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Geotécnia e Ingeniería Sísmica aplicadas a la Minería

San Juan, Argentina, 16 de Octubre de 2007

CÁMARA MINERA DE SAN JUAN

AGENCIA ESPAÑOLA DE COOPERACIÓN INTERNACIONAL

SECRETARIA DE ESTADO DE MINERIA

Jornada “Geotecnia e Ingeniería Sísmica aplicada a la Minería”

LA ROTURA DE LA PRESA DE AZNALCÓLLAR

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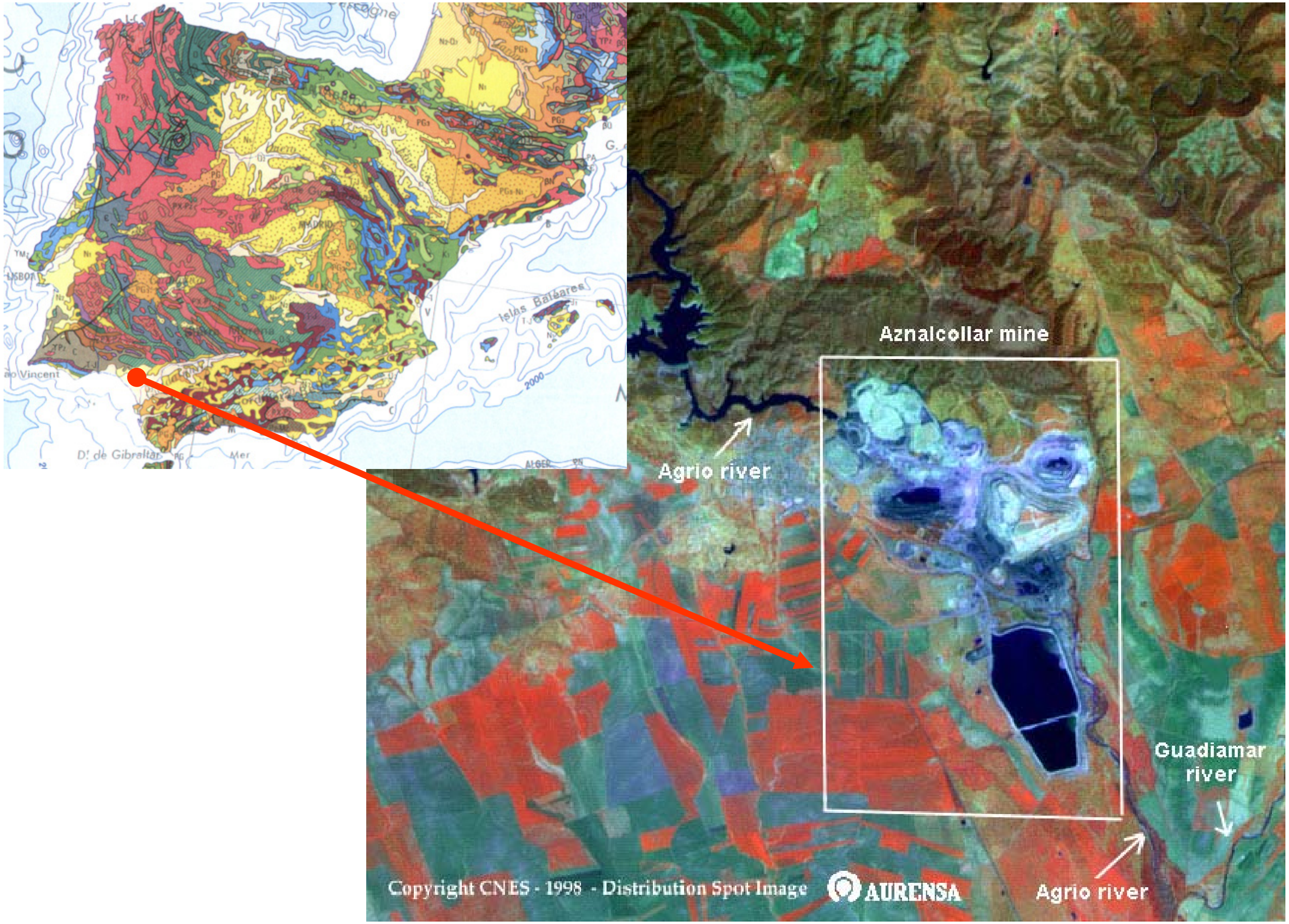
San Juan, Argentina, Martes 16 de Octubre de 2007

Location of Aznalcóllar tailings deposit

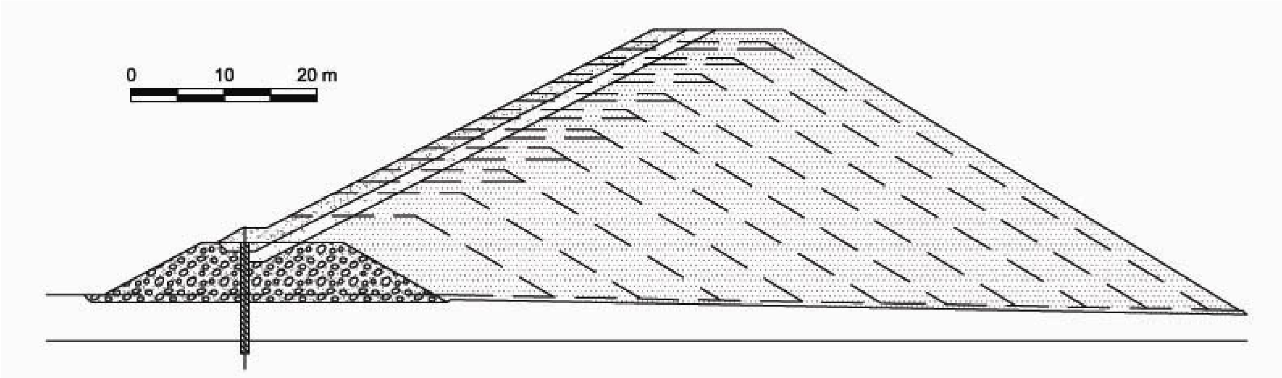


1000 m

Aznalcóllar dam failure

- Date and time of failure: April 25, 1998; early morning
- 1.3 Mm^3 of tailings and 5.5 Mm^3 of acid water flew out of the pond
- 24 km of the valleys of Agrio and Guadiamar rivers were inundated
- Emergency in Doñana National Park
- Social alarm. Public opinion was deeply involved
- Uncertainty on the reasons for the failure

Aznalcóllar dyke according to original project (1977)



FAILURE CAUSES PUBLISHED IN NEWSPAPERS

“Rafael Baena Escudero of the Department of Physical Geography and Regional Geographic Analysis stated: "In this case, a complete lack of foresight emerged. The dam was built on top of ***expansive clays****. Within these clays, deformations have occurred, which were propagated to the soil, readjusting the blocks whenever a movement occurred. In this sense, the seepage through the marls has the effect that these layers, the phylosilicates, swell and expand their volume. The opposite happens when they dry out and force the shrinking of the clay. This movement of expansion/contraction is constant and should have been accounted for. Especially, after the inclinometers had become deformed: something was moving. -* ***This is a matter of general negligence and not a problem of nature****." (El Mundo, May 25, 1998)”*

FAILURE CAUSES PUBLISHED IN NEWSPAPERS

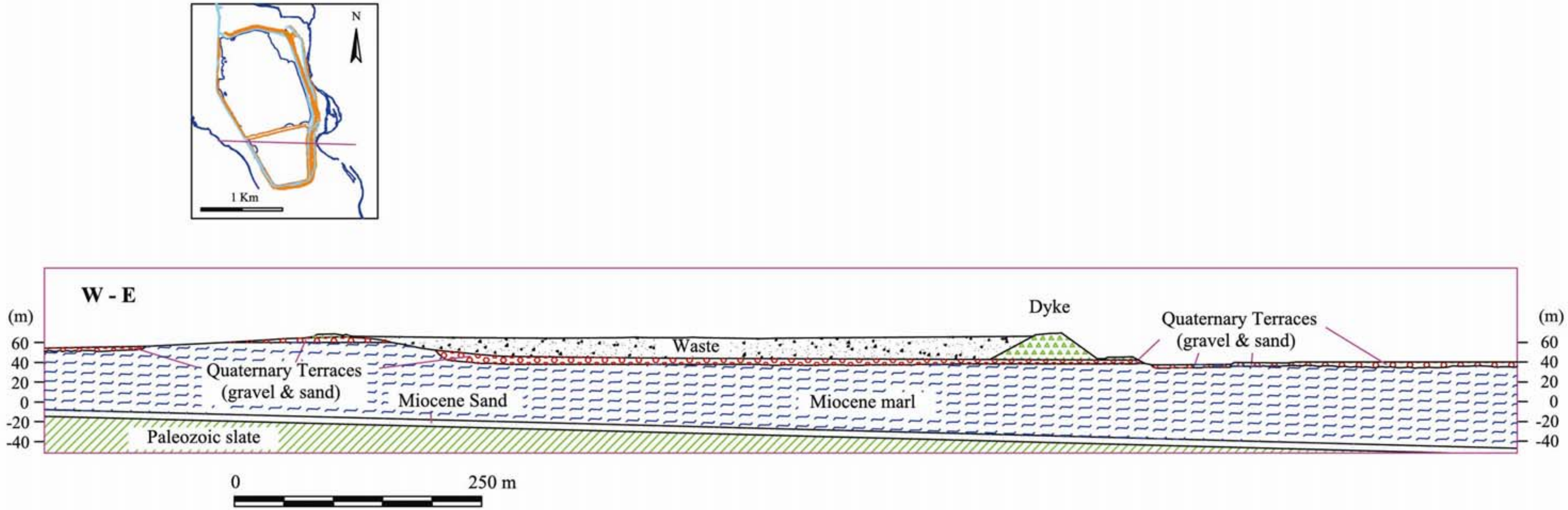
"Some unnamed geotechnical experts cited by El Mundo (May 19, 1998) suggest that the foundation failure was caused from ***chemical attack*** *of the impounded acidic pyritic slurries on the marl forming the dam foundation material. Marl consists of clay and calcium carbonate ($CaCO_3$). The calcium carbonate contained in this marl decomposed under the acid attack, deteriorating the mechanical stability of the soil".*

"The ***acidic seepage, combined with the continued blasting in the nearby open pit mine****, is also identified as the most probable hypothesis for the cause of the failure by Luis Berga, expert of the Universitat Politecnica de Catalunya . He presented the results of his study on 19 June in Barcelona at the Congress of the International Commission on Large Dams . (La Vanguardia of June 20, 1998)"*

Outline of Presentation

1. Geological and geotechnical observations
2. The geometry of the failure
3. Geotechnical characteristics of tailings
4. Geotechnical characteristics of foundation clay
5. Water pressures and stresses in the foundation
6. Failure analysis. Limit equilibrium
7. Failure analysis. Finite elements
8. Influence of dip of sedimentation planes
9. Dynamics of failure
10. Aznalcóllar failure and related cases
11. Final remarks

Representative profile



- Granular upper alluvial of Agrio river. Thickness: 4-5 m
- Thickness of marine clays (mio-pliocene): 60m
- Dip of sedimentation planes: 2º à 4º towards SSE
- Confined lower aquifer. Piezometric level at the surface
- Upper layer (2 to 5 m of thickness) of oxidized marl

1. Geological and geotechnical observations

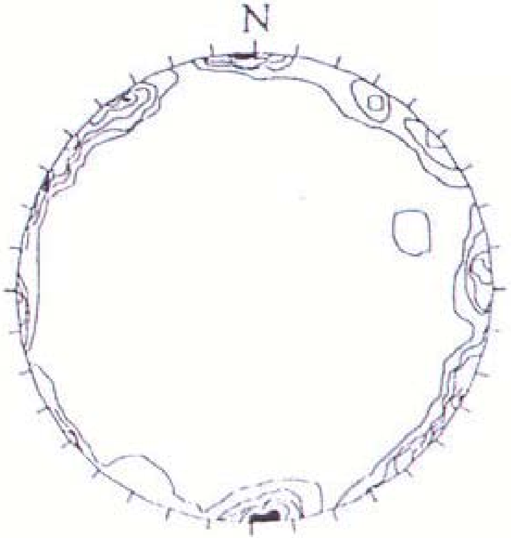
Sedimentation planes

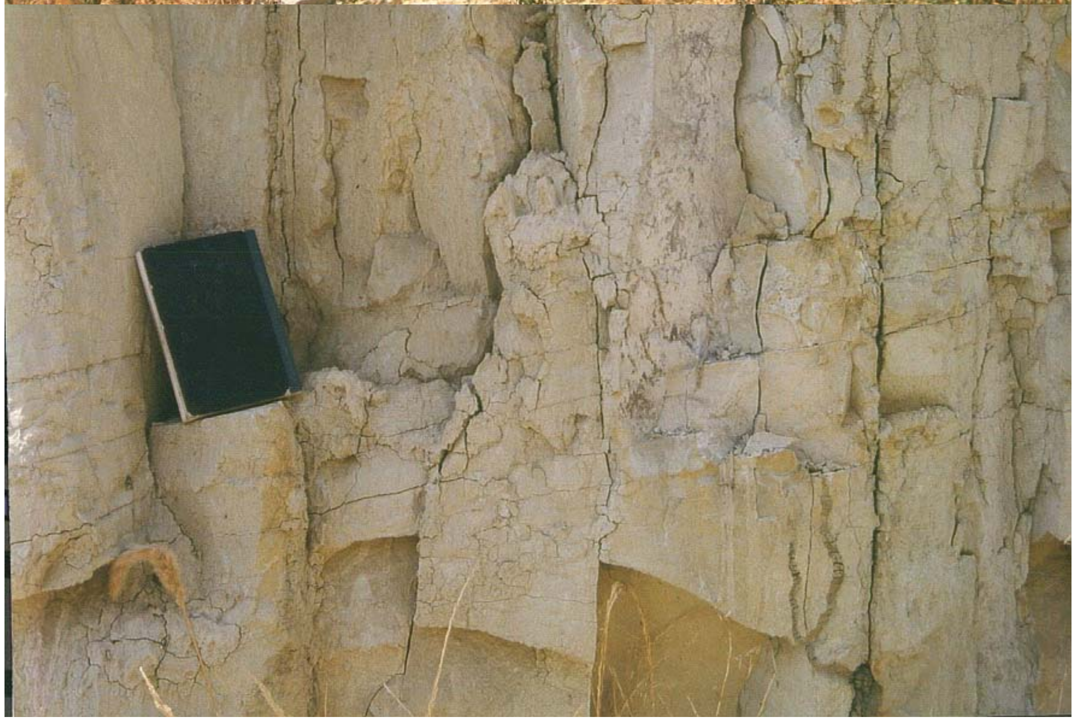
- Quasi-horizontal stratification
- Slickensides detected at some places
- High continuity (>40m)

1. Geological and geotechnical observations

Joints

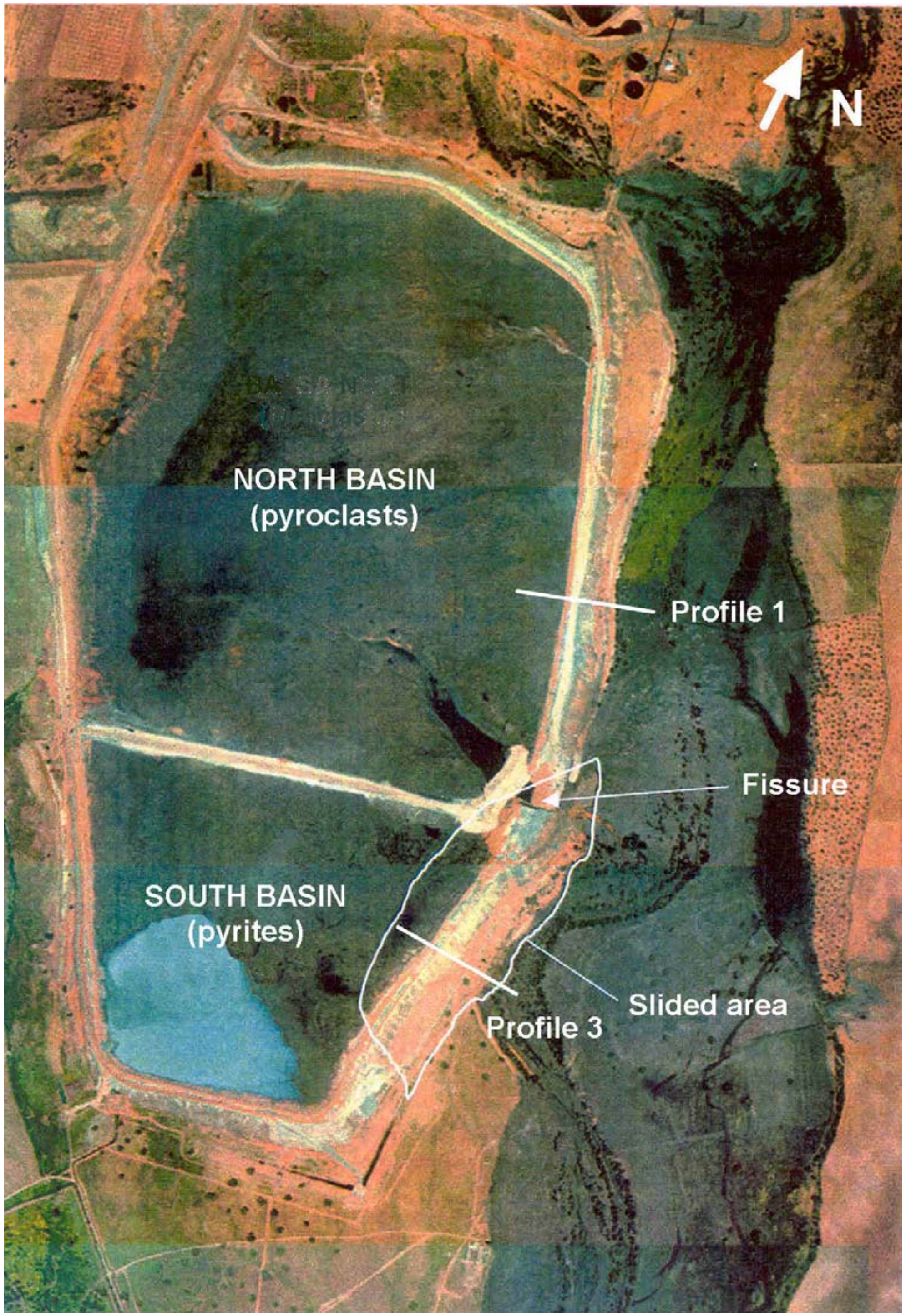
- Vertical dip
- Continuity in the range2-5 m
- Polished surfaces
- Three families identified
- Dominant family: NE-SW



