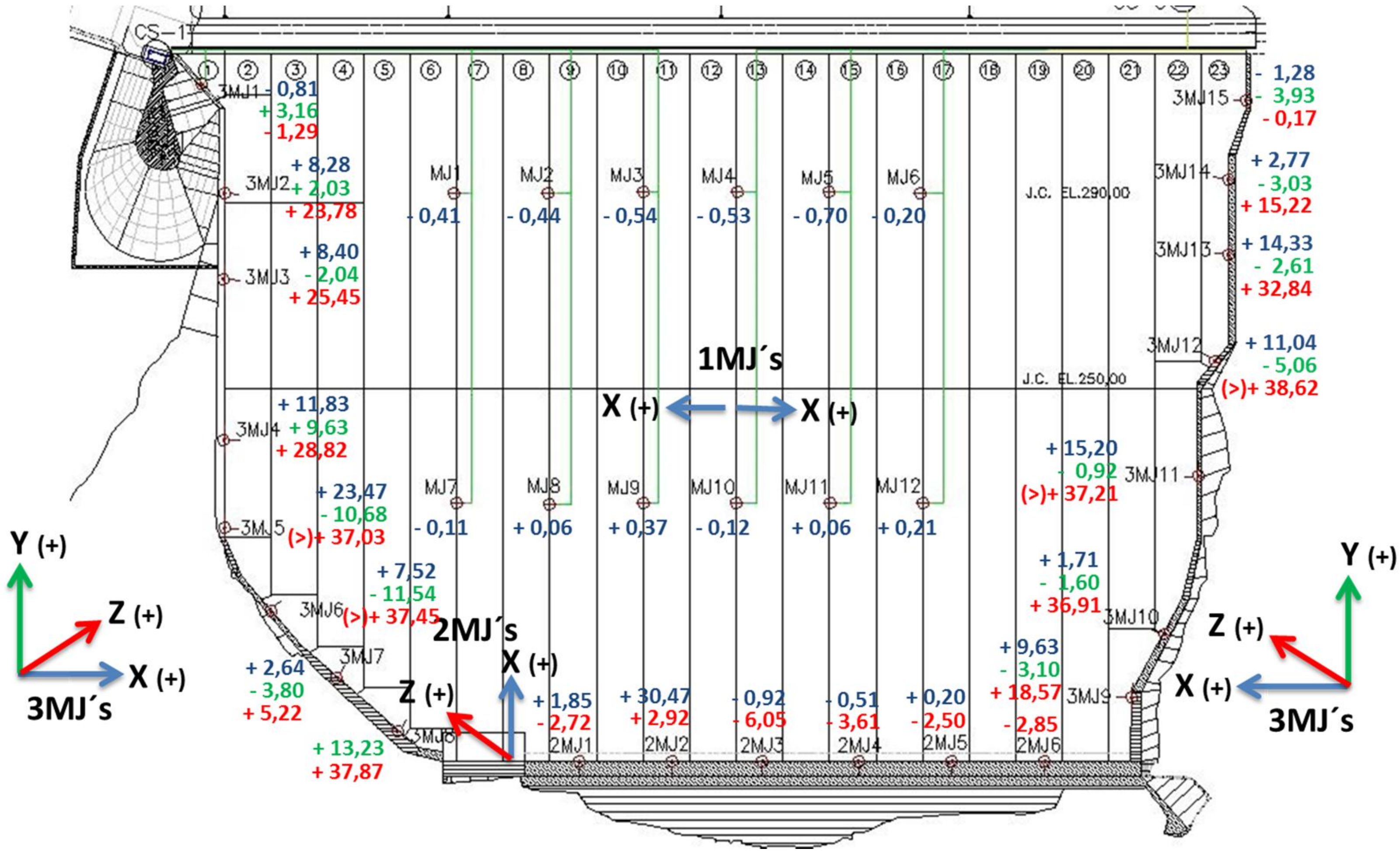
Joint total displacements at max. water elevation level 311 masl



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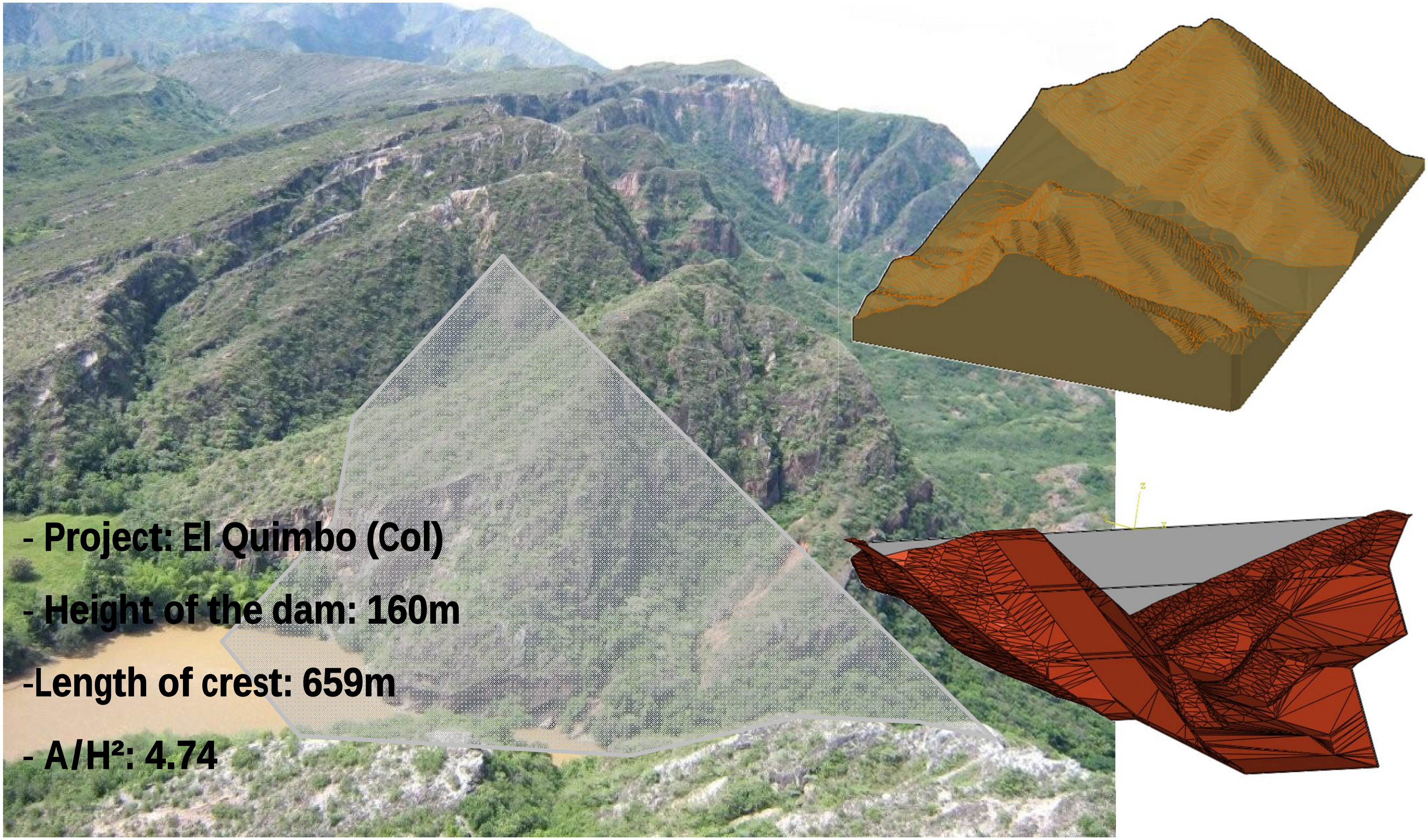
Main findings are:

- Perimeter joints movements perpendicular to the face were larger than the openings. This confirms observed behavior in another dam with almost vertical abutments.[14]
- Maximum movement perpendicular to the face close to the center of the canyon was about 30 cm.
- Openings and closing along the same joint varied with elevation.