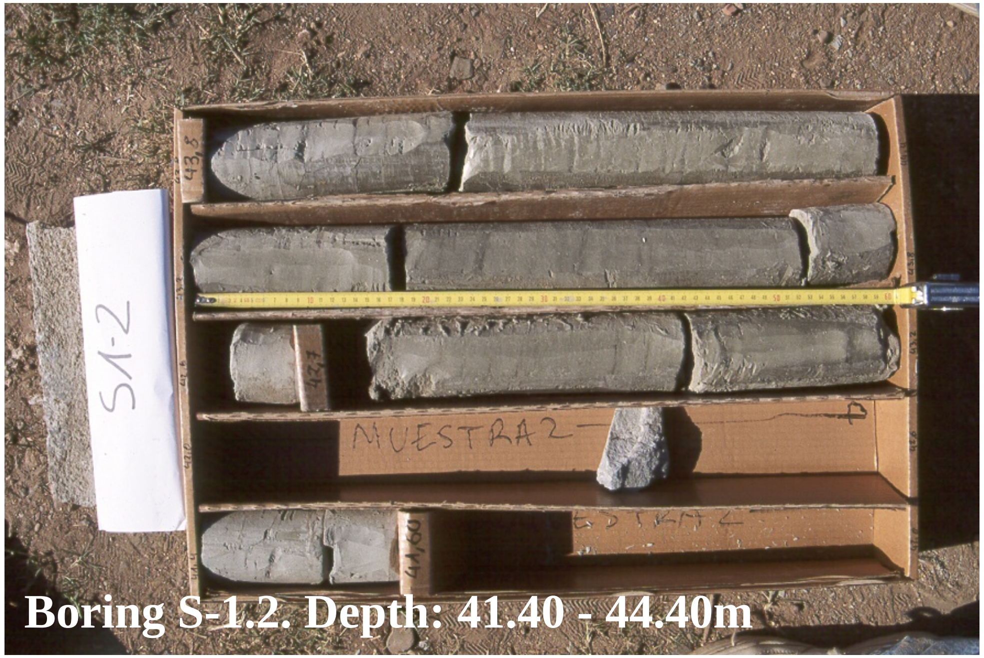


1. Geological and geotechnical observations

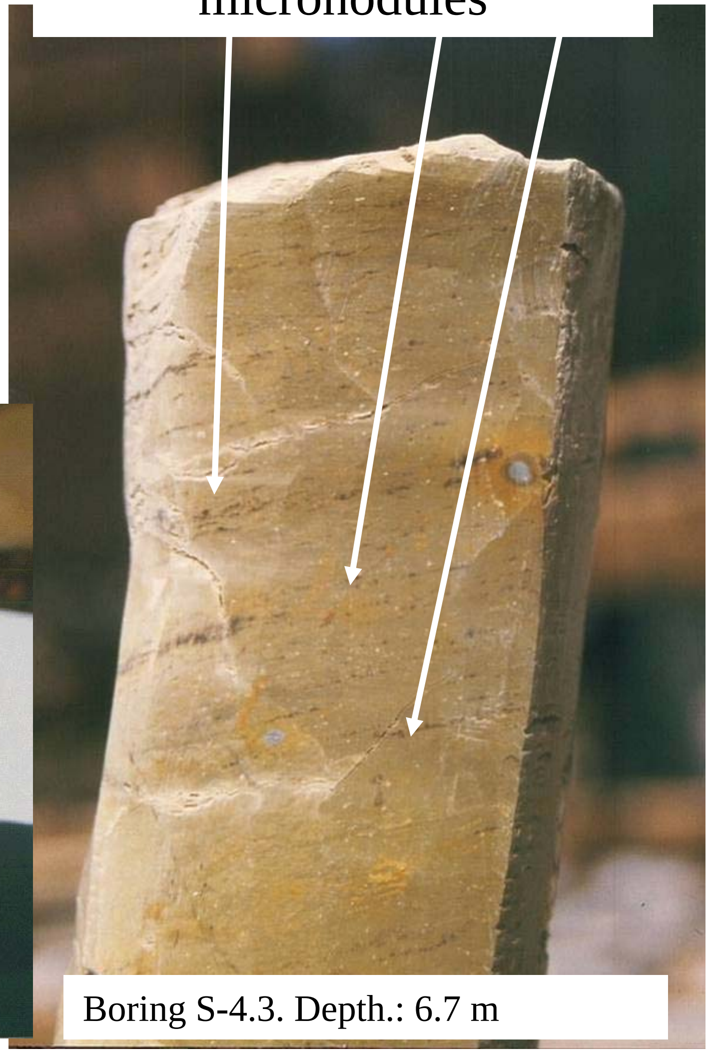
- View of the deposit after the failure
- Boreholes located in five profiles normal to dyke orientation
- Note the position of Profiles 1 and 3



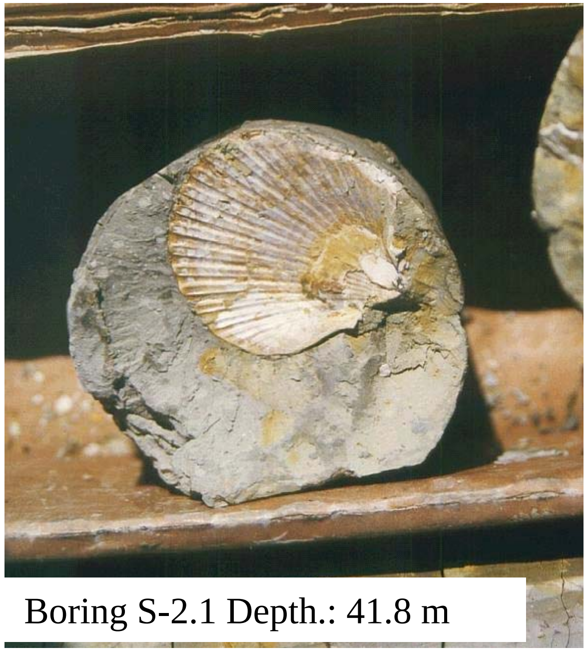
Observations in cores

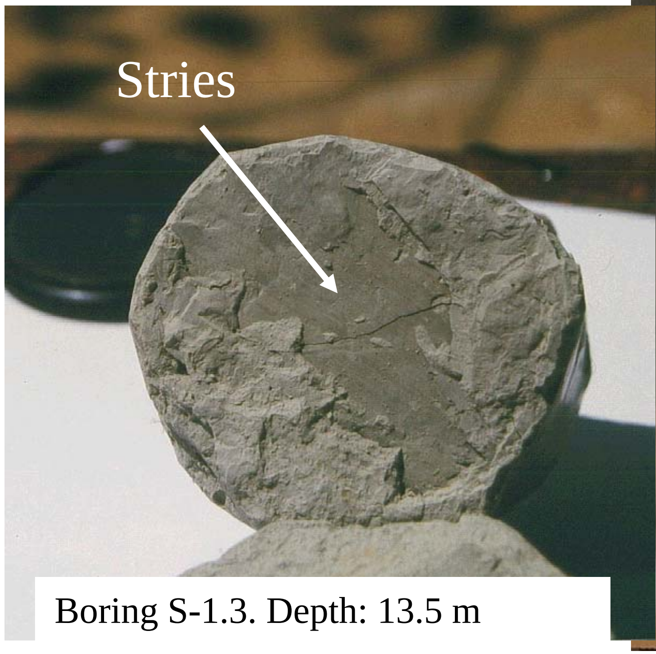


Boring S-1.2. Depth: 41.40 - 44.40m



Boring S-4.3. Depth.: 6.7 m


Boring S-2.1 Depth.: 41.8 m



Boring S-1.3. Depth: 13.5 m

1. Geological and geotechnical observations

Clay blocks « floating » on the tailings flow

Note:

- Parallelepipedic shape
- Sharp edges
- Joints covered by oxidation coatings

1. Geological and geotechnical observations

Discontinuity surfaces observed in blocks and excavations.
Note:

- Polished surfaces
- Slickensides

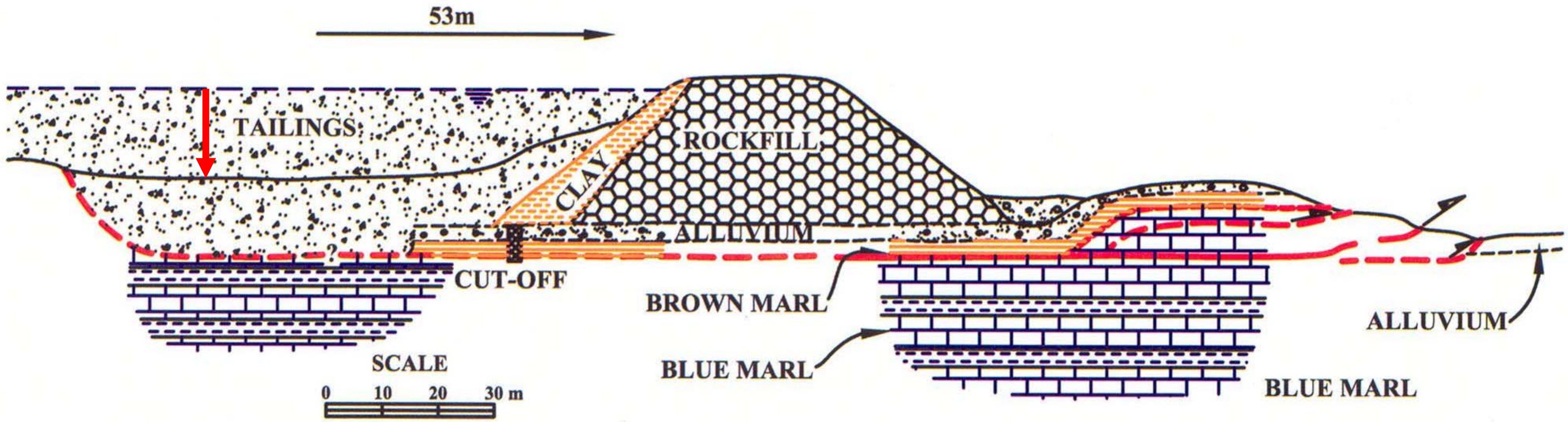
SEDIMENTATION PLANES

- Dipping 2º-4º towards SSE
- High degree of continuity (> 30 m)
- Spacing: Dense stratification bands every 2 m
- Roughness: Planar and smooth surfaces. Occasional slickensides

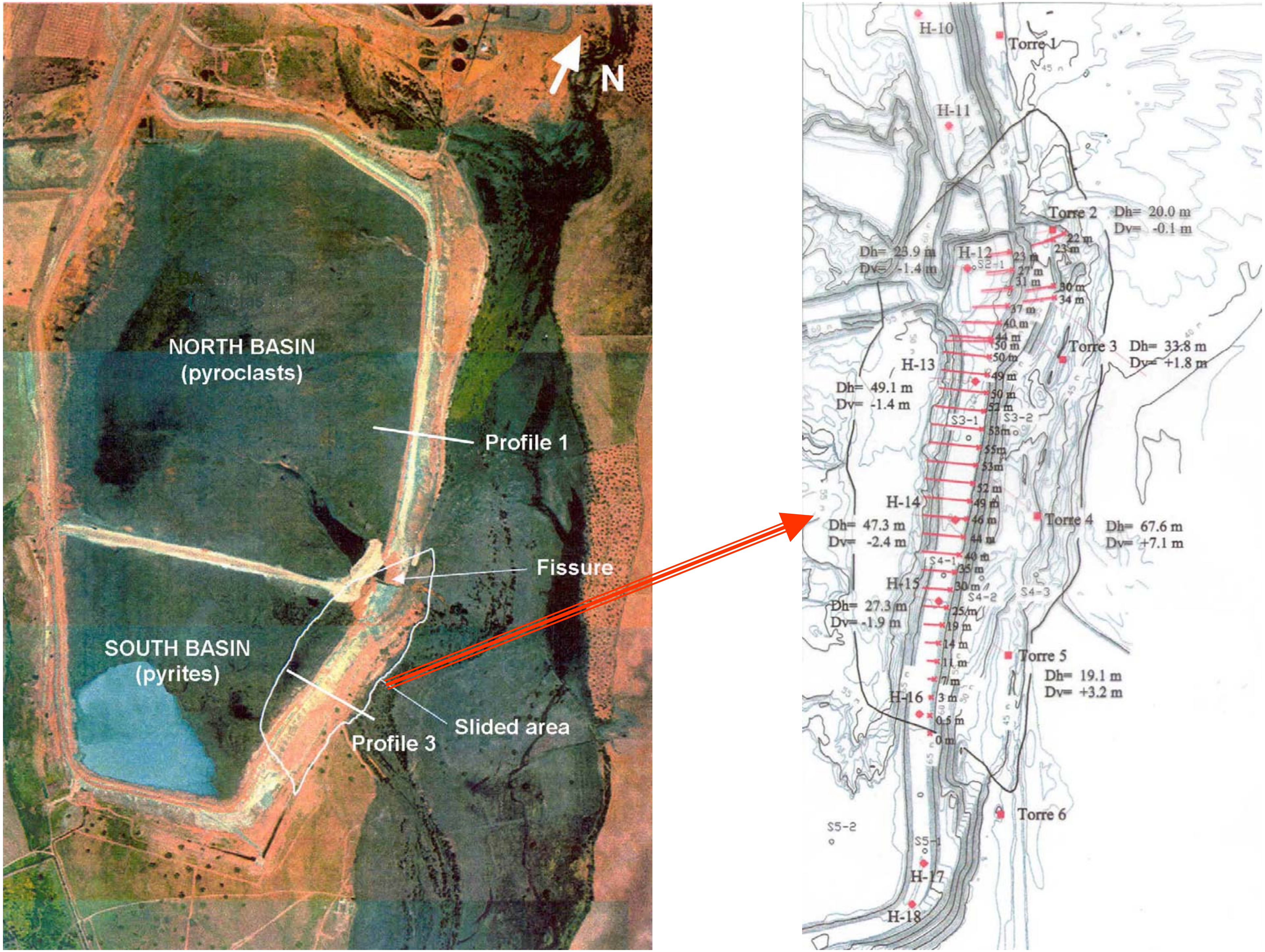
JOINTS

- Quasi-vertical dip (80º-90º)
- Orientation NE-SW dominates
- Continuity: More than 2-5 m
- Spacing: 30-40 cm
- Roughness: Very smooth. Ridges, 10 mm in height, parallel to slickensides (evidence of vertical displacement)

2. The geometry of the failure

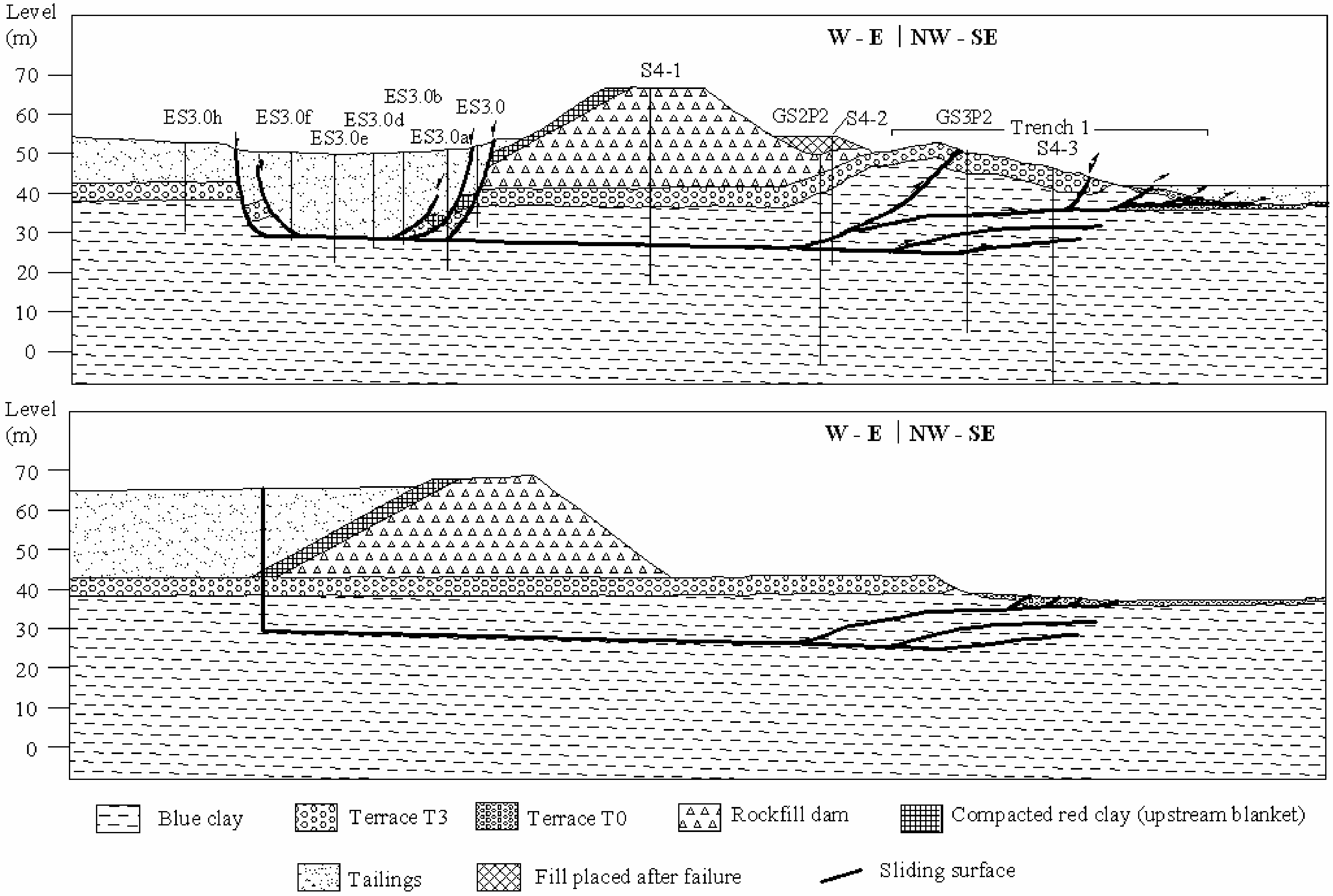


- Basal failure surface located within the blue clay at 12-14m under the surface)
- Solid rigid motion of dyke, upper alluvium and an upper layer of clay
- Downstream accumulation of folded strata
- Upper trough partially filled by tailings



N
NORTH BASIN
(pyroclasts)
Profile 1
Fissure
SOUTH BASIN
(pyrites)
Slided area
Profile 3
H-10
Torre 1
H-11
Torre 2
Dh= 20.0 m
Dv= -0.1 m
Dh= 23.9 m
Dv= -1.4 m
H-12
22 m
23 m
27 m
31 m
30 m
34 m
37 m
40 m
44 m
50 m
50 m
Torre 3
Dh= 33.8 m
Dv= +1.8 m
H-13
Dh= 49.1 m
Dv= -1.4 m
49 m
50 m
52 m
S3-1
S3-2
53m
55m
53m
52 m
49 m
H-14
46 m
44 m
Dh= 47.3 m
Dv= -2.4 m
Torre 4
Dh= 67.6 m
Dv= +7.1 m
40 m
S4-1
35 m
H-15
30 m
S4-2
S4-3
Dh= 27.3 m
Dv= -1.9 m
25 m
19 m
14 m
11 m
7 m
Torre 5
Dh= 19.1 m
Dv= +3.2 m
H-16
3 m
0.5 m
0 m
Torre 6
S5-2
S5-1
H-17
H-18

Central zone of failure. Displaced and reconstituted profiles



(Moya, 2000)

2. The geometry of the failure

The downstream edge of slide

- Apparent heave of the ground (7-8 m of vertical displacement)
- Surface bent and cracked. Cracks parallel to dyke
- Folded layers identified
- The motion had a slight rotation towards the South

2. The geometry of the failure

Head of the slide

- Vertical upstream cliff located in the original position of the foot of the dyke upstream slope
- The slide motion led to the opening of a large upstream depression basin. The red clay mantle became unstable

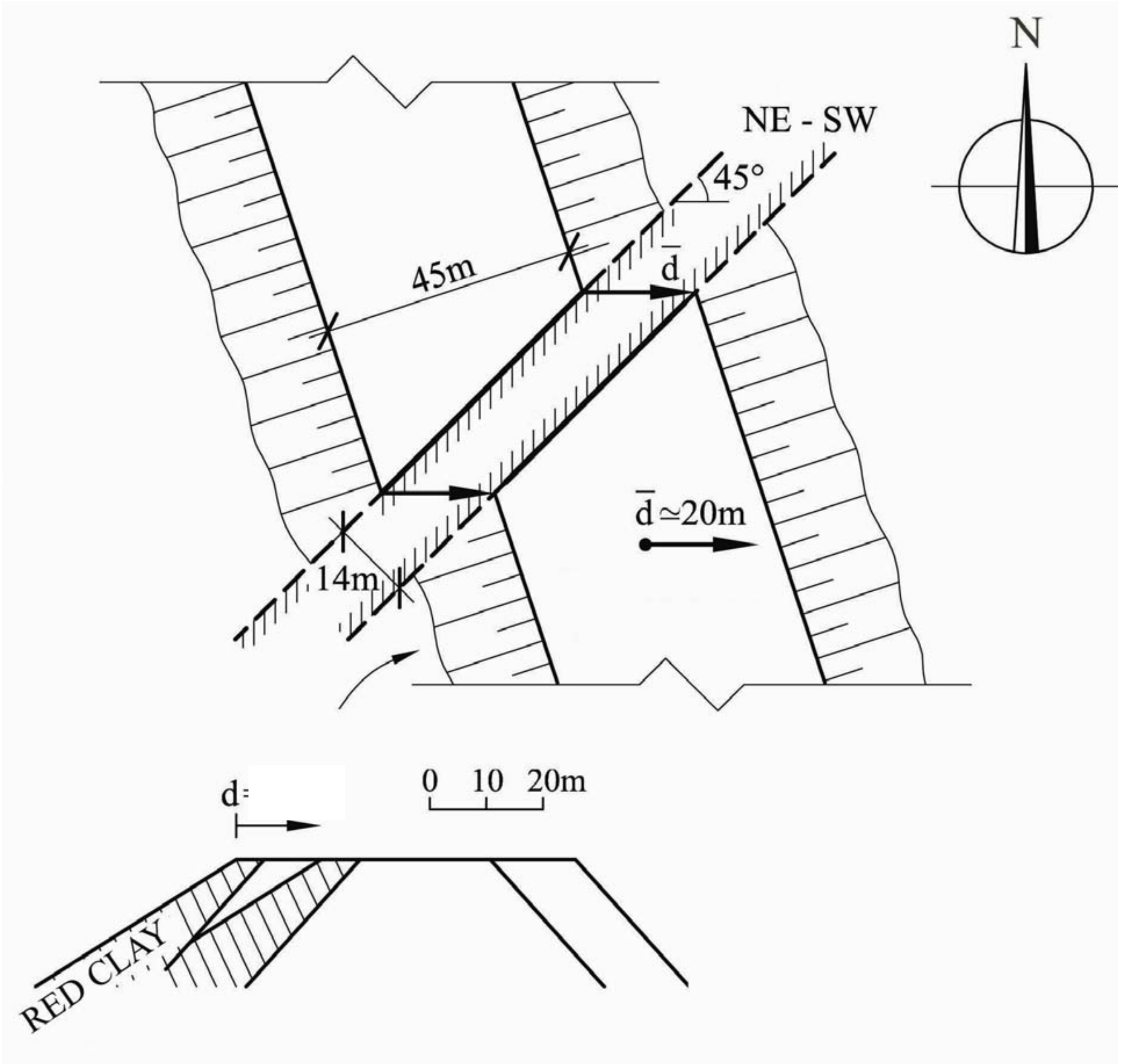
Head of the slide

- Mud volcanoes were observed disseminated on the upstream depression surface

(Cortesía de J.M. Rodríguez Ortiz)

Rupture breach in the dyke

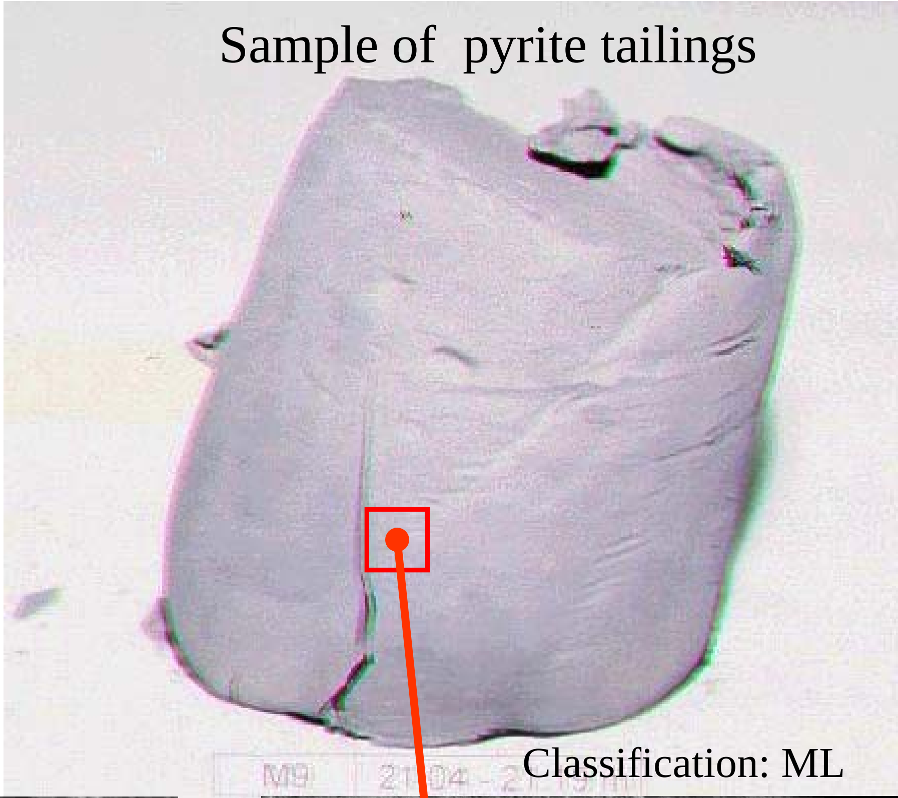
- It was interpreted that the breach orientation was controlled by a joint of the NE-SW family
- The slide motion implies an opening of the breach
- It is estimated that the initial channel had an opening of around 14 m since the dyke was displaced d=20 m in the East direction

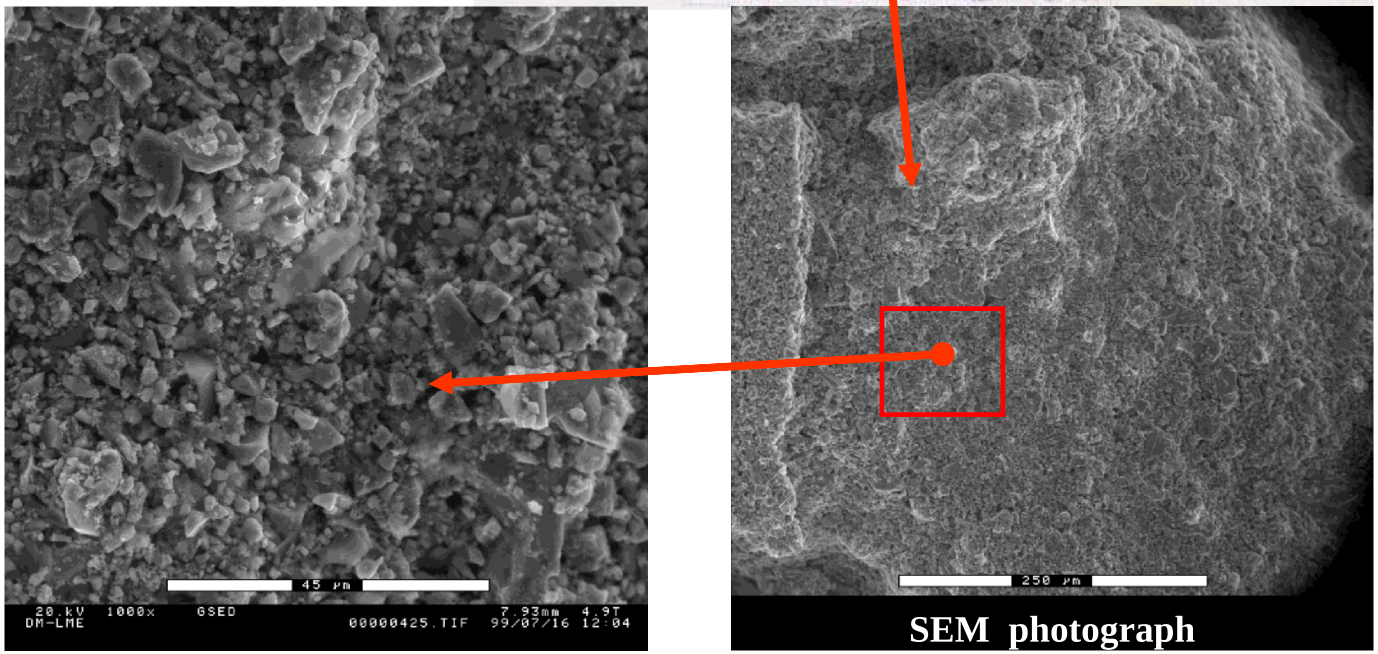


3. Geotechnical characteristics of tailings

- Tailings composition: Pyrite finely crushed + other metallic and non-metallic minerals + chemical compounds
- It is a granular soil: fine sand and silt sizes

➢Objectives: Permeability, cementation and the possibility of static liquefaction

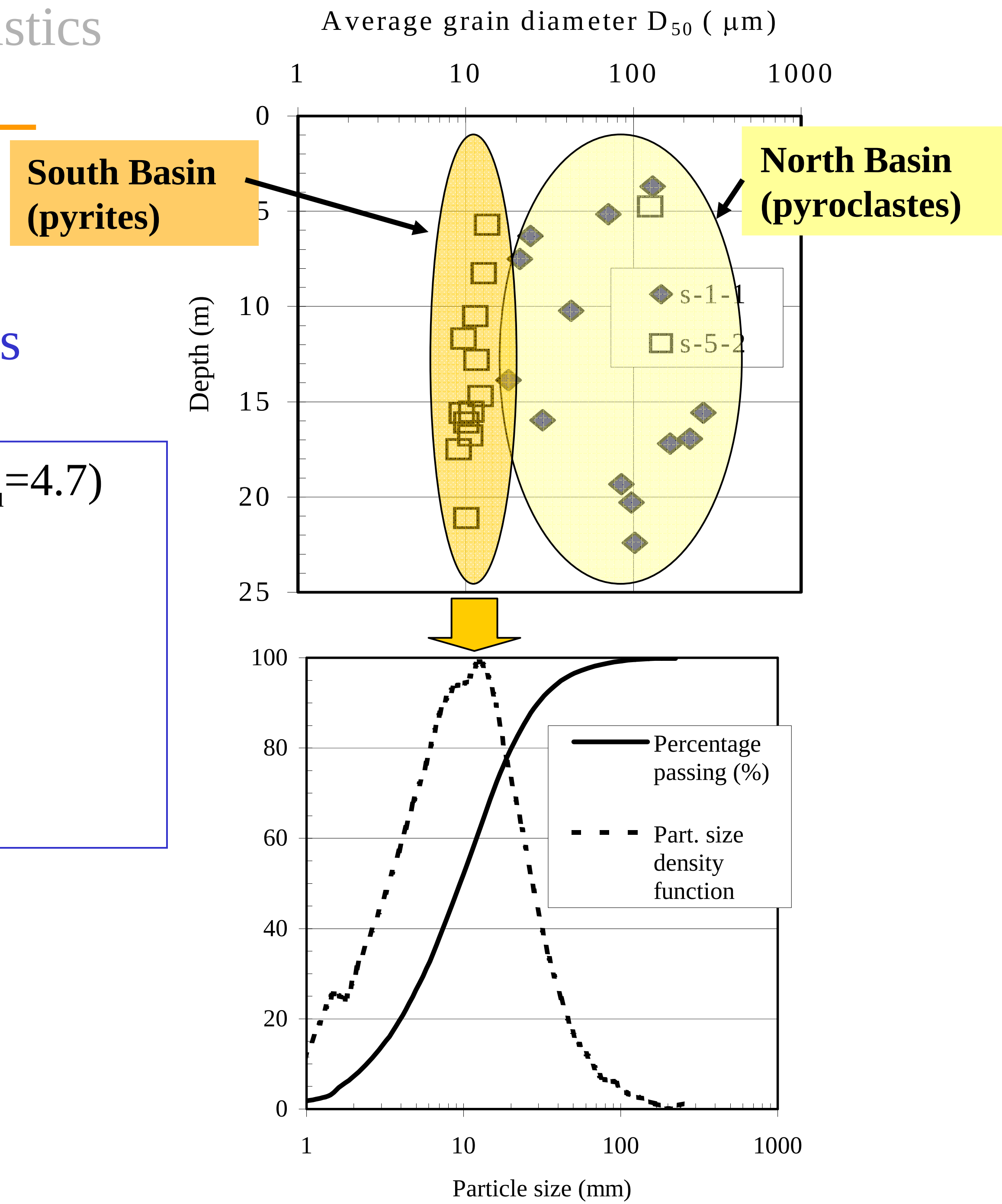




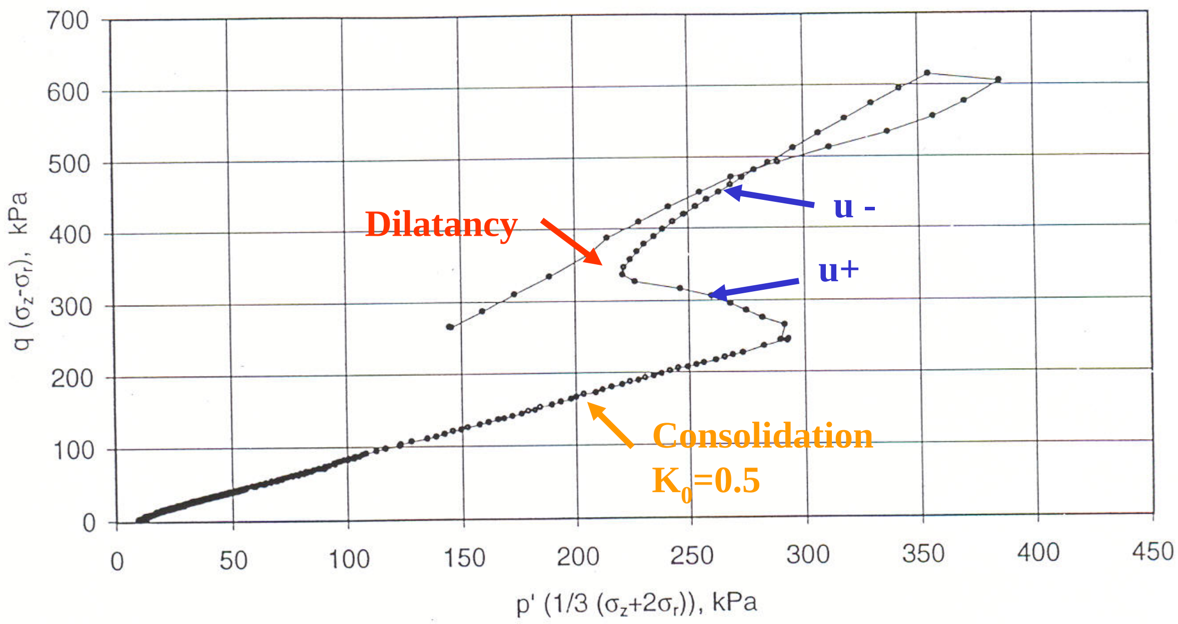
3. Geotechnical characteristics of tailings