EL QUIMBO PROJECT - COLOMBIA



- Project: El Quimbo (Col)
- Height of the dam: 160m
- Length of crest: 659m
- A/H^2: 4.74

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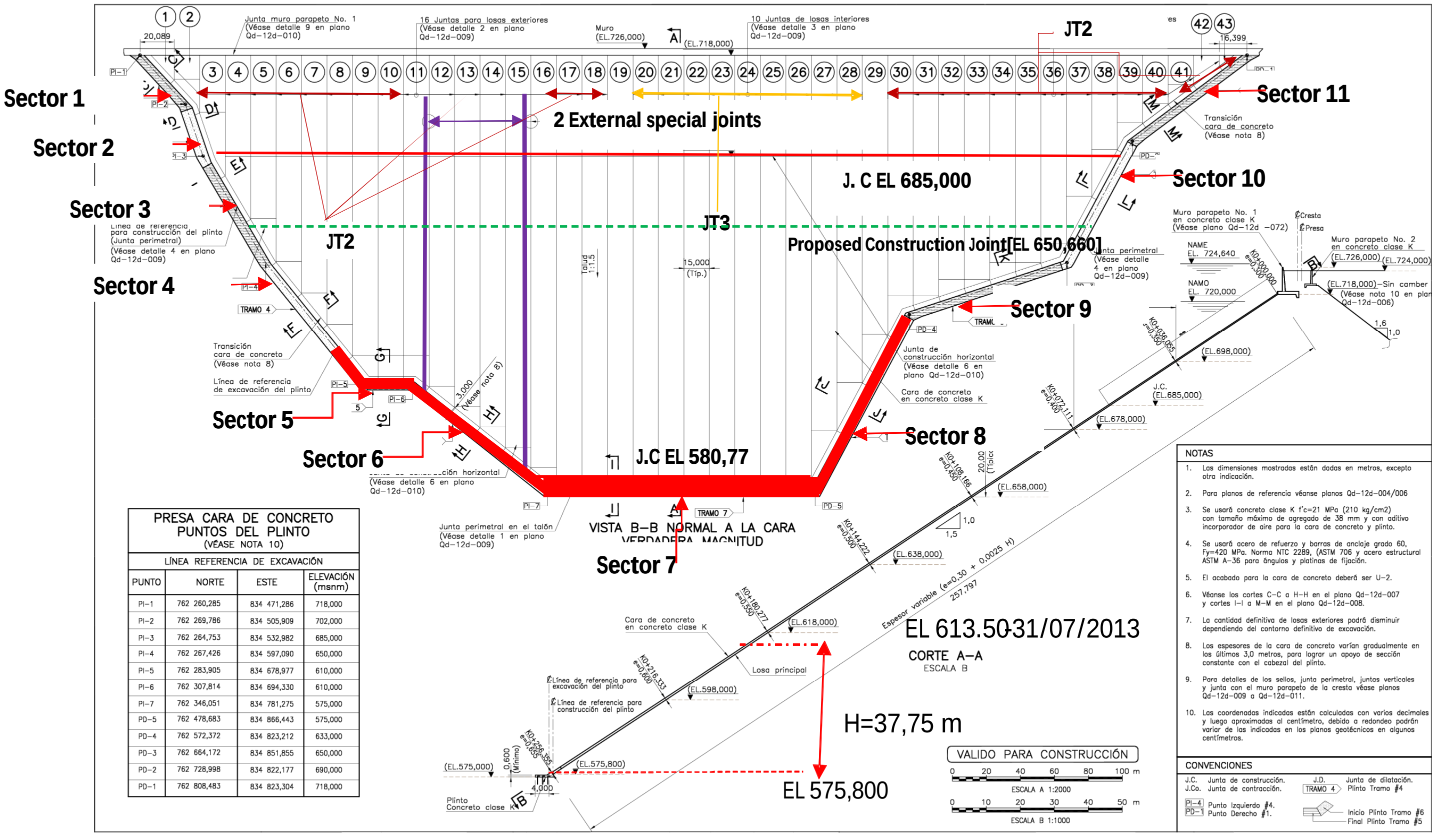
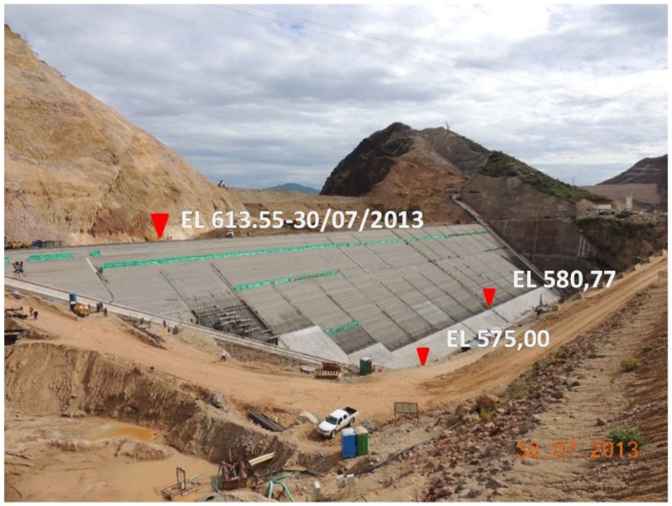
HYDROELECTRIC PROJECT EL QUIMBO

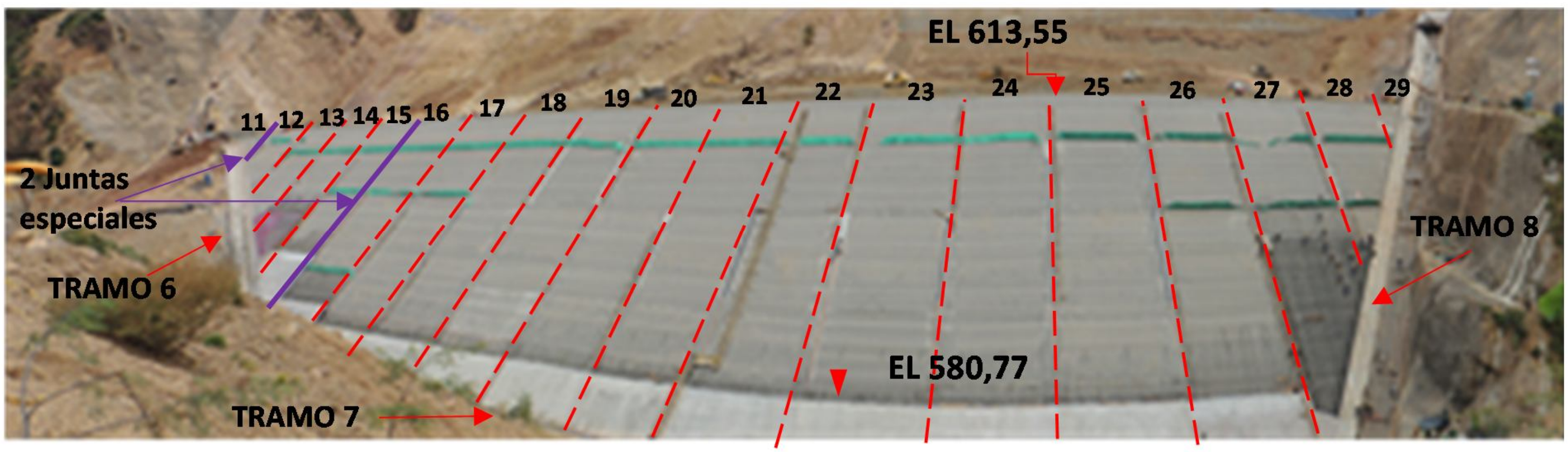


PRESA CARA DE CONCRETO PUNTOS DEL PLINTO (VÉASE NOTA 10)			
LÍNEA REFERENCIA DE EXCAVACIÓN			
PUNTO	NORTE	ESTE	ELEVACIÓN (msnm)
PI–1	762 260,285	834 471,286	718,000
PI–2	762 269,786	834 505,909	702,000
PI–3	762 264,753	834 532,982	685,000
PI–4	762 267,426	834 597,090	650,000
PI–5	762 283,905	834 678,977	610,000
PI–6	762 307,814	834 694,330	610,000
PI–7	762 346,051	834 781,275	575,000
PD–5	762 478,683	834 866,443	575,000
PD–4	762 572,372	834 823,212	633,000
PD–3	762 664,172	834 851,855	650,000
PD–2	762 728,998	834 822,177	690,000
PD–1	762 808,483	834 823,304	718,000

INGETEC INGENIEROS CONSULTORES

EL QUIMBO HYDROLECTRIC PROJECT





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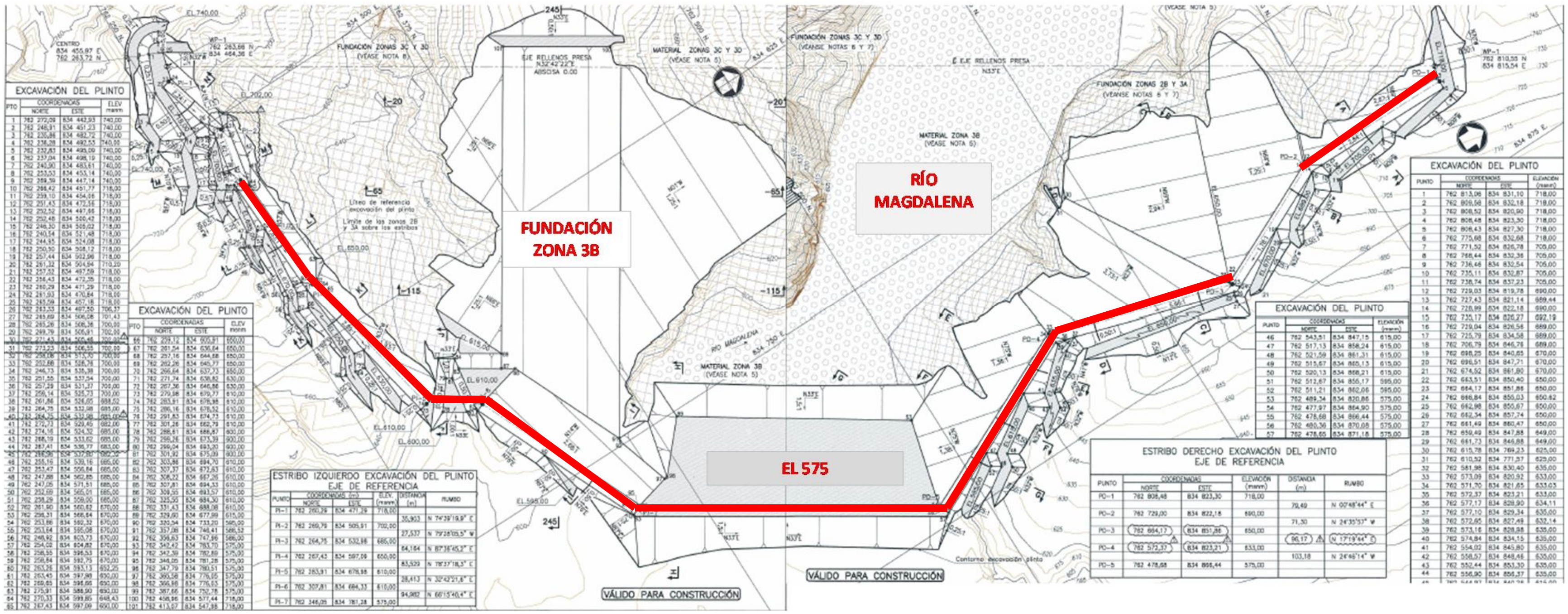
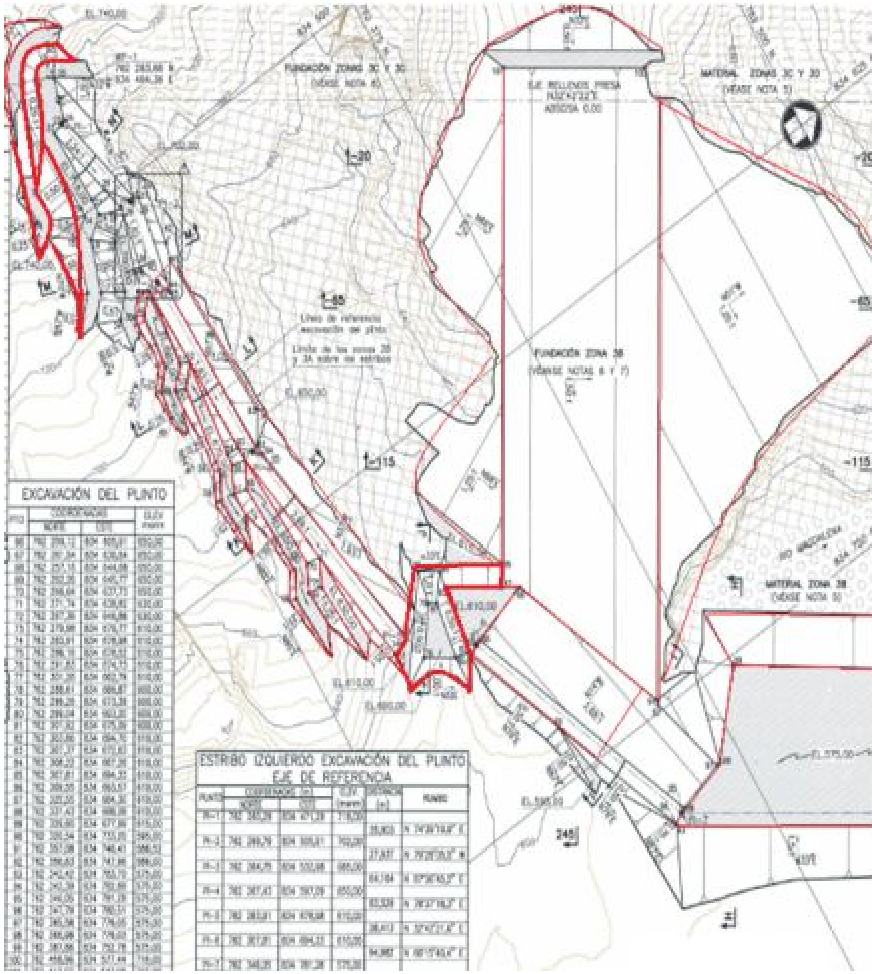
HYDROELECTRIC PROJECT EL QUIMBO