South basin. Pyrite tailings

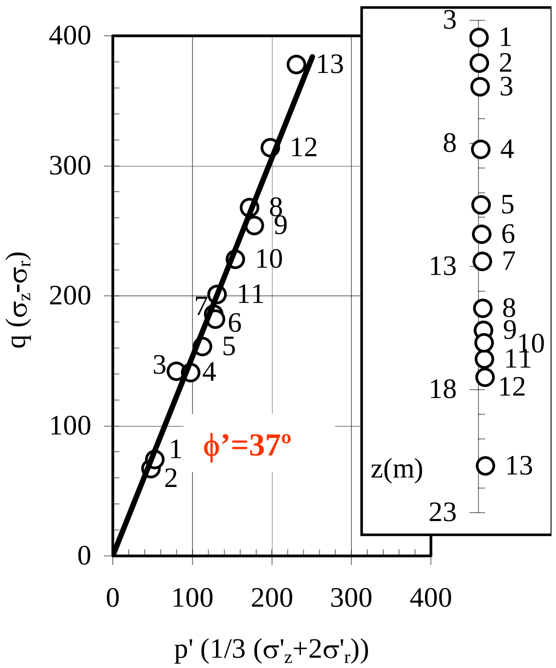
- Homogeneous grain size (C_u=4.7)
- Void ratio: 0.6 to 0.8
- High density of particles:
  - γ_s = 4.3 g/cc
  - γ_{nat} = 3.1 g/cc



Undrained triaxial tests on pyrite specimens



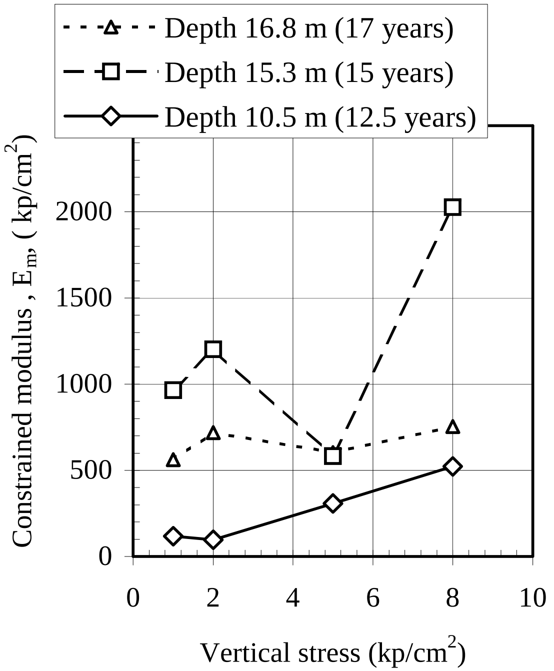
Tests CAU on specimens taken in boring S-5.2



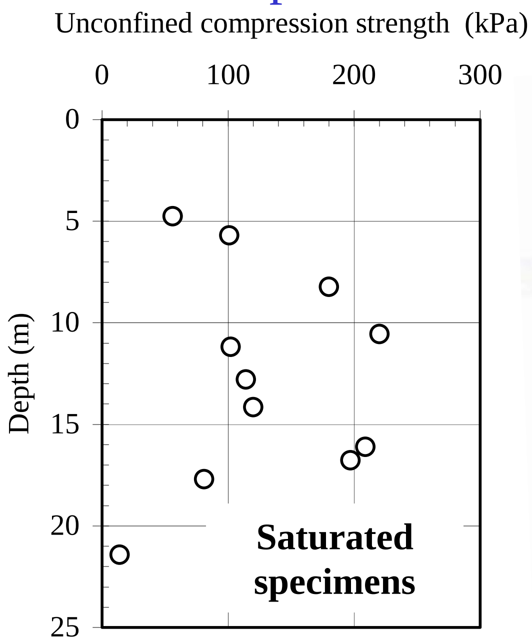
- Static liquefaction was not observed
- Homogeneous failure conditions in depth

Additional tests on pyrite specimens

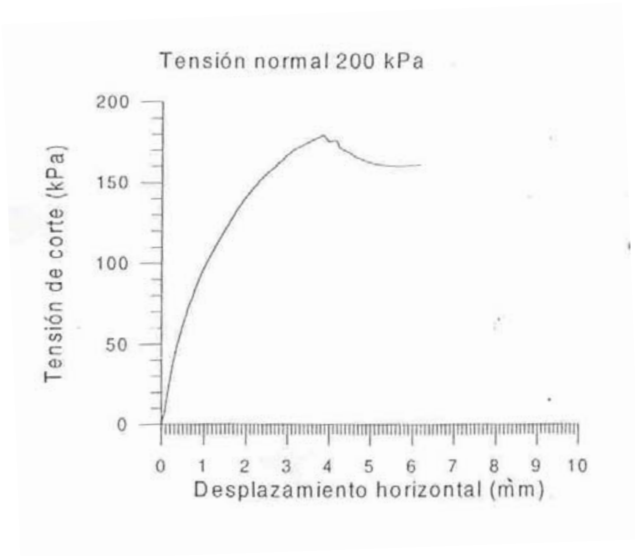
Oedometer



Unconfined compression



Direct shear



- K= 10^{-7} to 10^{-6} cm/s
- E_m increases with age

- q_u=100-200 kPa
- Marked peak behaviour

- Peak: c'=17 kPa; ϕ'=42.6º
- Res: c'=0 kPa; ϕ'=41º

➤ Evidence of significant cementation

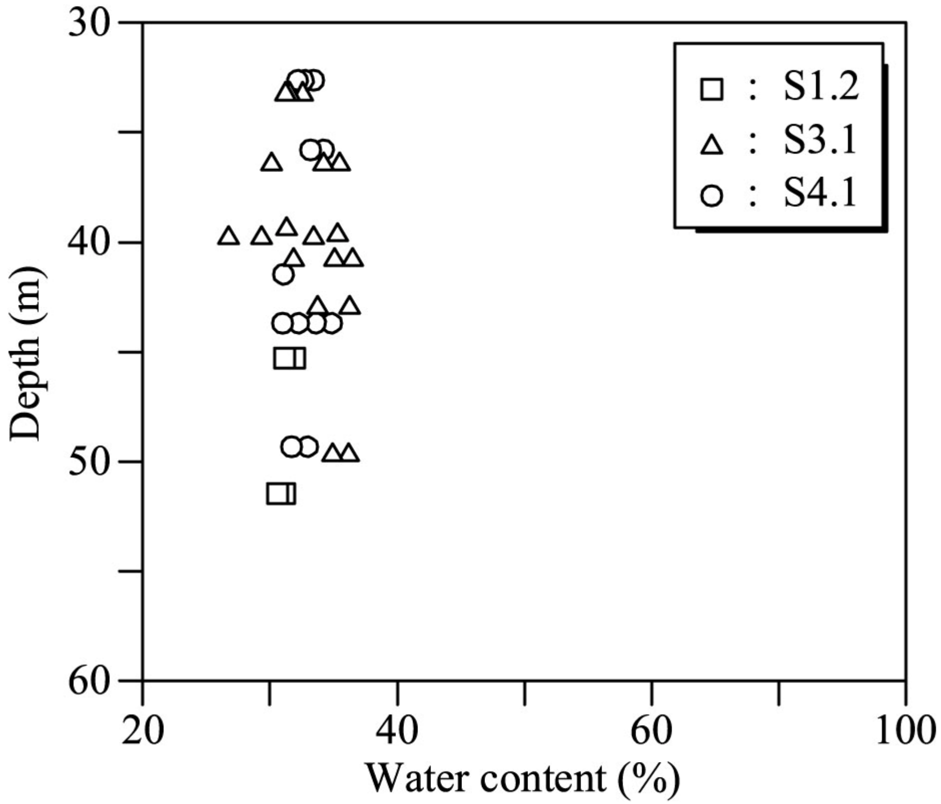
Pyrite tailings

- Percentage of fine particles: 100%
- Non plastic (Classification: ML)
- Void ratio: 0.5 to 0.8
- High « in situ » specific weight: 3.0 à 3.4 g/cc
- Low permeability: (10^{-6} to 10^{-7} cm/s)
- High friction ($\phi' = 37^o - 42^o$)
- Significant cementation : (q_u = 100-200 kPa; $c'_{(b.c.)}$ = 17kPa) (saturated specimens)

4. Geotechnical characteristics of foundation clay

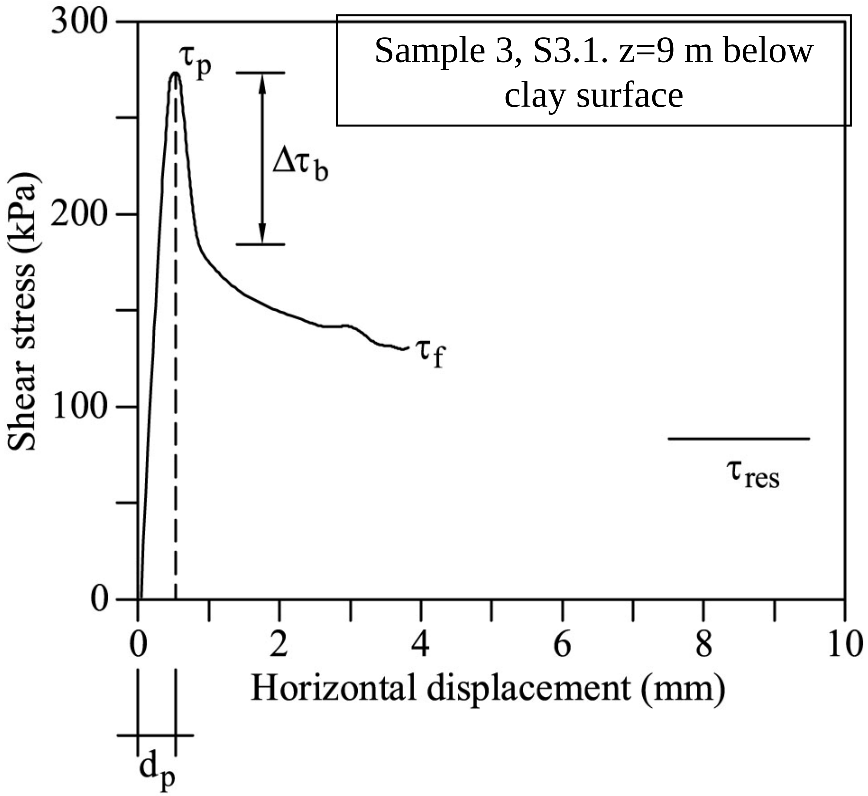
Basic identification

- Slight variation of density, water content and void ratio with depth (w = 30-35%; γ_{nat}= 1.90-1.98 g/cc; e = 0.8 - 1.0)
- Percentage of : Fines: >98% ; Clay: 47-58%
- Plasticity: w_l = 63 - 67% ; IP = 32 – 35%
- Classification: MH or CH



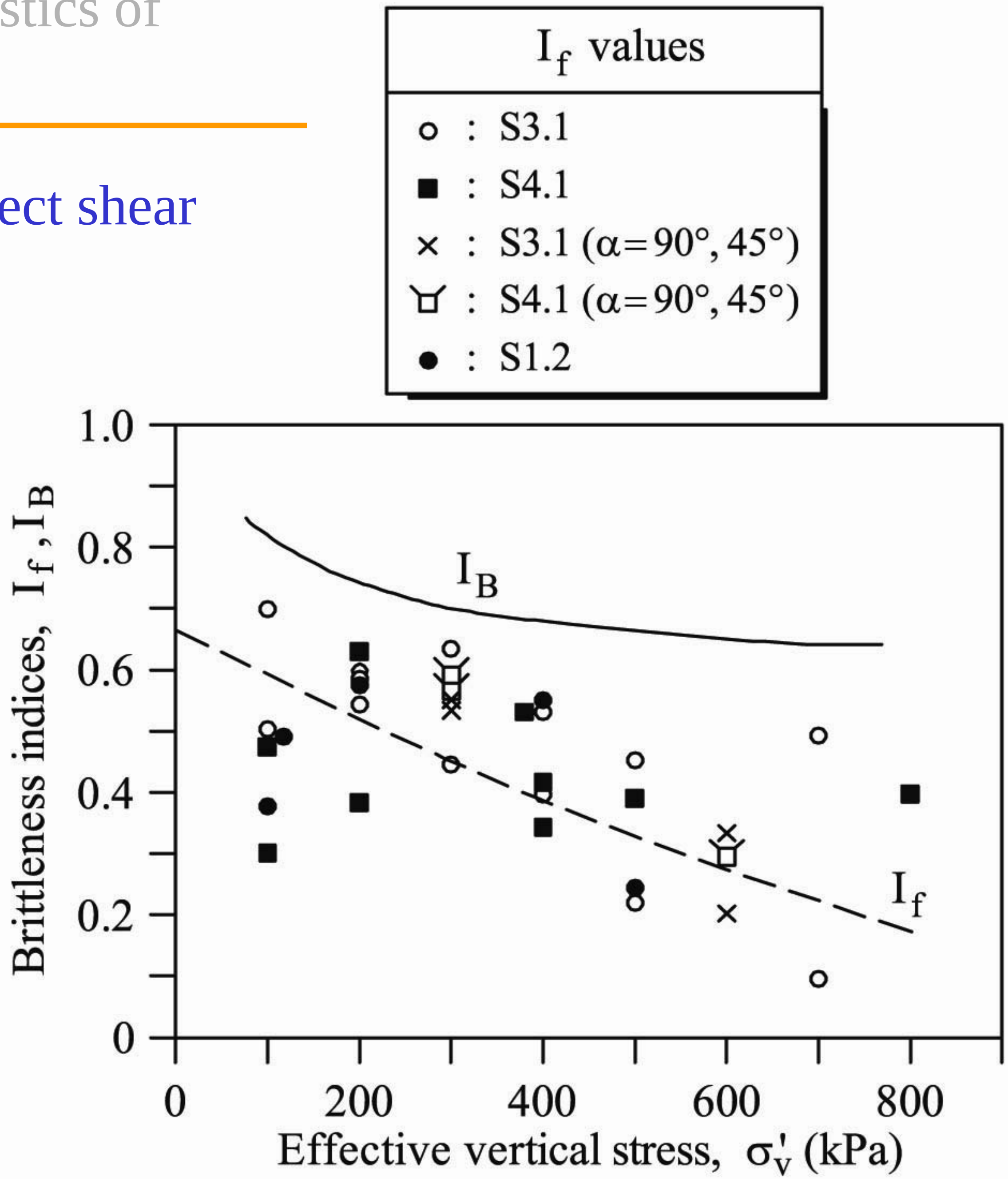
Clay matrix. Drained direct shear

- Quasi-brittle behaviour
  - Displacement (d_p) at the peak strength, d_p<1mm
  - Sudden strength reduction, $\Delta\tau_b$
    - $\Delta\tau_b = 0.55(\tau_p - \tau_f)$
    - $\Delta\tau_b = 0.35(\tau_p - \tau_r)$
  - Brittleness indexes:
    - $I_f = (\tau_p - \tau_f)/\tau_p$; $I_f = 0.40 - 0.65$
    - $I_B = (\tau_p - \tau_{res})/\tau_p$ $I_B = 0.70 - 1.00$