Plinth Excavations

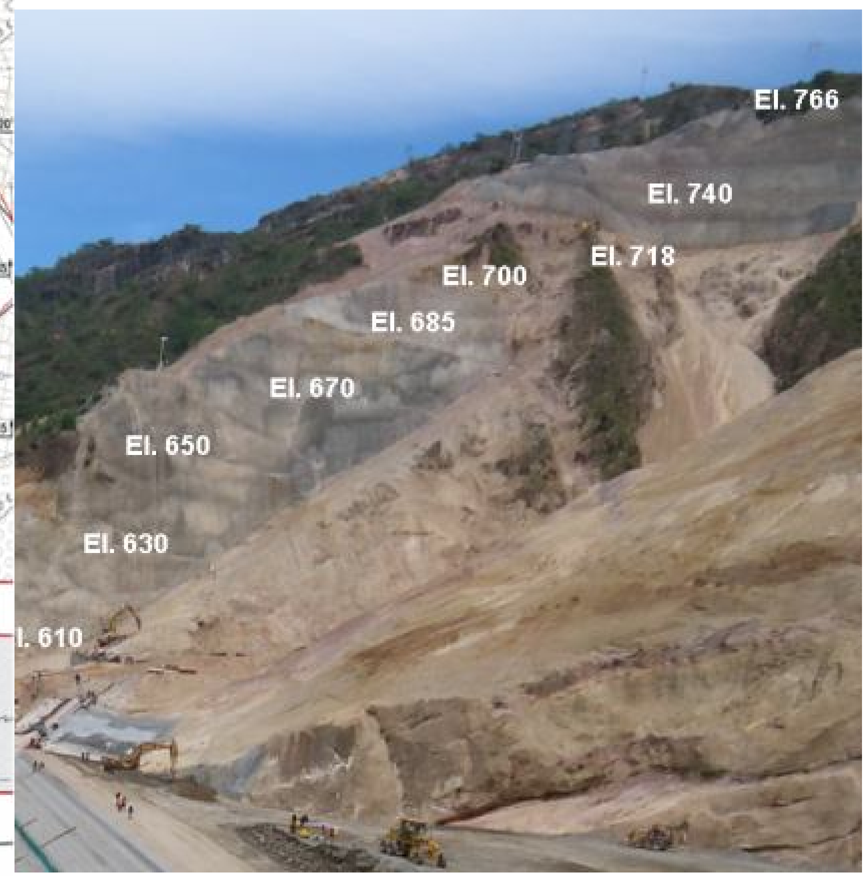


Plan View: Excavations for 11 sectors of the plinth

HYDROELECTRIC PROJECT EL QUIMBO



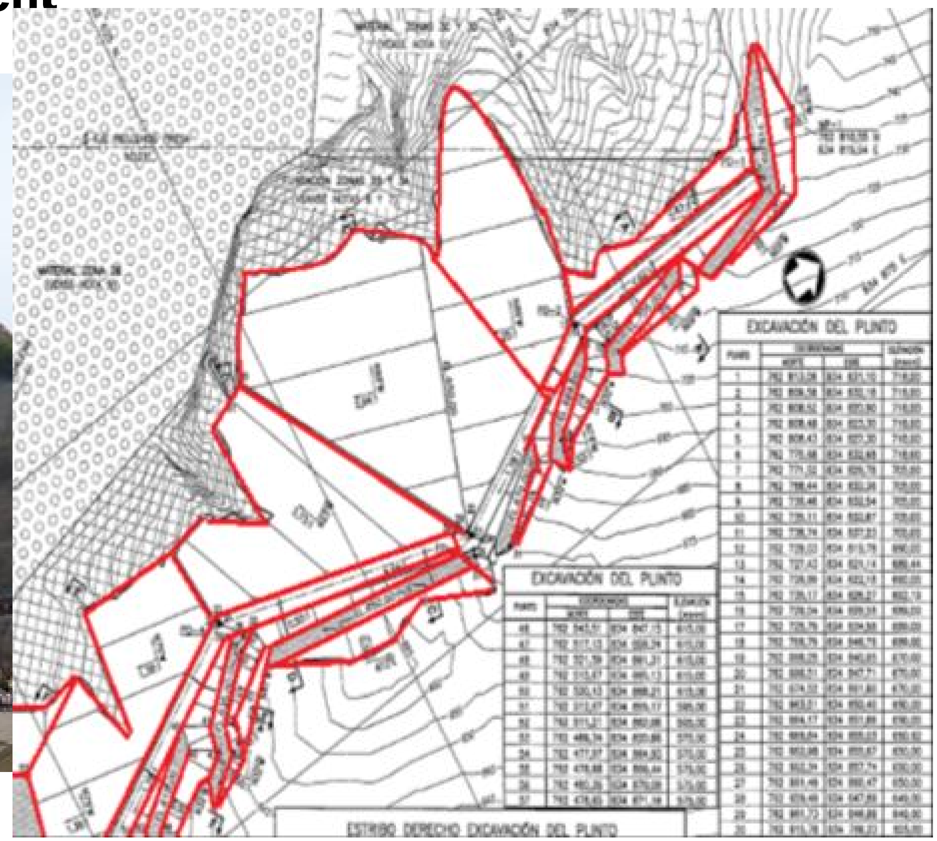
Plinth: Excavations for the left abutment