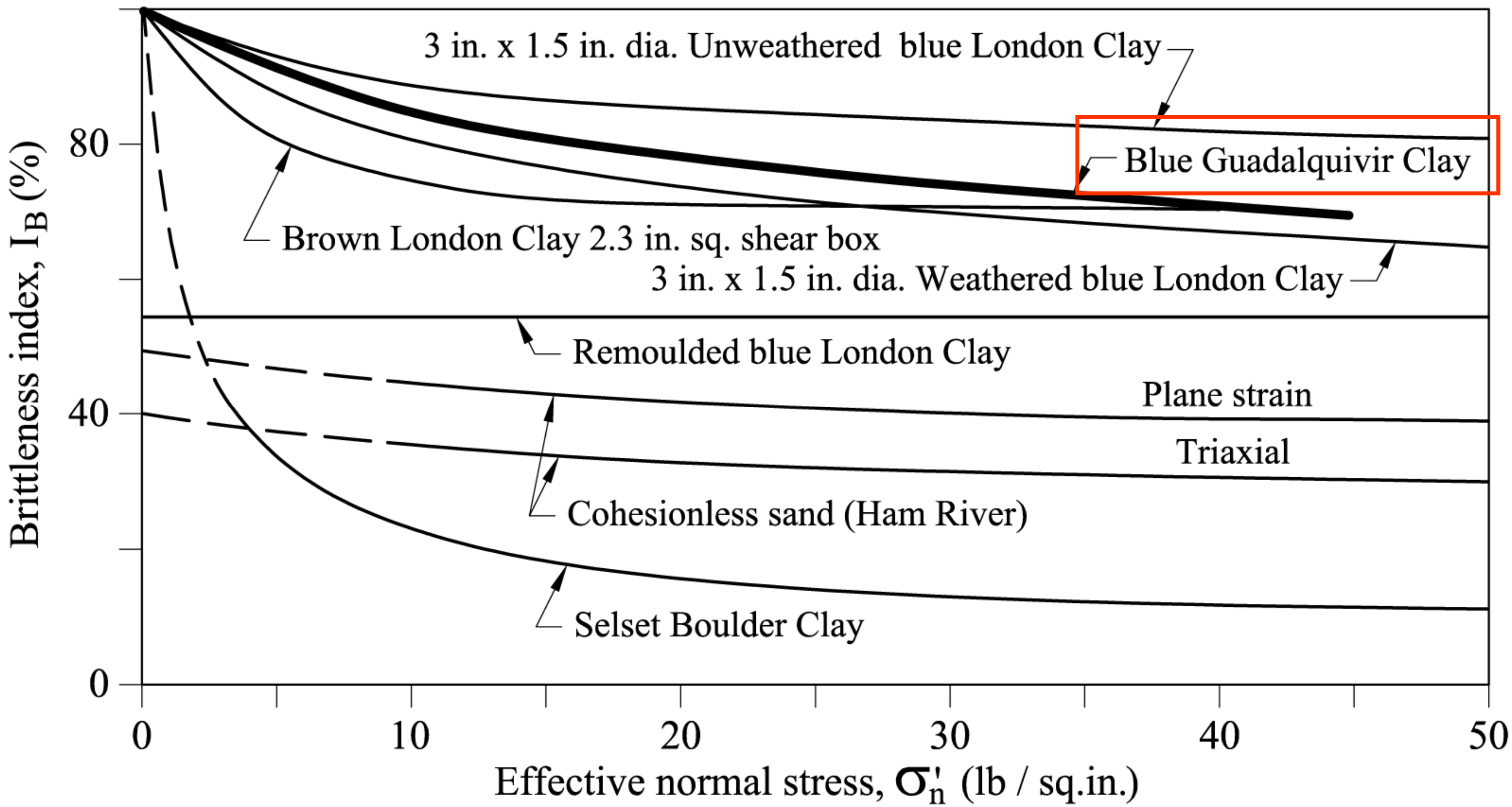
Clay matrix. Drained direct shear

Variation of clay brittleness indexes I_B and I_f with effective stress



Clay matrix. Drained direct shear

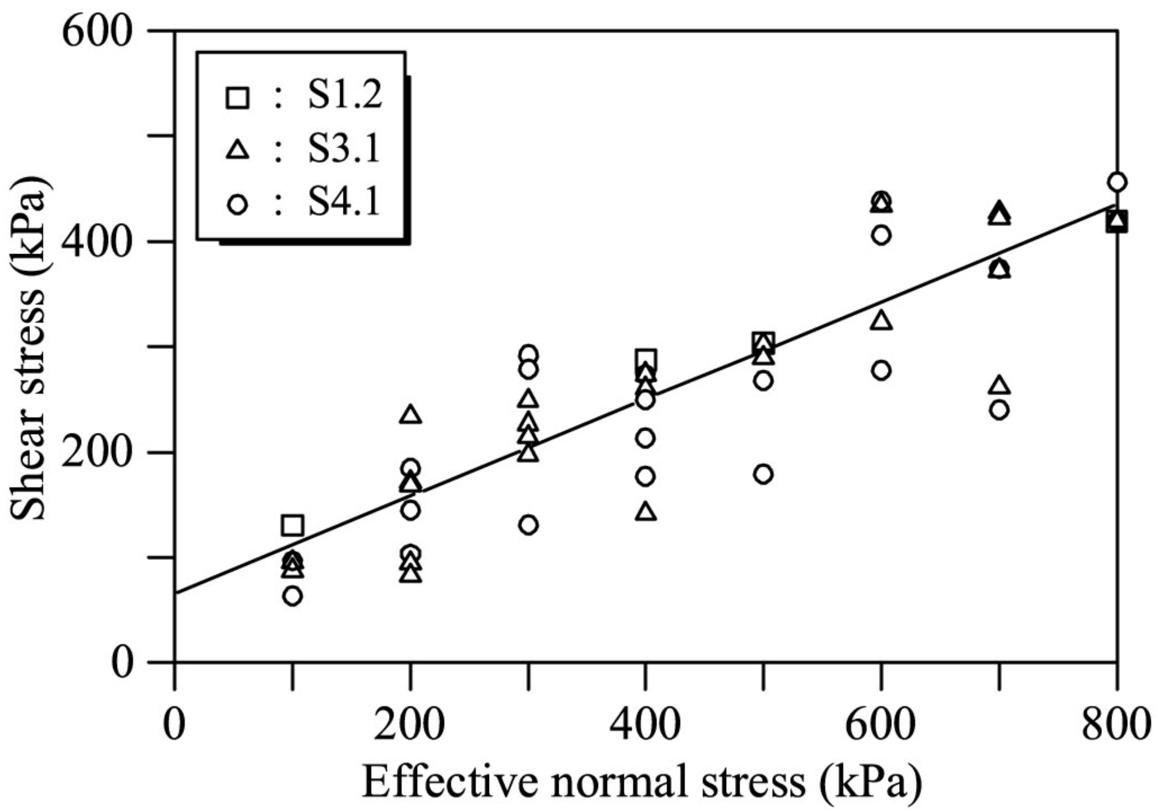
Comparison with other clays



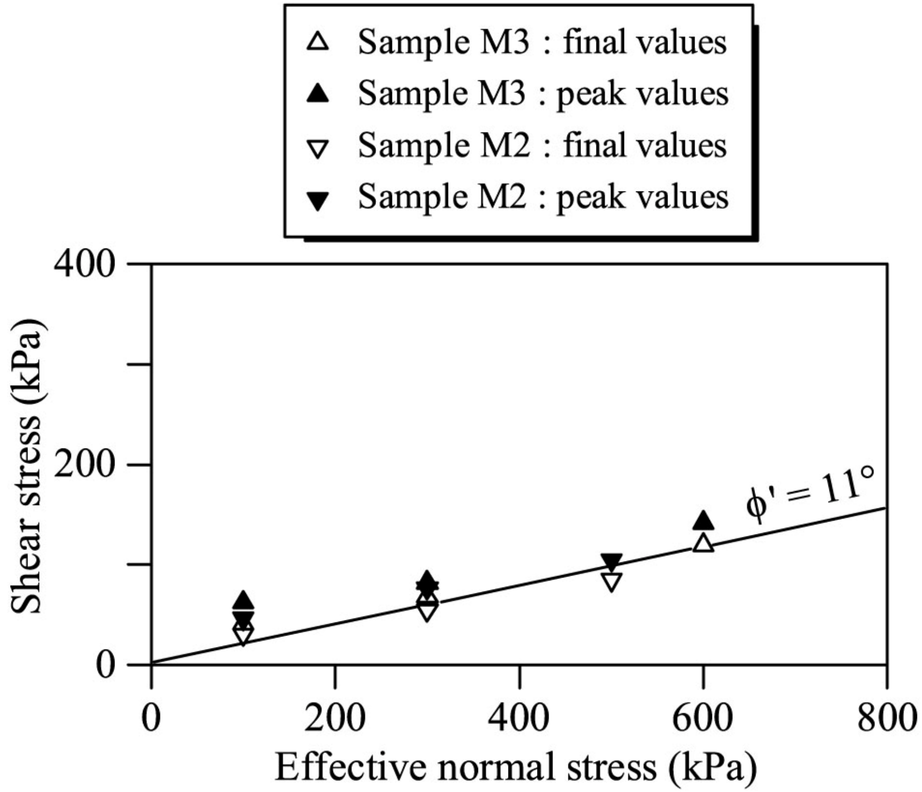
Bishop et al (1971)

Clay matrix. Drained direct shear

- Some dispersion observed in tests
- Average drained strength parameters:
  - Peak: c'=65 kPa; ϕ'=24.1º. End of test: c'=0; ϕ'=15º-23º



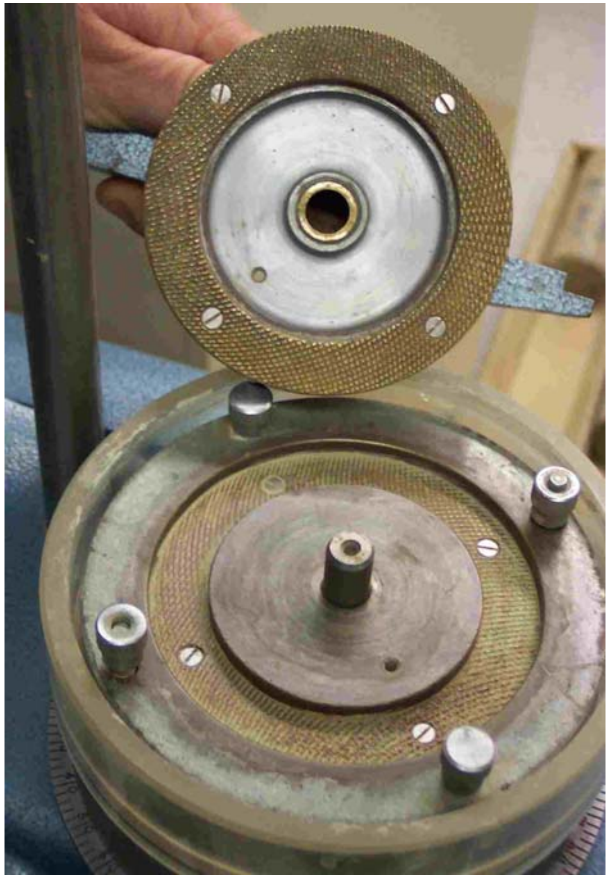
Direct shear tests on natural discontinuities

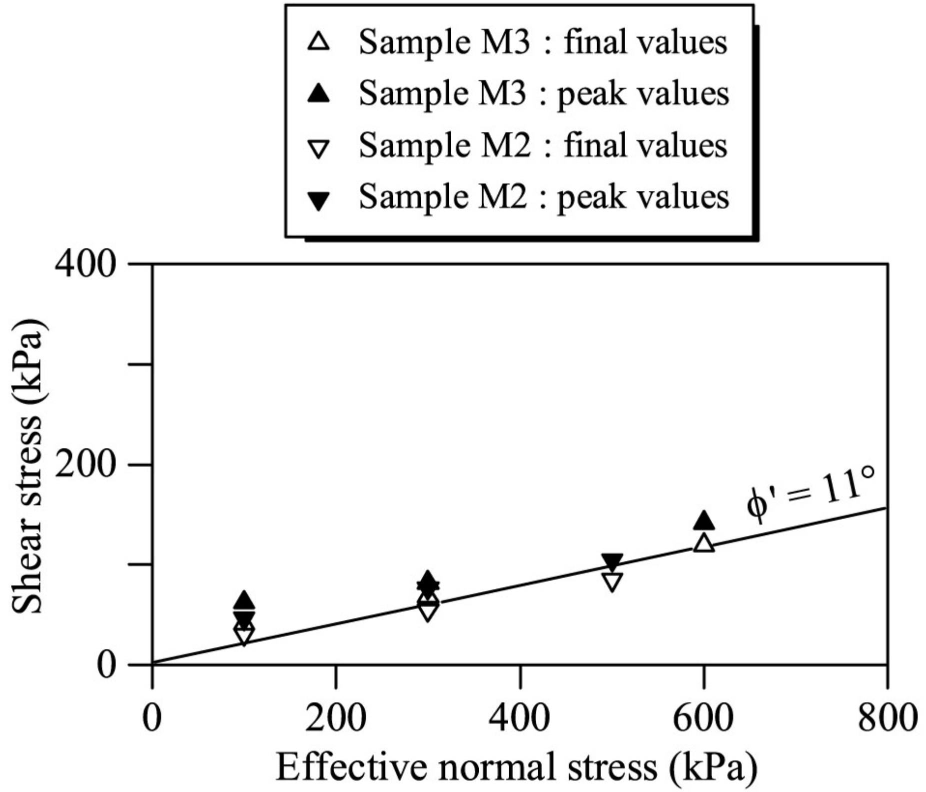


- Strength parameters:

  $c' = 0$; $\phi' = 11^{o}$

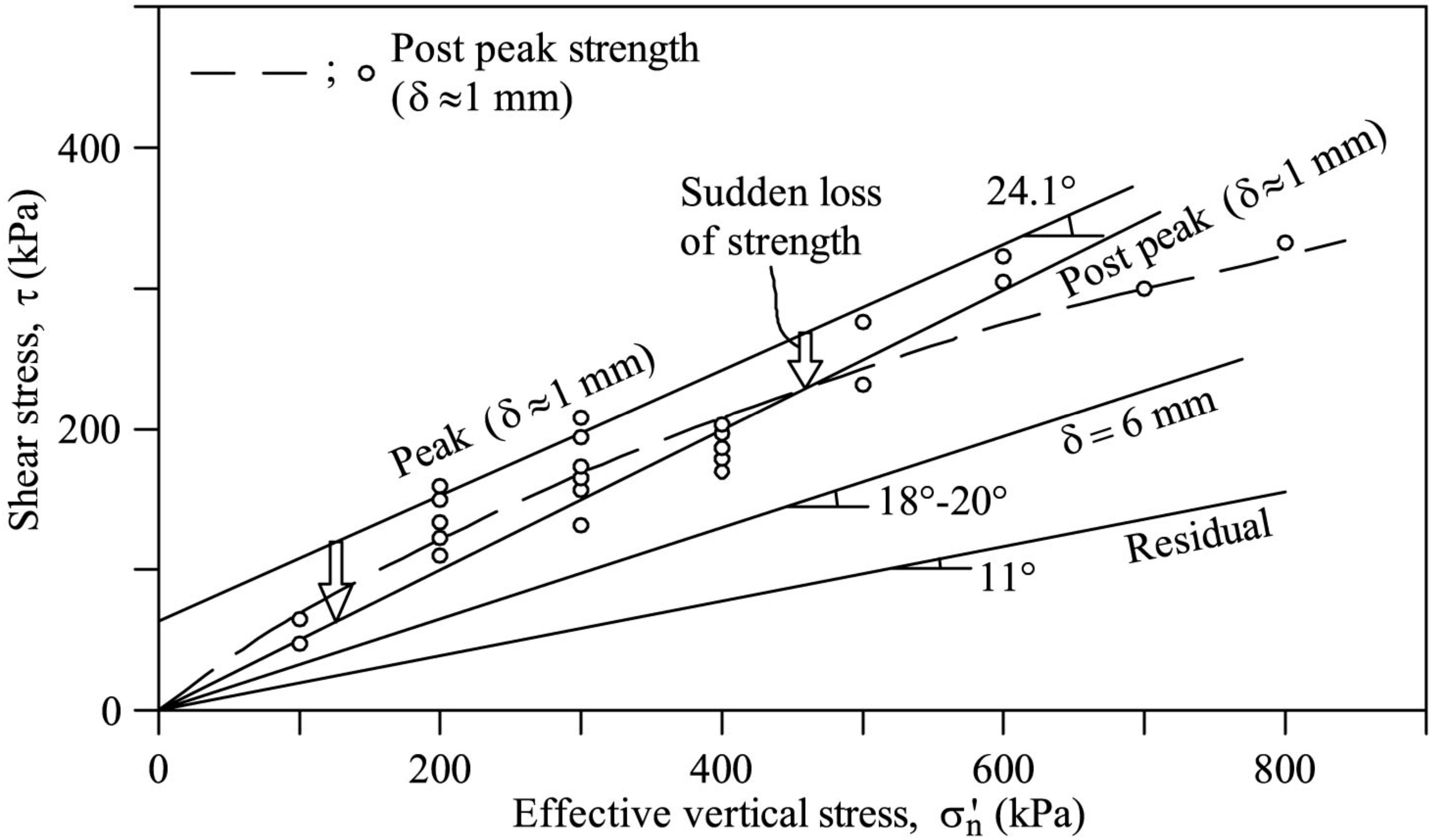
Ring shear tests





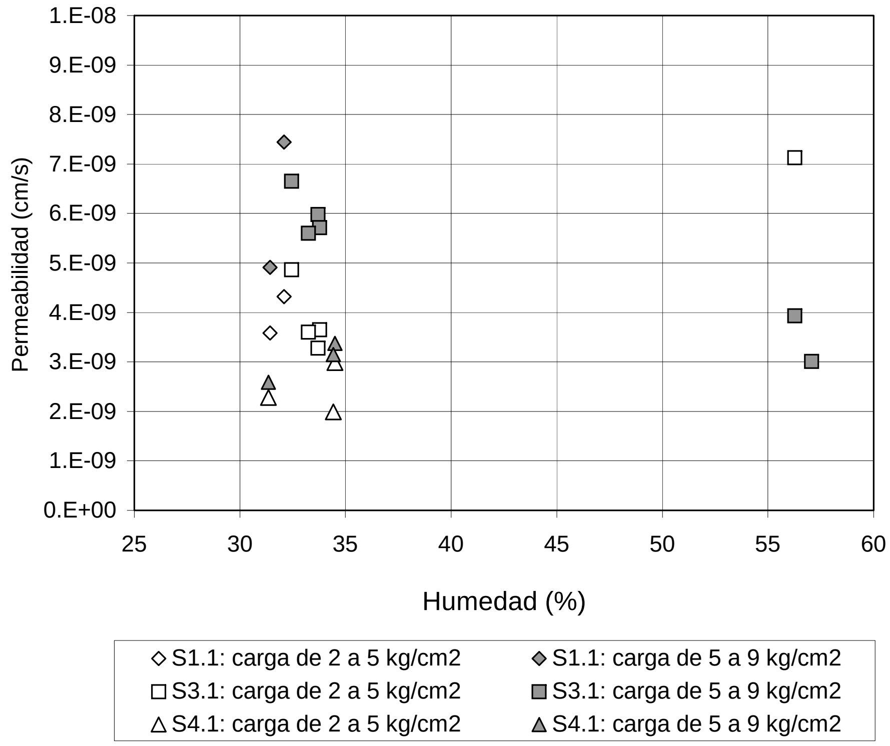
- Remoulded soil (samples recovered in borings; block samples)
- ϕ'_{res}=13º (average)
- Uniformity of ϕ'_{res} in the upper 20 m of clay

Shear strength. Synthesis of results



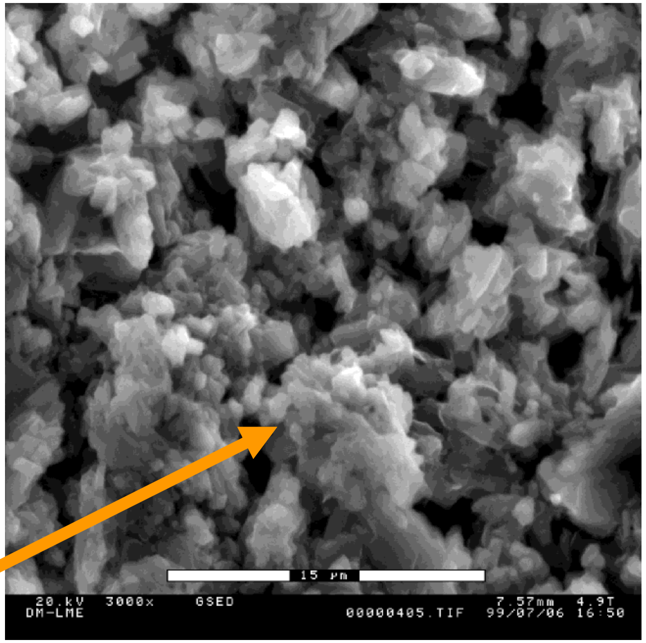
Oedometer tests. Coefficient of consolidation. Permeability