Plan view: Excavations of the left abutment for 6 sectors of the plinth

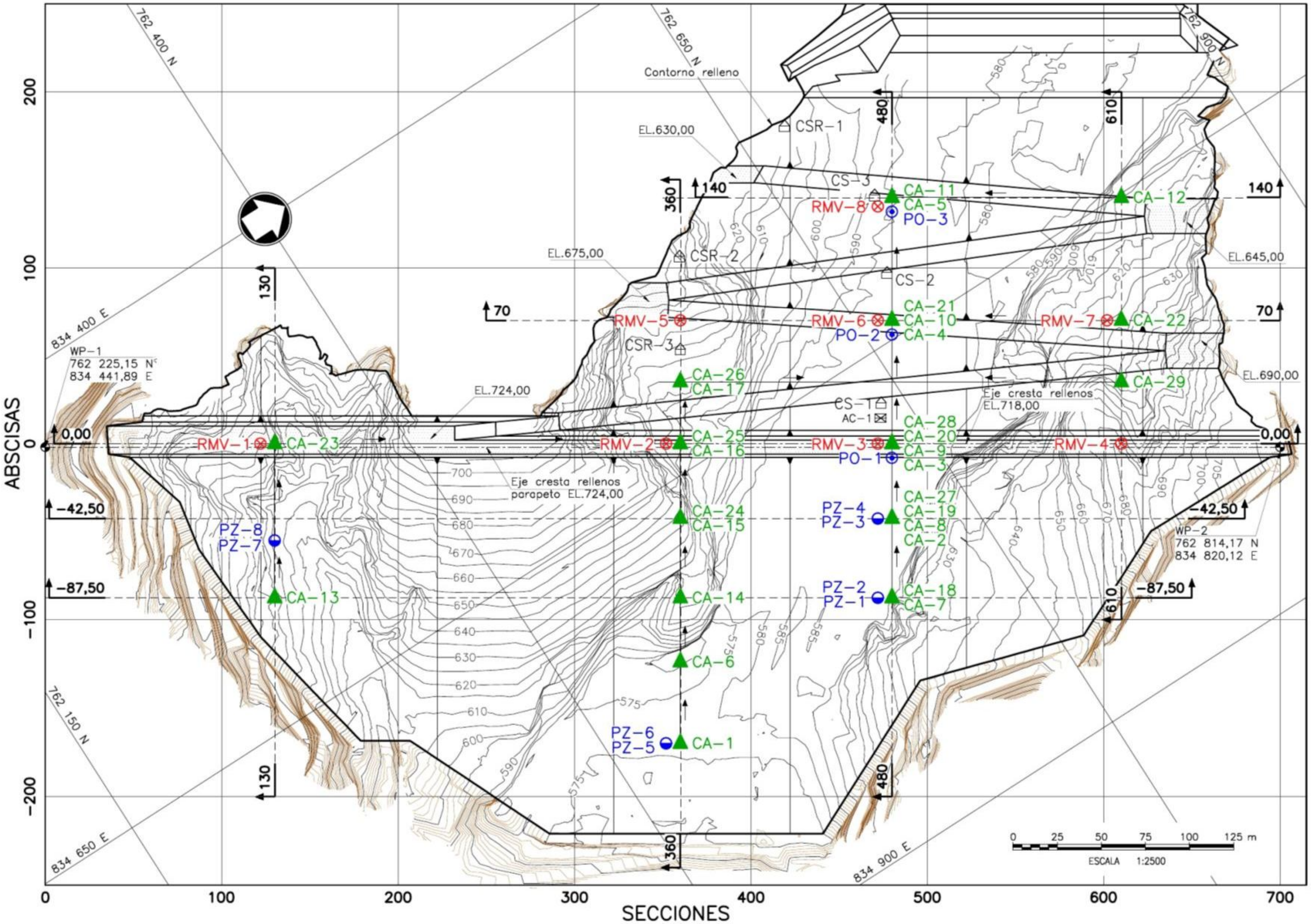
HYDROELECTRIC PROJECT EL QUIMBO

Plinth: Excavations for the right abutment



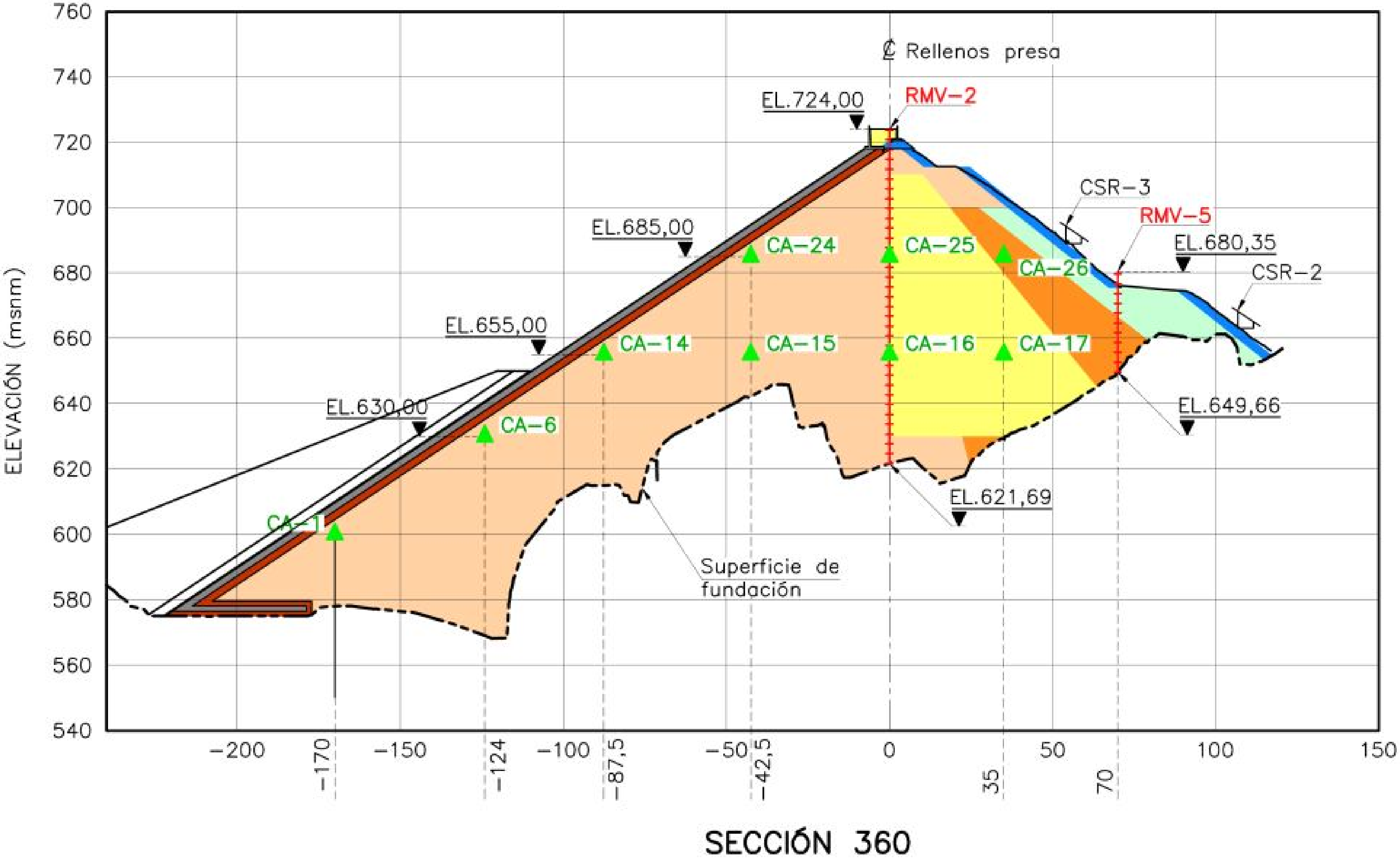
Plan view: Excavations of the right abutment for 4 sectors of the plinth