- Tests were interpreted through a model: Primary consolidation + secondary + initial deformation(δ_0, c_v, E_m, C_α) (Back analysis)
- c_v: 0.5 to 1.5×10^{-3} cm^2/s
- K : 2 to 7×10^{-9} cm/s



Mineralogy, structure

- Non clay minerals: Calcite + quartz : 30%
- Clay minerals:
  - Calcic Smectite : 35%
  - Illite + Kaolinite: 35%
- Mineralogical and chemical composition of surfaces of discontinuity is similar to the mass composition. The iron content was different
- Basic structural unit: clay aggregates (Diameter: 5μm)
- Abundant microfossils (Moderate cementation)

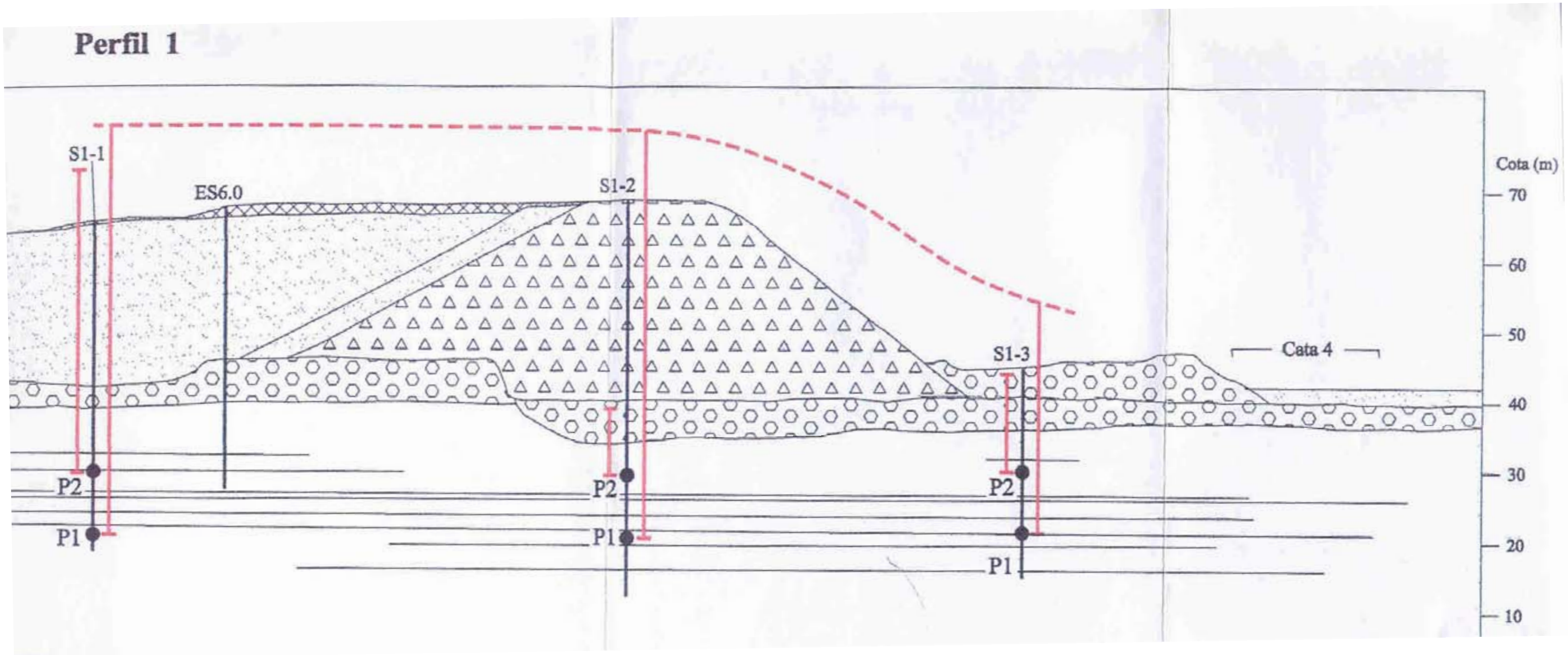


Summary of properties

- Very homogeneous geotechnical unit
- High plasticity (w_l = 63%-67%; IP = 32%-35%; MH-CH)
- Highly brittle (I_f = 0.7 - 0.8)
- Drained peak strength parameters: c' = 65 kPa ; ϕ' = 24.1º
- Residual friction angle: ϕ' = 11º. Same value in natural discontinuities
- Low consolidation coefficient: $c_v = 10^{-3}$ cm^2/s
- Low permeability: K = 2 to 7 x 10^{-9} cm/s

5. Water pressures and stresses in the foundation

- Objectives:
  - To develop a simple calculation method, based on analytical solutions, for a first approximation to the problem.
  - To interpret pore water pressure measurements after failure and to derive pwp likely acting on the sliding plane at the time of the failure.



Measured pwp profile after failure (in October 1999, after stabilisation)

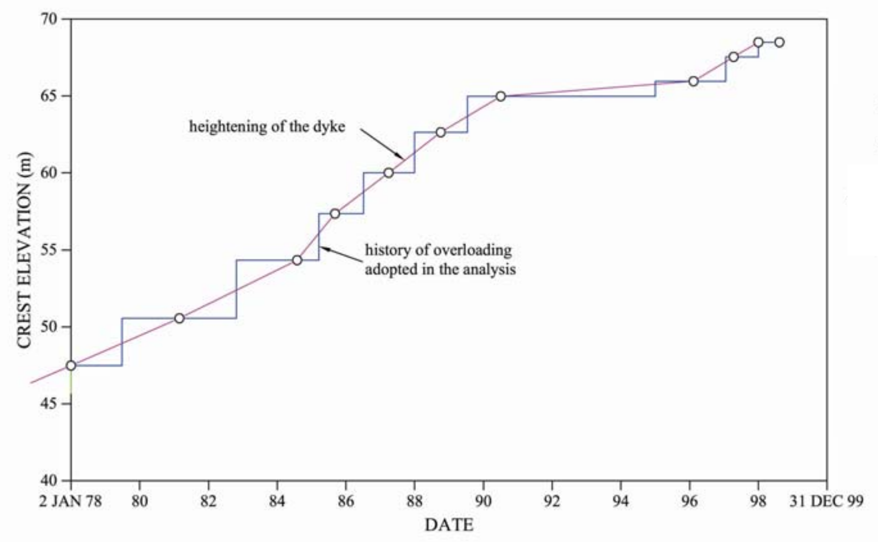
Calculation model

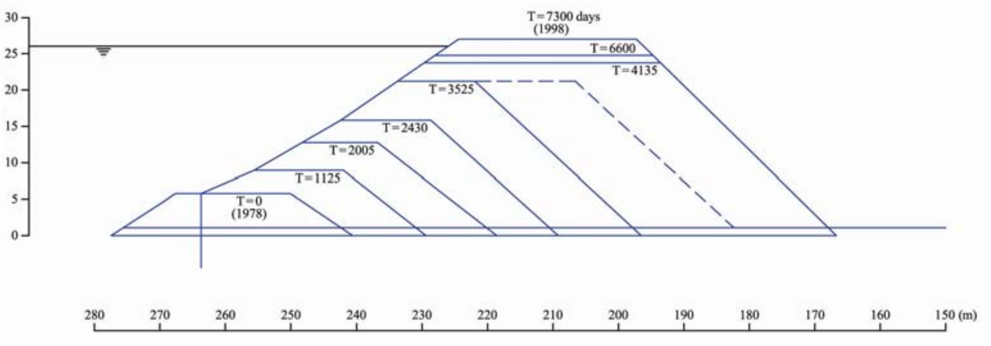
- Total stresses: according to analytical solutions (embankment loading). Superposition of construction phases

- Initial increment of pore pressure = Increment of mean stress

- Dissipation of pwp according to one dimensional theory of consolidation. Drained boundaries at the surface (upper alluvium) and at depth (lower pervious aquifer)

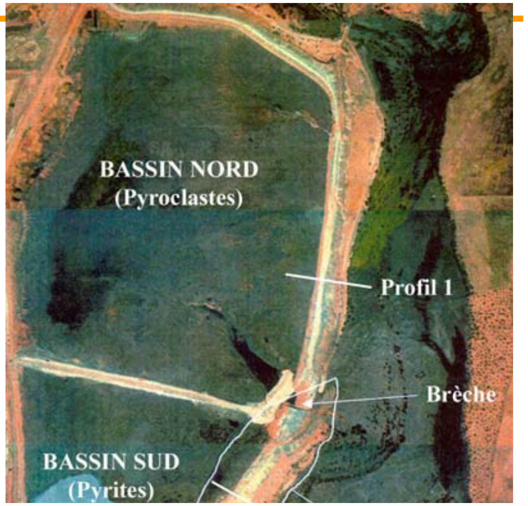
- Superposition of pressures for each one of the construction phases

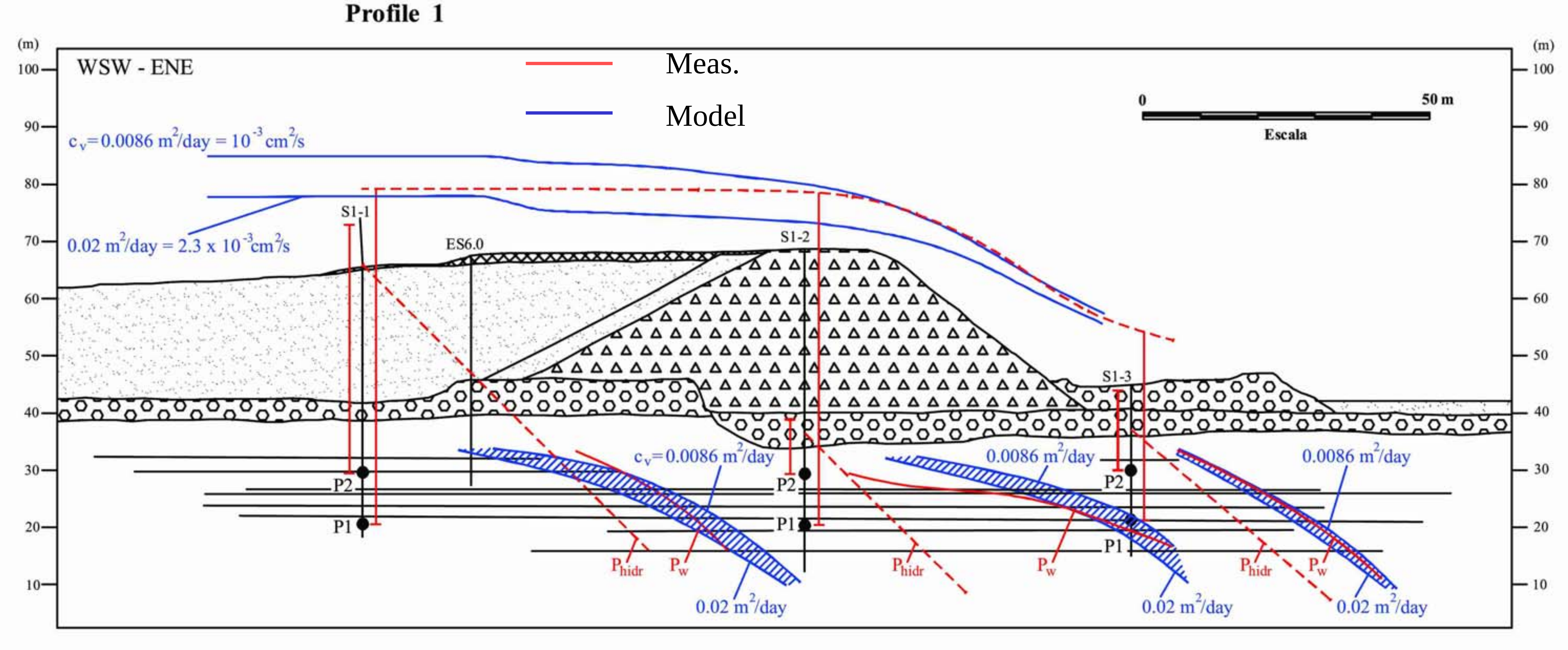




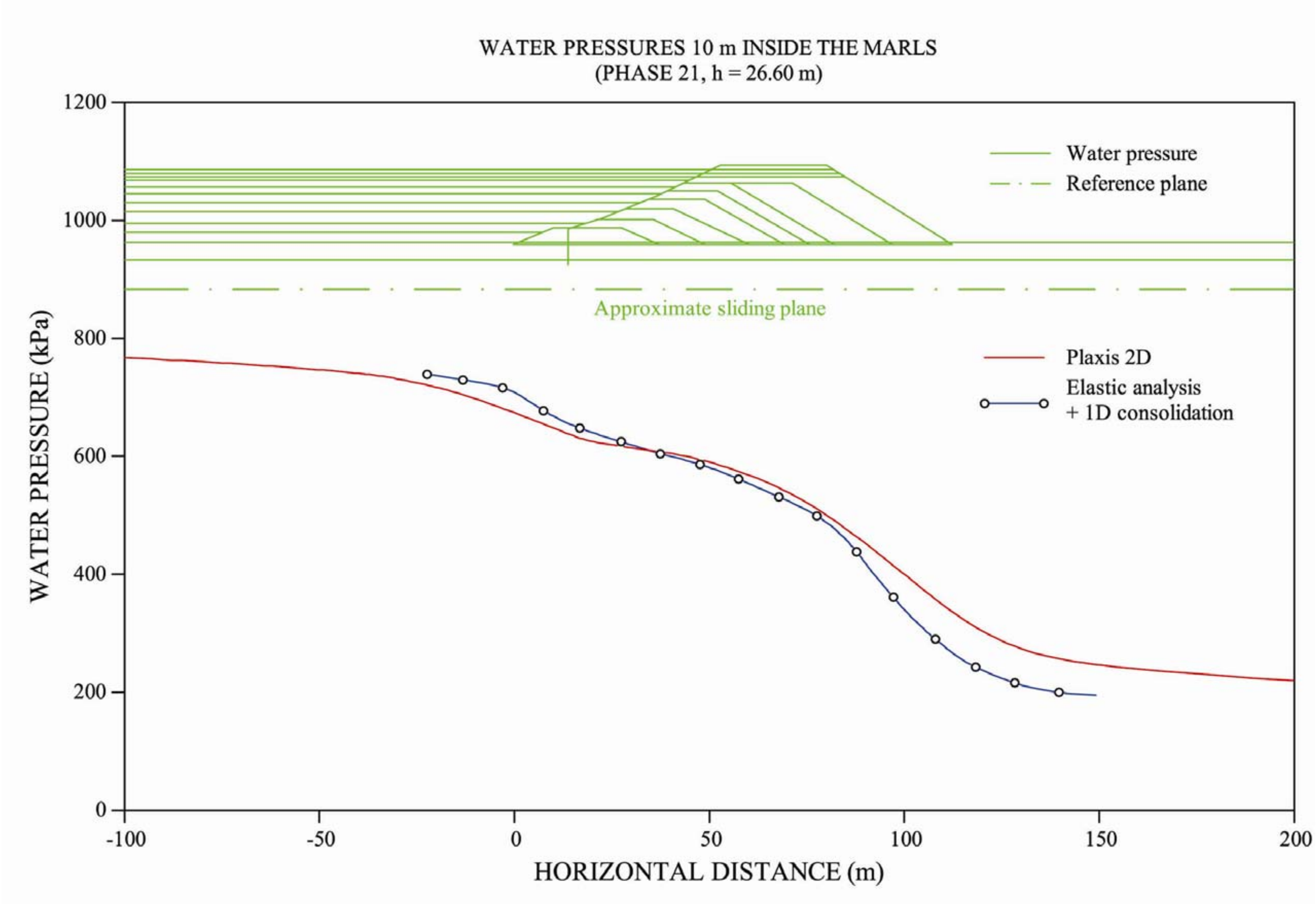
Water pressures in section section 1 (pyroclaste basin)