HYDROELECTRIC PROJECT EL QUIMBO - Instrumentation



INGETEC INGENIEROS CONSULTORES

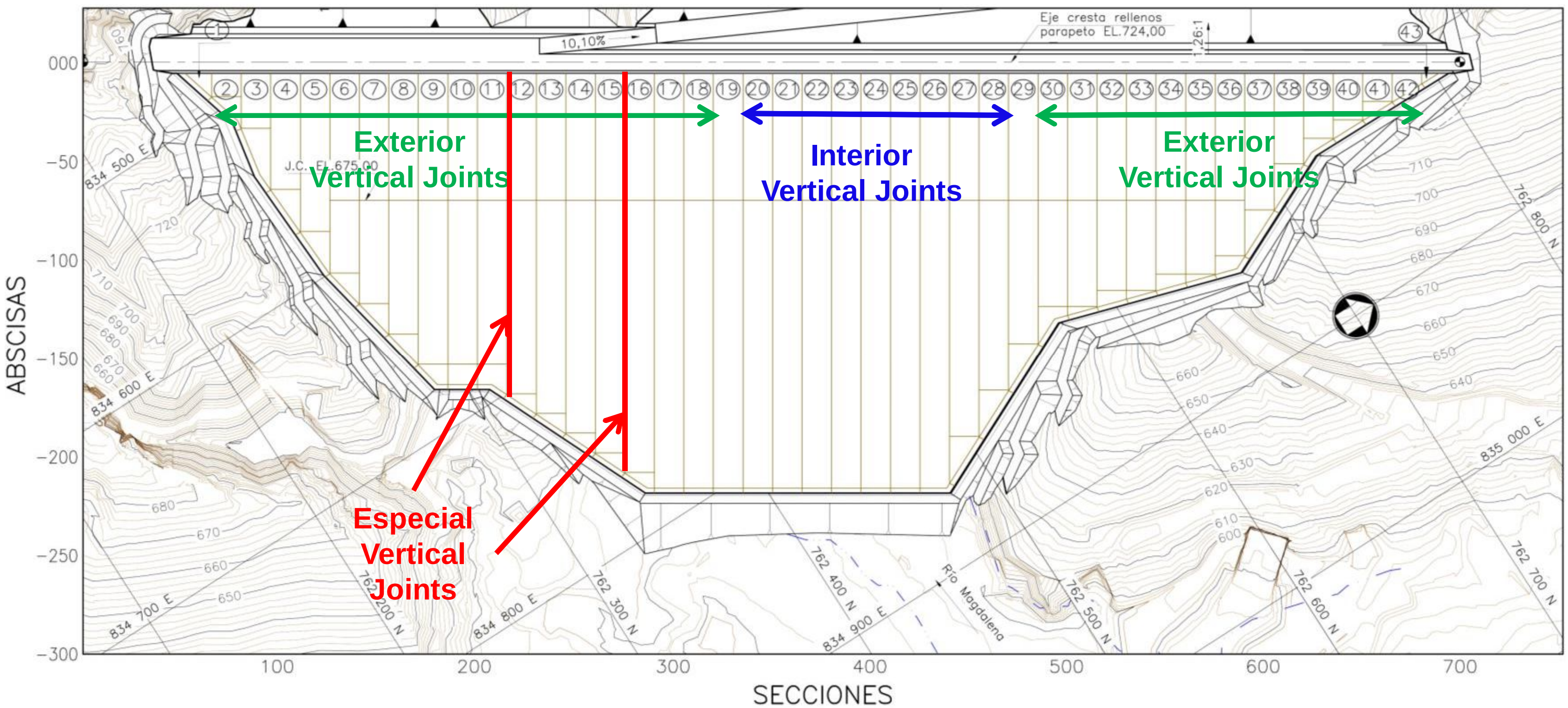
HYDROELECTRIC PROJECT EL QUIMBO – Dam Sections Instrumentation



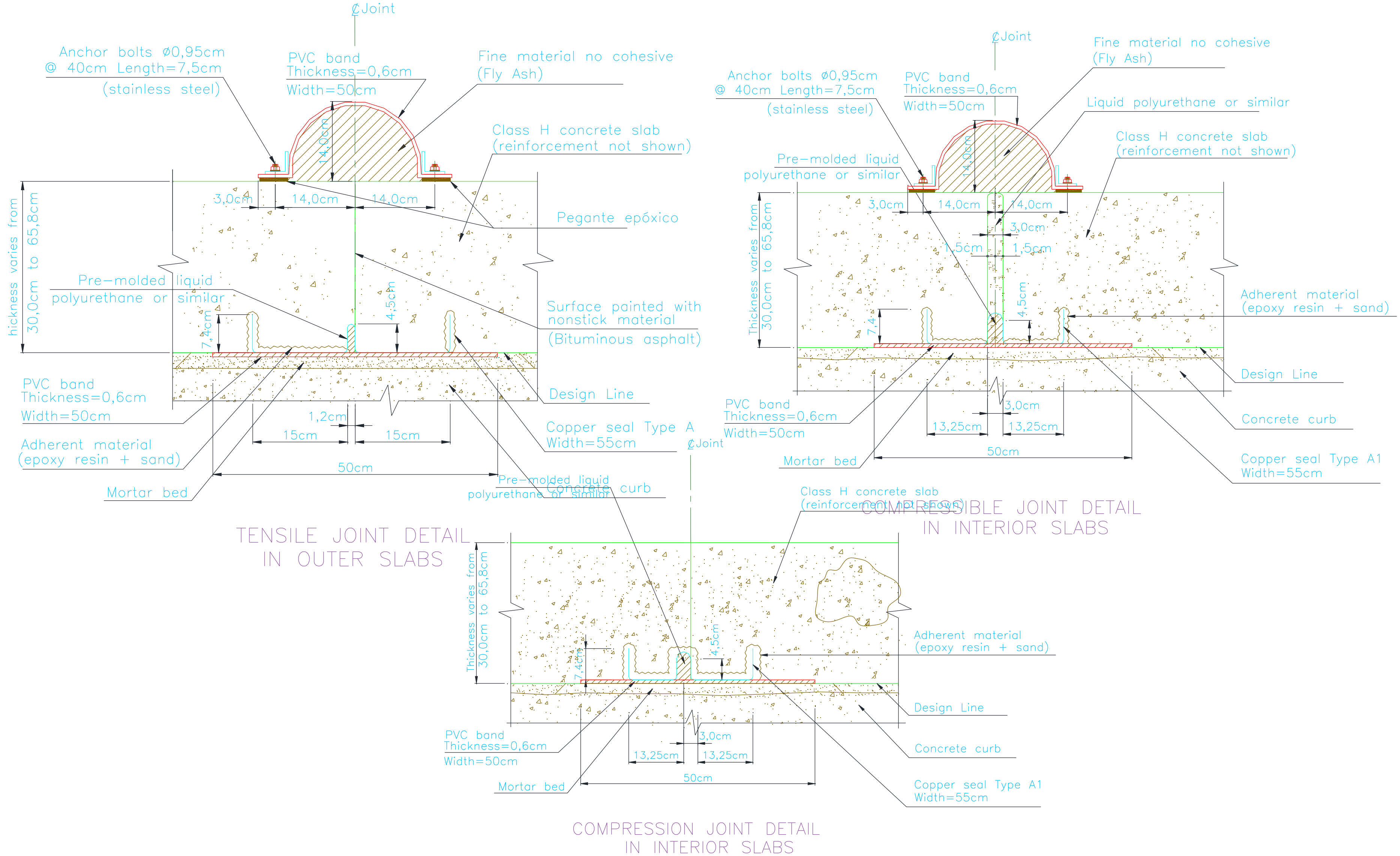
INGETEC INGENIEROS CONSULTORES

CONCRETE FACE

VERTICAL JOINTS – DETAILS



CONCRETE FACE
VERTICAL JOINTS – DETAILS



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HYDROELECTRIC PROJECT EL QUIMBO

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HYDROELECTRIC PROJECT EL QUIMBO