Comparison between calculations and measurements



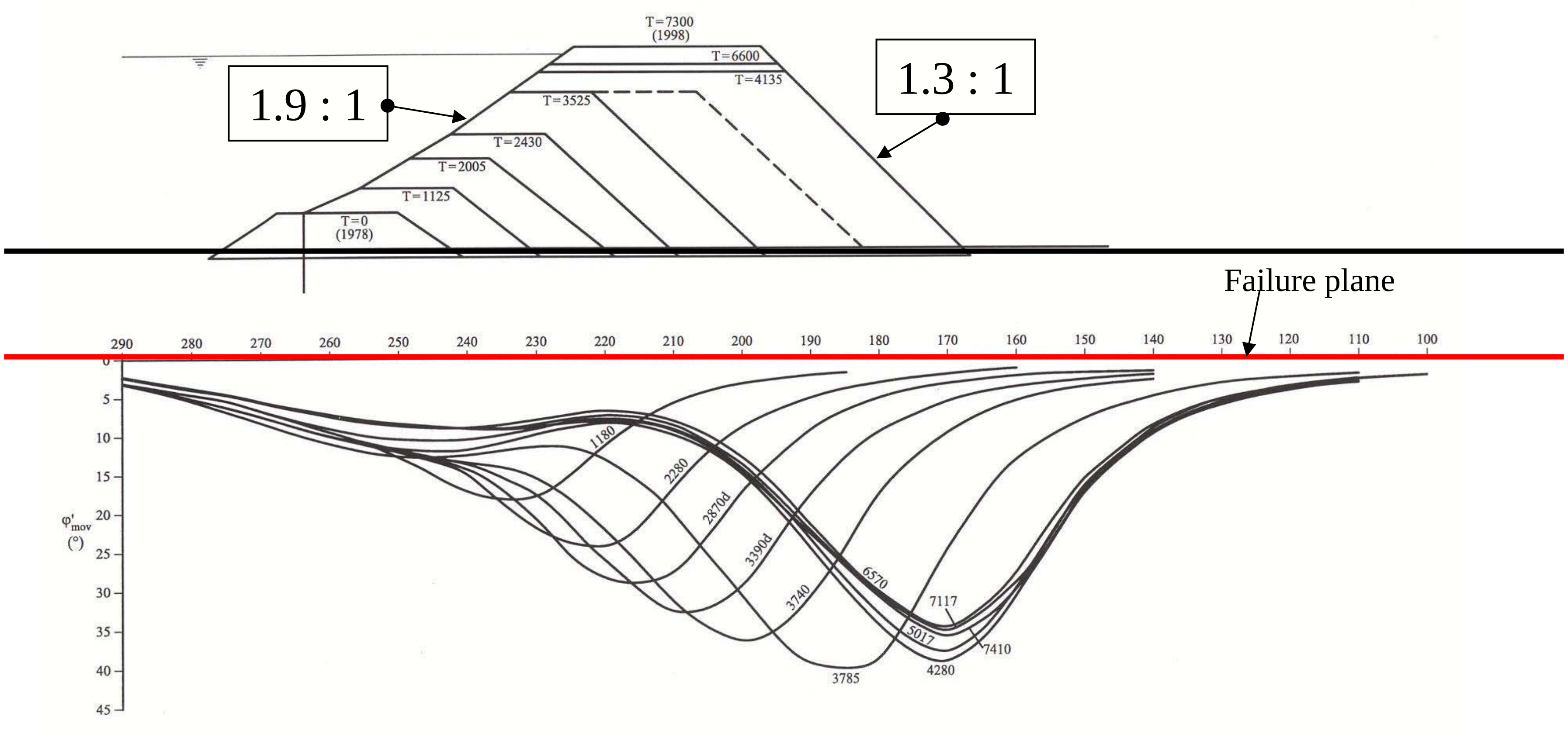


Calculated water pressures in section 3 (pyrites basin) on the sliding plane at the time of the failure

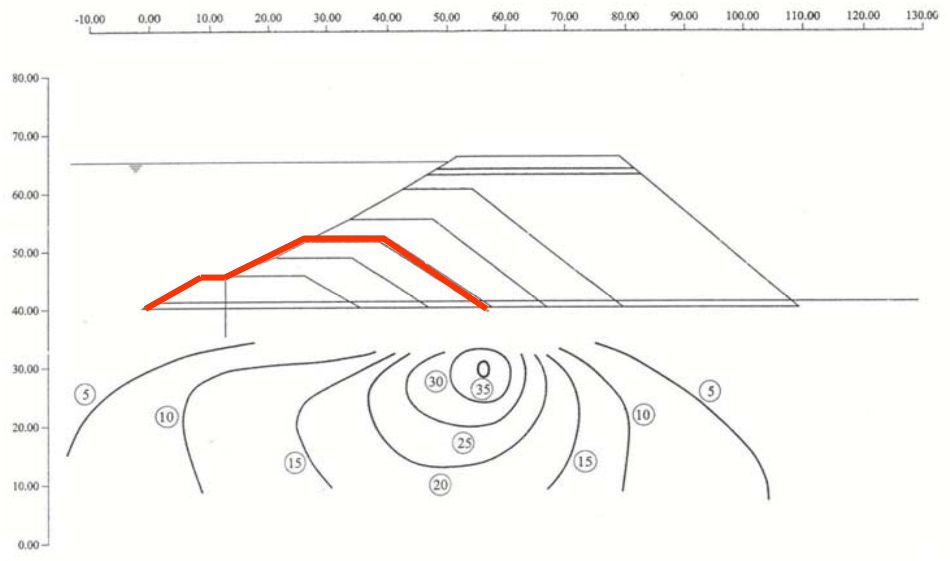


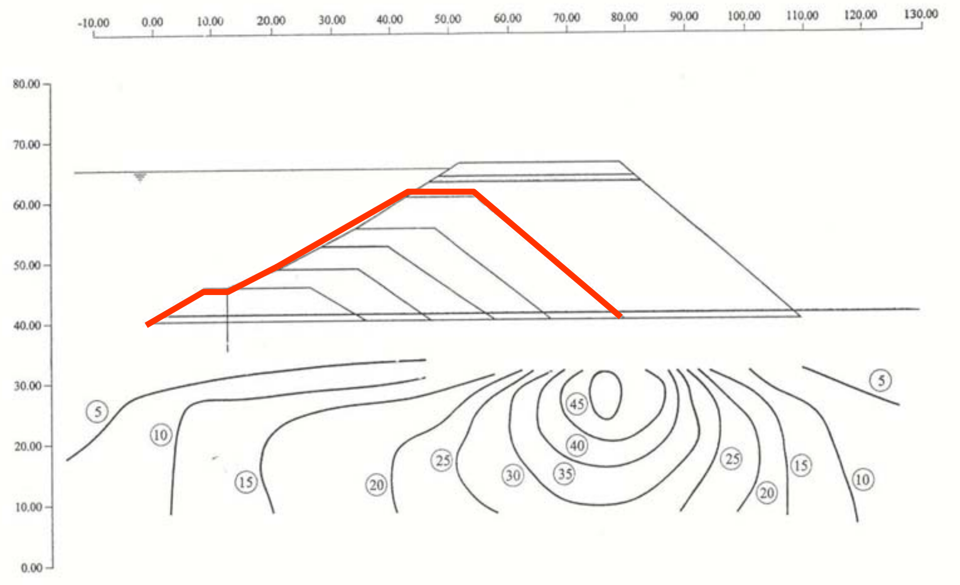
Stresses under the dyke (pyrites basin)

- Local friction angle mobilised on the failure plane: $\varphi'_{mov} = \text{arc tan}\left(\dfrac{\tau}{\sigma'_n}\right)$
- Progressive downstream increase of φ'_{mov}
- Progressive failure phenomena could start when the dyke reached a height of 18 m



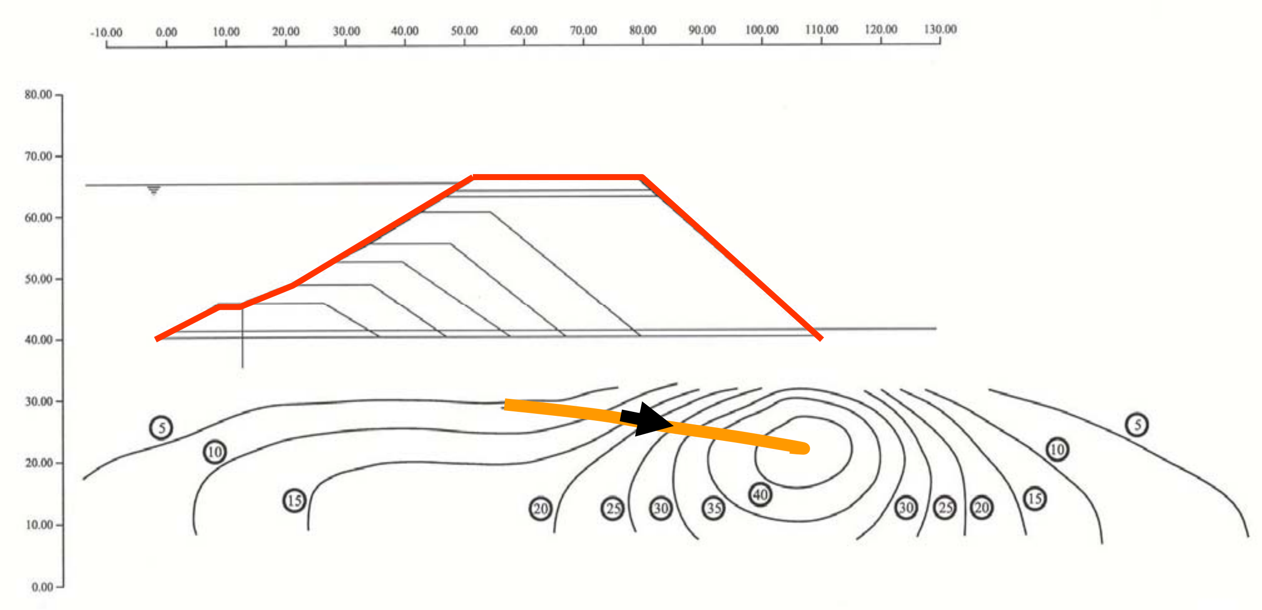
Distribution of mobilised stress ratio in the foundation





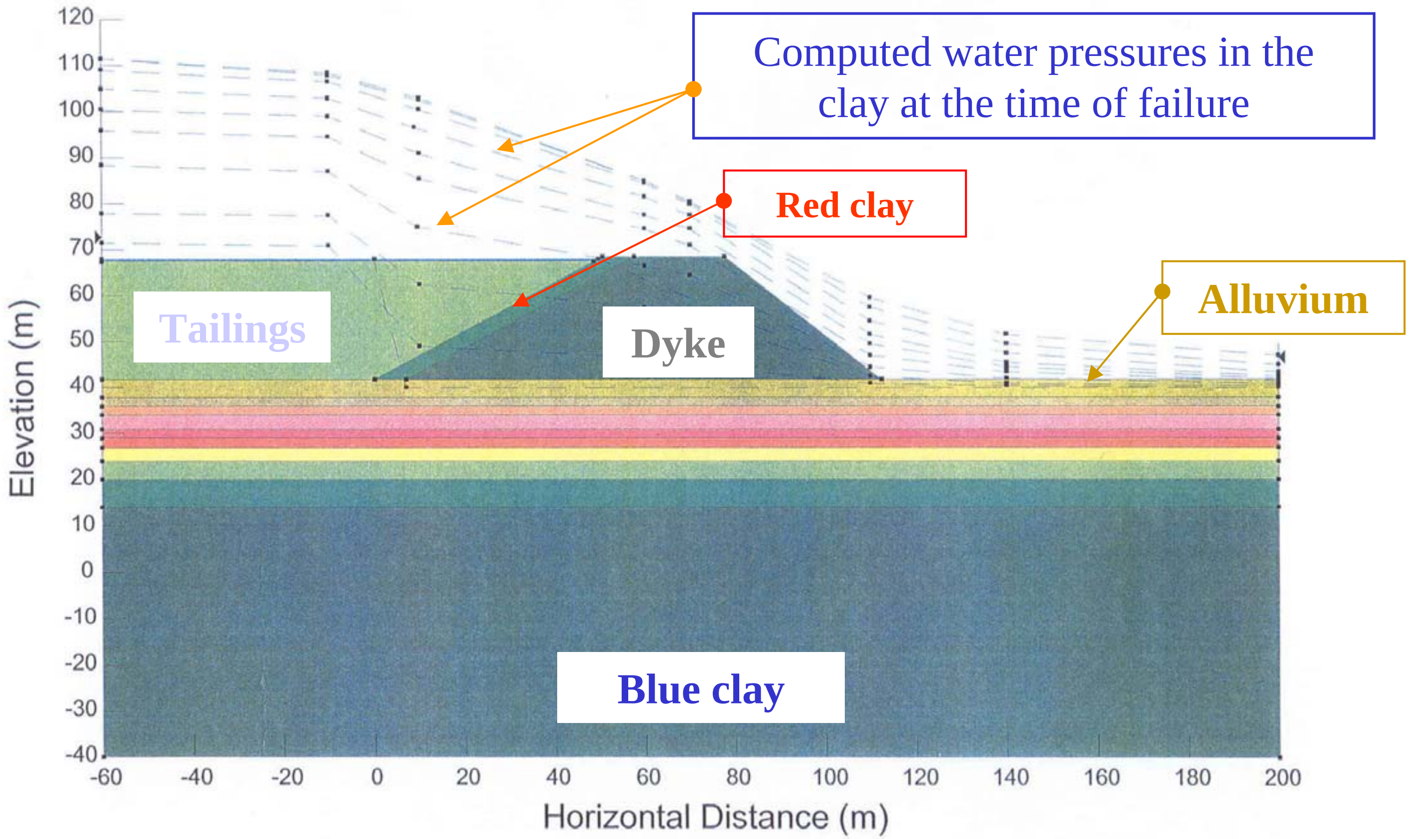
Evolution of mobilised stress ratio in the foundation

- The value of τ/σ'_n reaches a maximum value at a certain depth under the downstream foot of the dyke
- This depth changes slowly, as the size of the dyke increases
- Thee curve joining the position of maxima marks probably the position of the sliding surface and the evolution of progressive failure



6. Failure analysis. Limit equilibrium

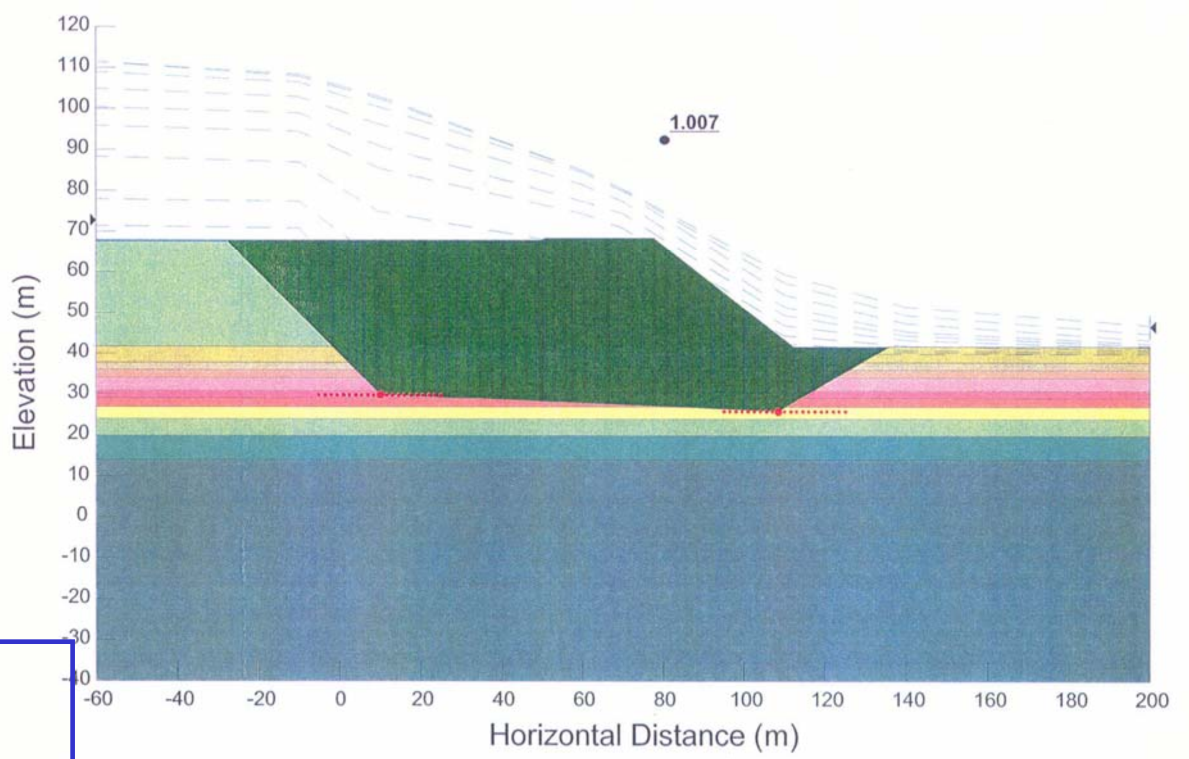
Reference cross section for calculations



Material	Density (kN/m3)	Friction (º)	Cohesion (kPa)
Tailings	31	37	0
Red clay	21	27	0
Dyke	20	40	0
Alluvium	20	35	0
Blue clay	19.5	Variable	Variable

Back analysis

- F.S.= 1 if (ϕ'=17º, c'=0) for a critical surface close to the real one



Stability at intermediate construction phases

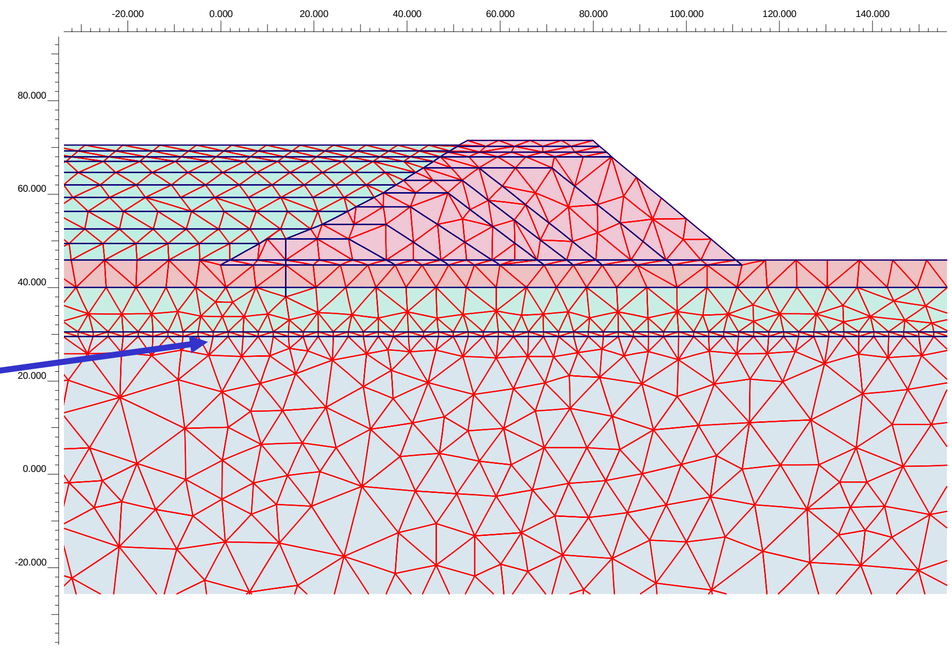
Dyke height (m)	Consolidation time (days)	F.S. if ϕ' = 17º	ϕ' if F.S.=1 (c'=0)
12.5	2220	1.06	16º
21.15	3740	0.89	19.5º
23.5	4280	0.98	17.5º
27	7300	1.04	16º