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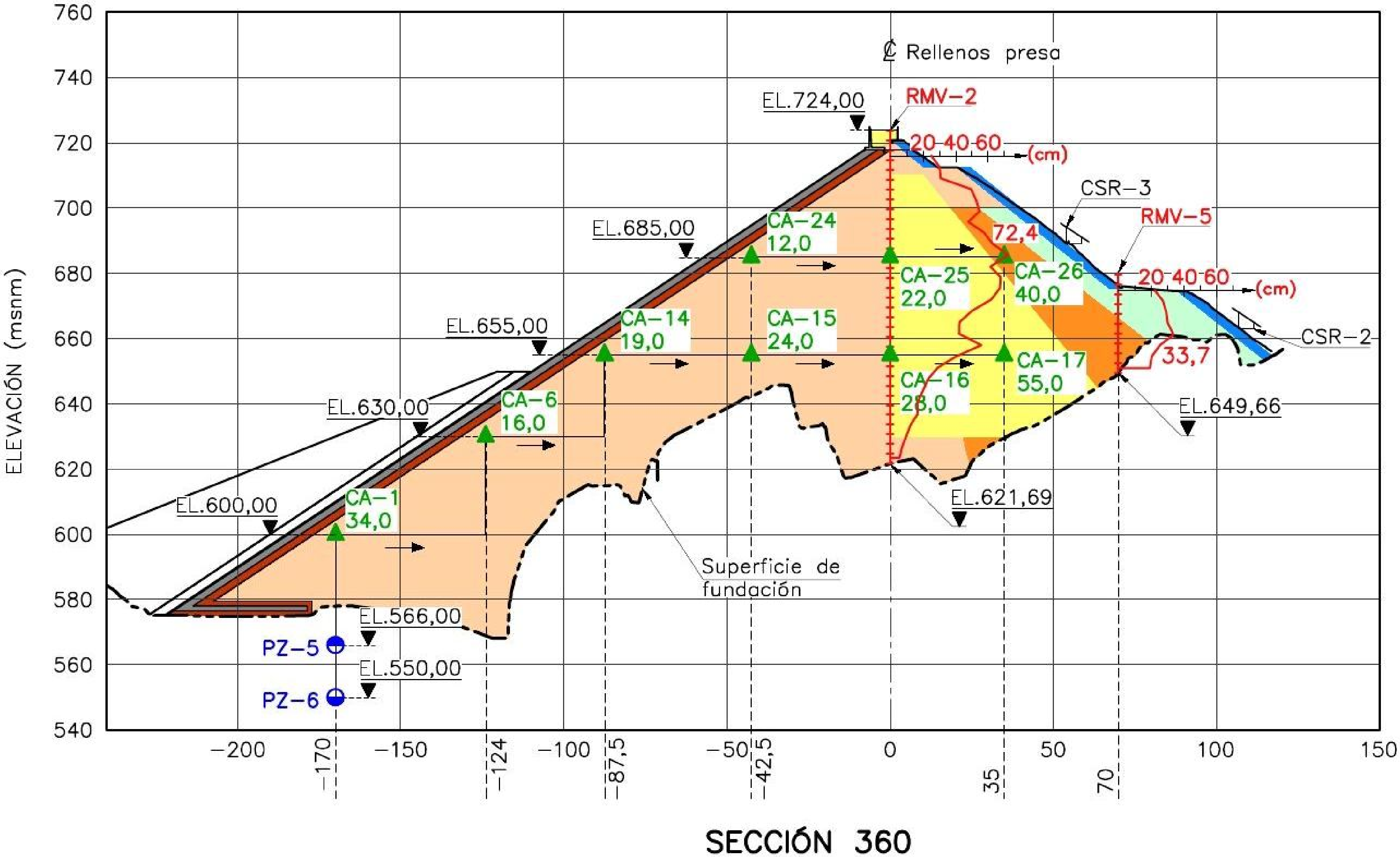
HYDROELECTRIC PROJECT EL QUIMBO

INGETEC INGENIEROS CONSULTORES

HYDROELECTRIC PROJECT EL QUIMBO

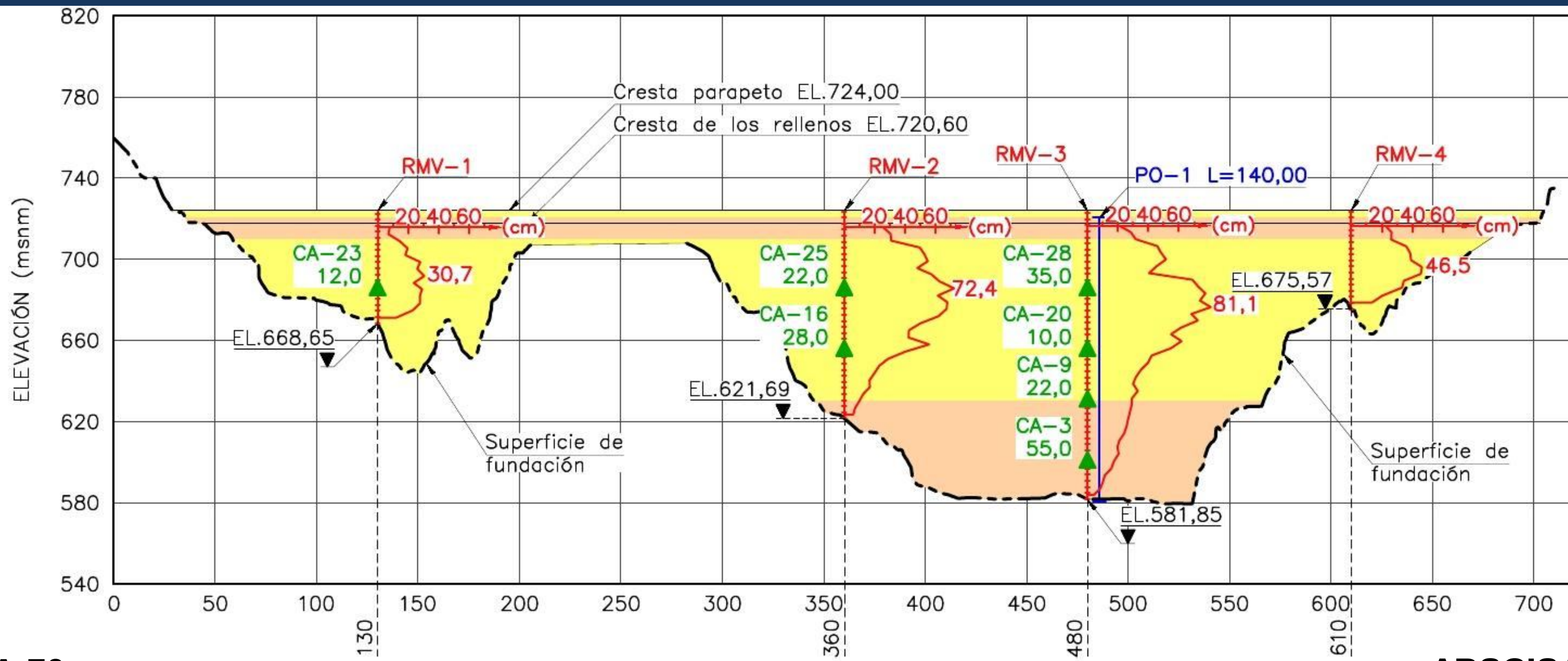
HYDROELECTRIC PROJECT EL QUIMBO – Dam Sections Instrumentation



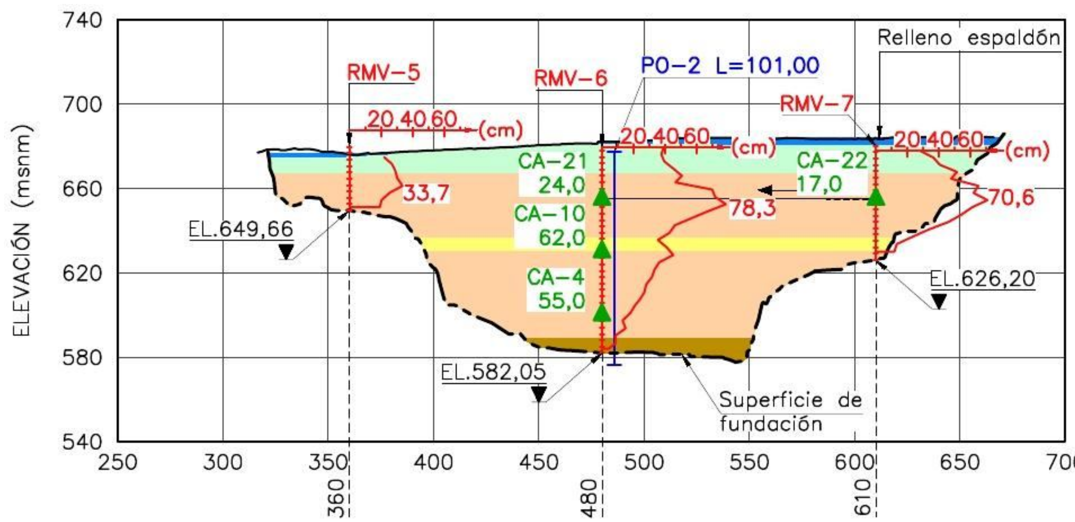
INGETEC
INGENIEROS CONSULTORES

HYDROELECTRIC PROJECT EL QUIMBO Instrumentation

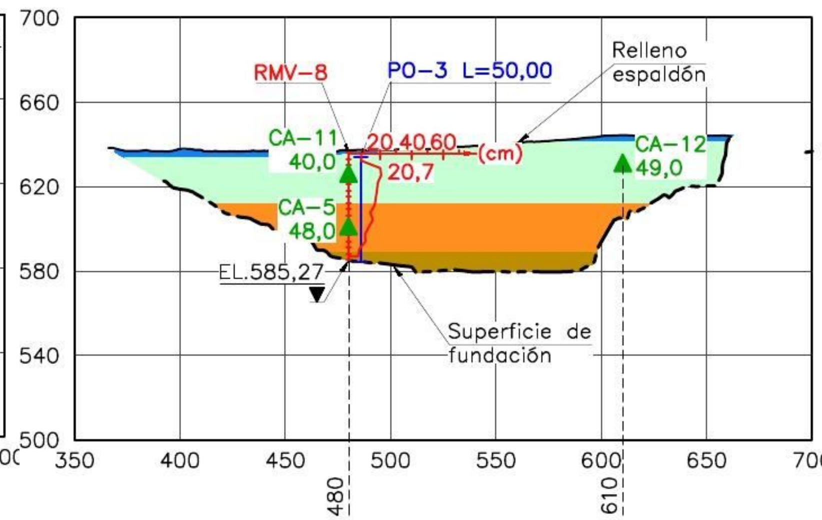
ABSCISA 0,00



ABSCISA 70

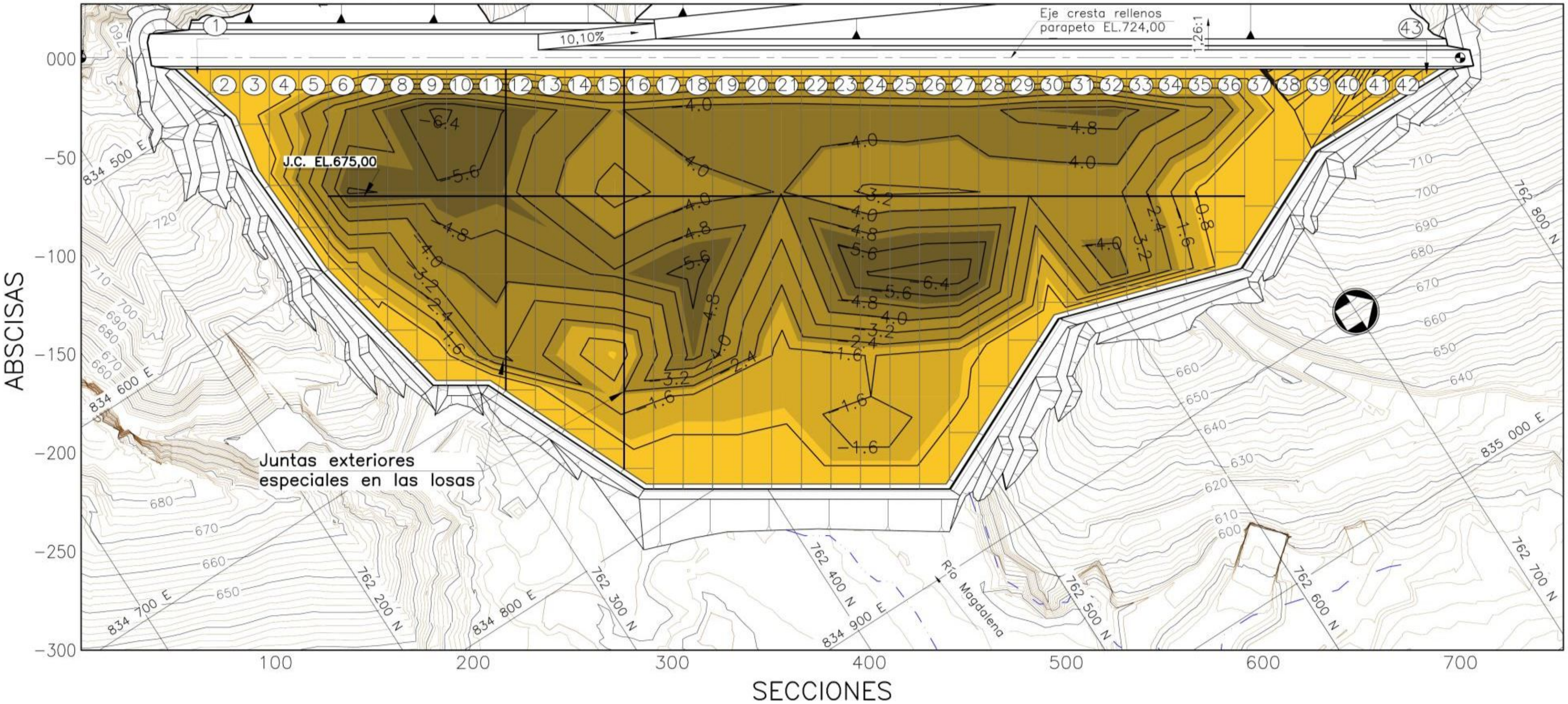


ABSCISA 140



INGETEC INGENIEROS CONSULTORES

CONCRETE FACE
SETTLEMENT MEASUREMENTS



SETTLEMENT (cm)			
1	0,00	-1,44	
2	-1,44	-2,40	
3	-2,40	-2,74	
4	-2,74	-3,40	
5	-3,40	-5,90	

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CONCLUSIONS

- The design and development of CFRD dams, have been based primarily on precedent and empiricism, however, recent incidents have shown that the extrapolation of precedent with the current procedures can have serious consequences. The framework described provides a rational and systematic approach for evaluating the rockfill properties that complemented with the use of numerical methods to predict the response of CFRD's which will consequently contribute to the proper evolution of these dams. It will enhance and integrate precedent with numerical modeling

CONCLUSIONS

- Based on the observed behavior, it can be concluded that the main problem in the dams that have behaved adversely was the characteristics of the rockfill, which confirmed that a "good" rockfill is not defined by the existence of hard fill particles but by its gradation, key in obtaining a less deformable rockfill. This issue was well recognized several decades ago by Marsal, however, is likely that given the pressure to further reduce costs and time for this already economical dam, this fundamental knowledge on rockfill behavior was somehow overlooked. The adequate processing of a rockfill, including gradation, sluicing and compaction are essential to obtain an adequate behavior of a rockfill dam

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CONCLUSIONS

- The physical mechanisms involved in the interaction of the different structural components of a CFRD dam are a very complex to model; however, with the current computational capabilities available, the development of very sophisticated analyses can than aid considerably the design process of CFRD dams. The analyses are quite useful to evaluate the effectiveness of different mitigation measures to alleviate stresses in the concrete face. Nevertheless, the results of current numerical analysis cannot be taken as absolute and precise values. The analyses should point up tendencies and estimates of stress – strain behavior of the different components of the dam, so that engineers with good judgment can make further decisions

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