Indication of progressive failure

7. Failure analysis. Finite elements

- Objectives:
  - Understanding processes leading to failure
  - Integration of construction phases, generation of p_w, consolidation, deformation and failure

- **Models developed:**
  - **M1:**
    - Homogeneous clay
    - Peak parameters
    - Mohr- Coulomb. Perfect plasticity
  - **M2:**
    - Failure band
    - Reduced strength parameters in the band



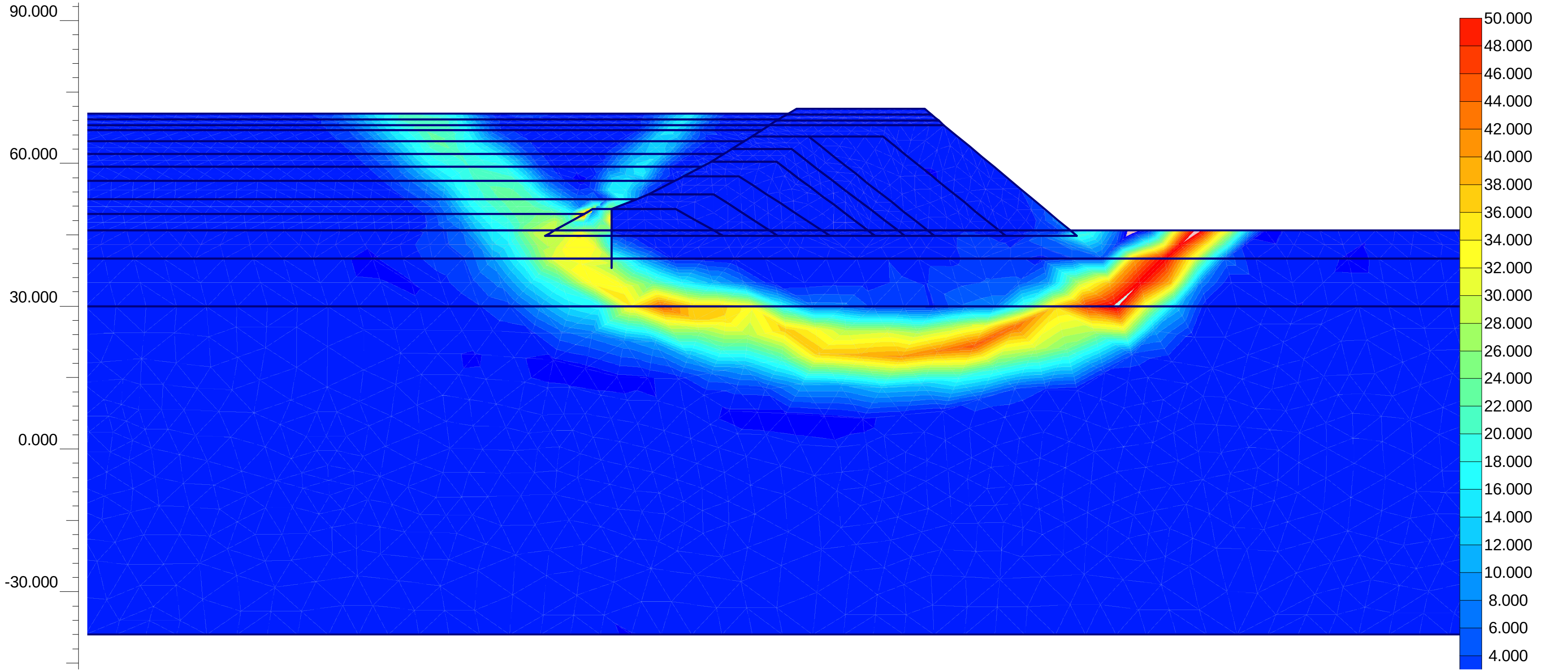
Parameters

Symbol	Alluvium	**Blue clay**	Embankment	Tailings	Units
E	20000	**40000**	40000	3000	kPa
ν	0.3	**0.3**	0.3	0.3	
c'	1	**variable**	-	1	kPa
ϕ'	35	**variable**	-	37	º
Ψ	0	**0**	-	0	º
K	$1.5x10^{-3}$	**$1.5x10^{-6}$**	$1.5x10^{-3}$	$1.5x10^{-3}$	m/day

- Calculation:
    - 11 construction phases
    - For each phase: undrained calculation and consolidation (21 phases)

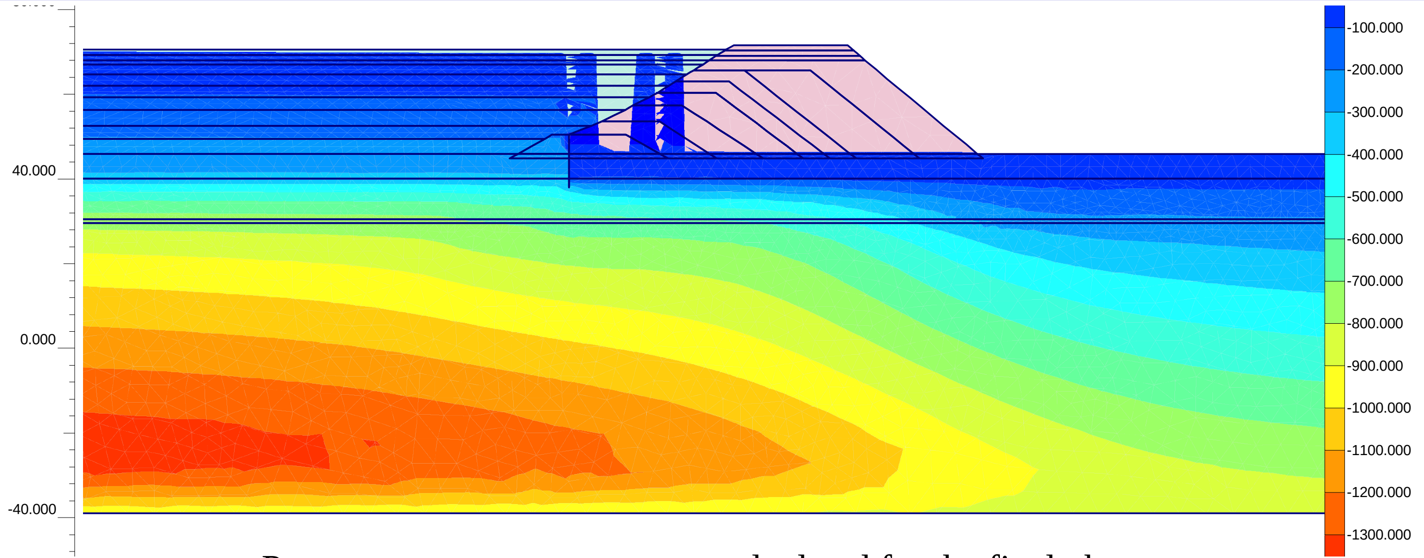
Analysis with homogeneous clay (M1)

- If c'= 65 kPa et ϕ'= 24º (peak) there are no plastic points: progressive failure does not develop
- Deep failure surface mechanism (similar to the worst mechanism found through limit equilibrium)



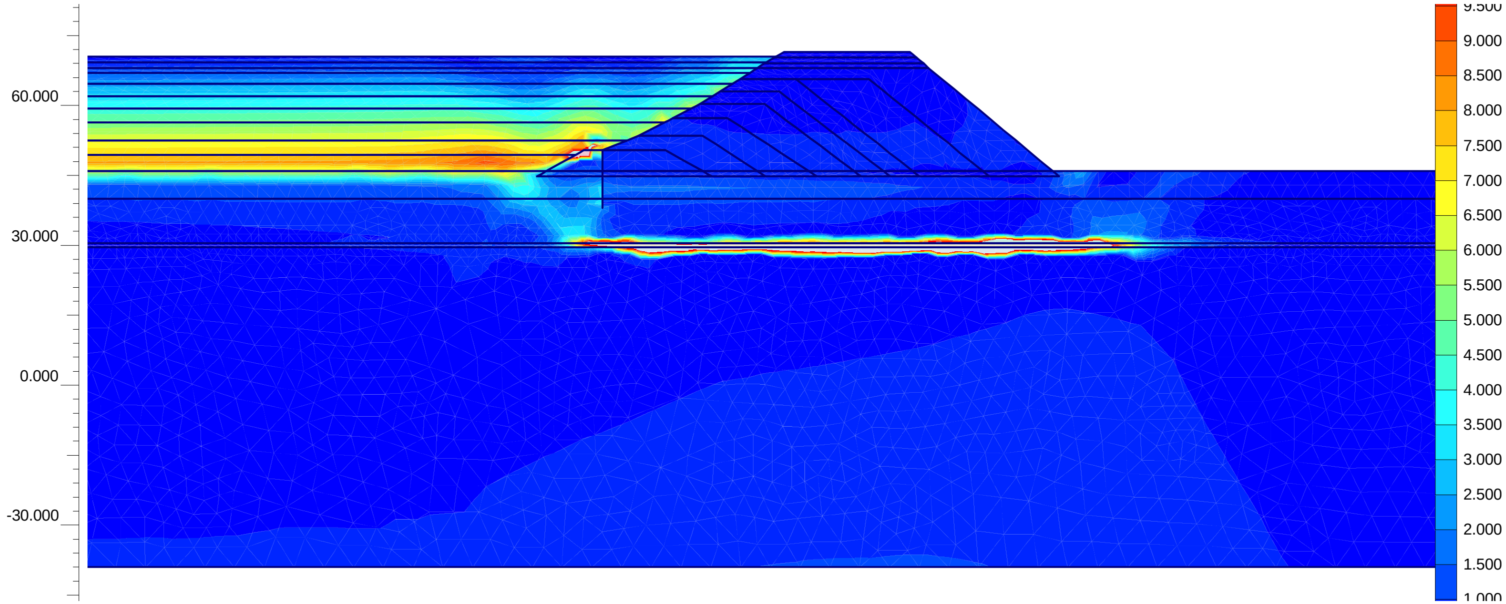
Analysis including a planar failure surface (M2)

- If the chosen strength parameters reproduce the final failure, it is found that the dyke was unstable in Phase 11 (1988)
- Chosen parameters:
  - Clay in failure band and upper zone (1978-1988): c' = 15kPa, φ' = 21.5º
  - Clay in failure band and upper zone (1988-1998): c' = 1kPa, φ' = 21.5º
  - Clay below the failure band : c' = 15kPa, φ' = 21.5º

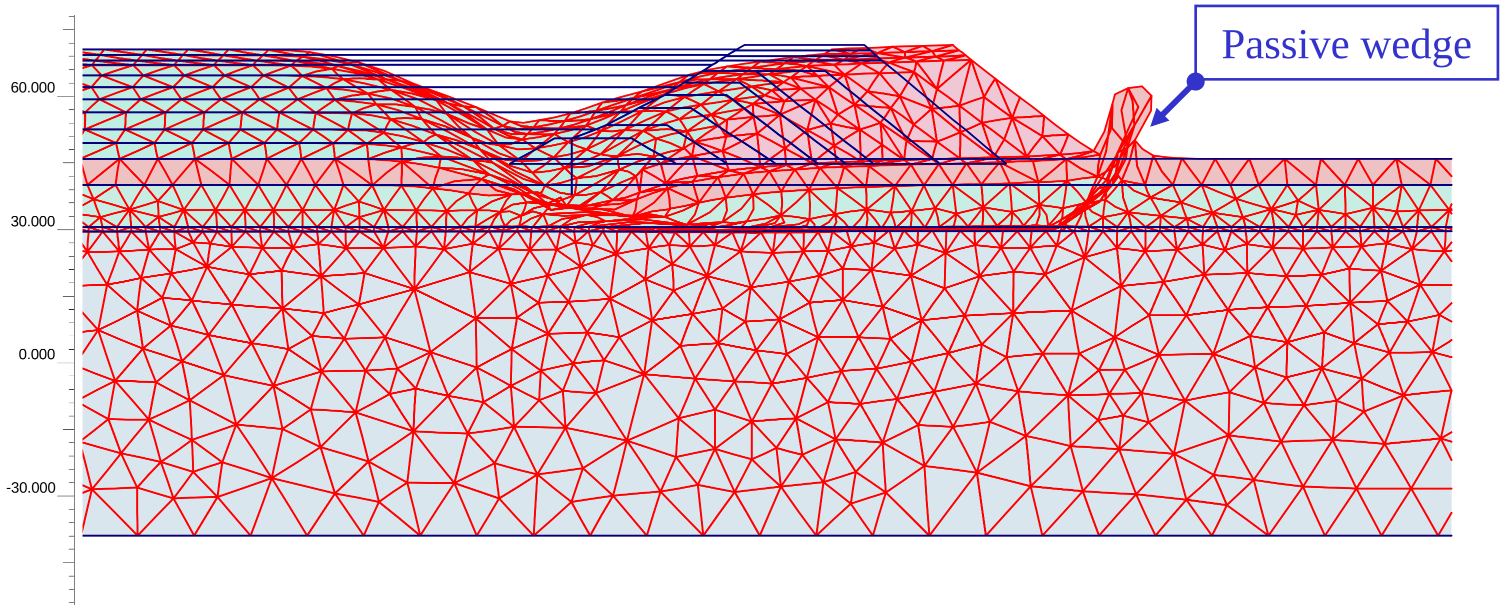


Pore water pressures contours calculated for the final phase

Analysis including a planar failure surface (M2)



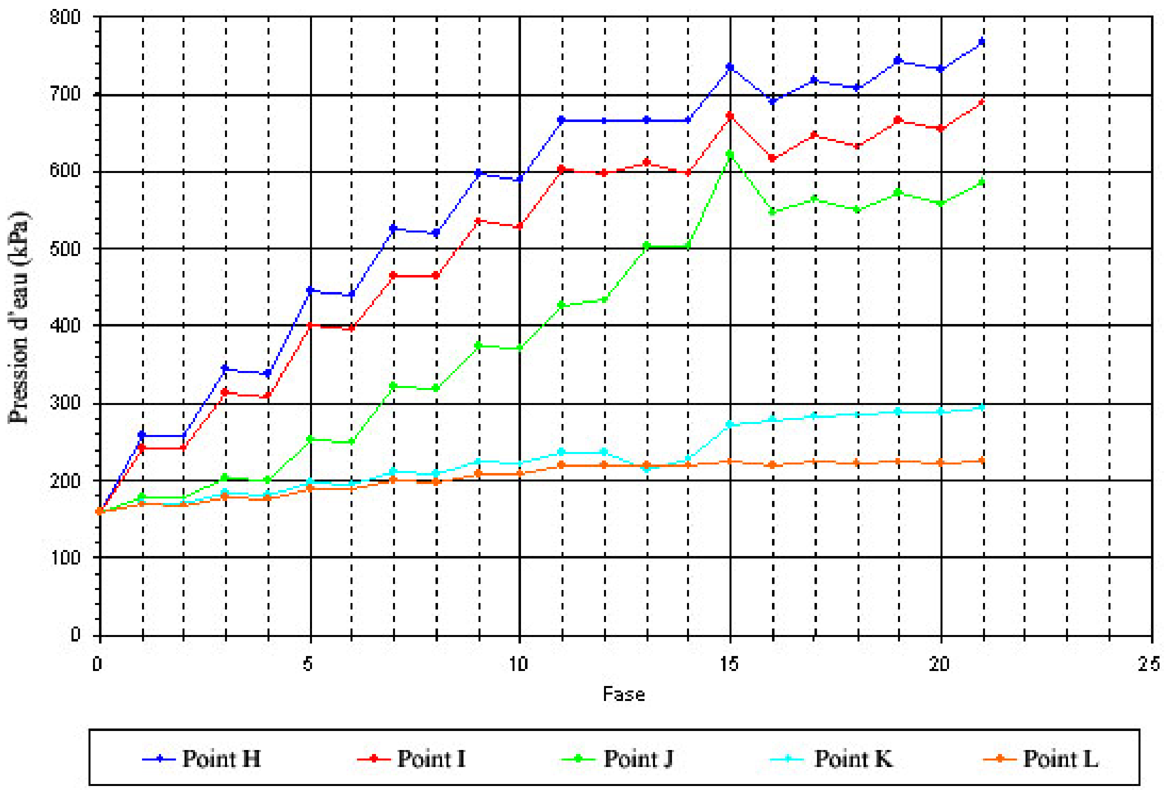
Contours of equal accumulated shear deformation in final phase



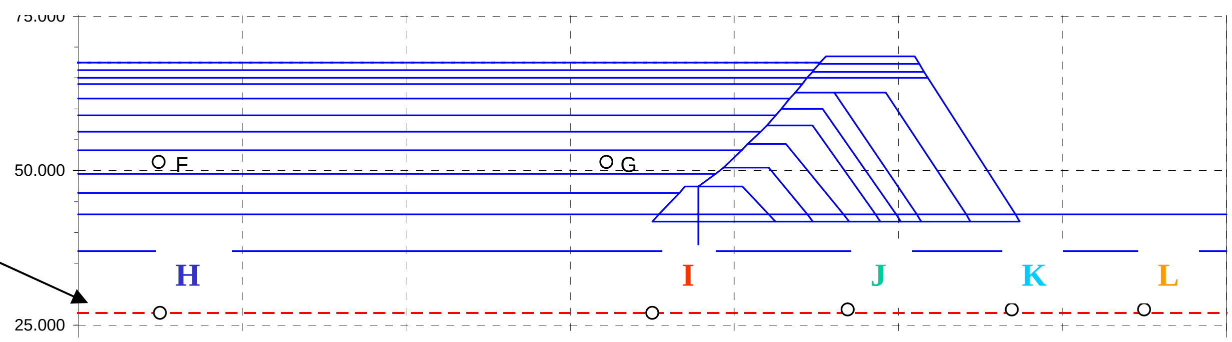
Mesh deformation close to failure

Development of pore water pressures

- Very limited dissipation of pwp



Position of points in failure surface



First failures in over consolidated clays and shales *(Skempton, 1970, 1977; Chandler, 1984..)*

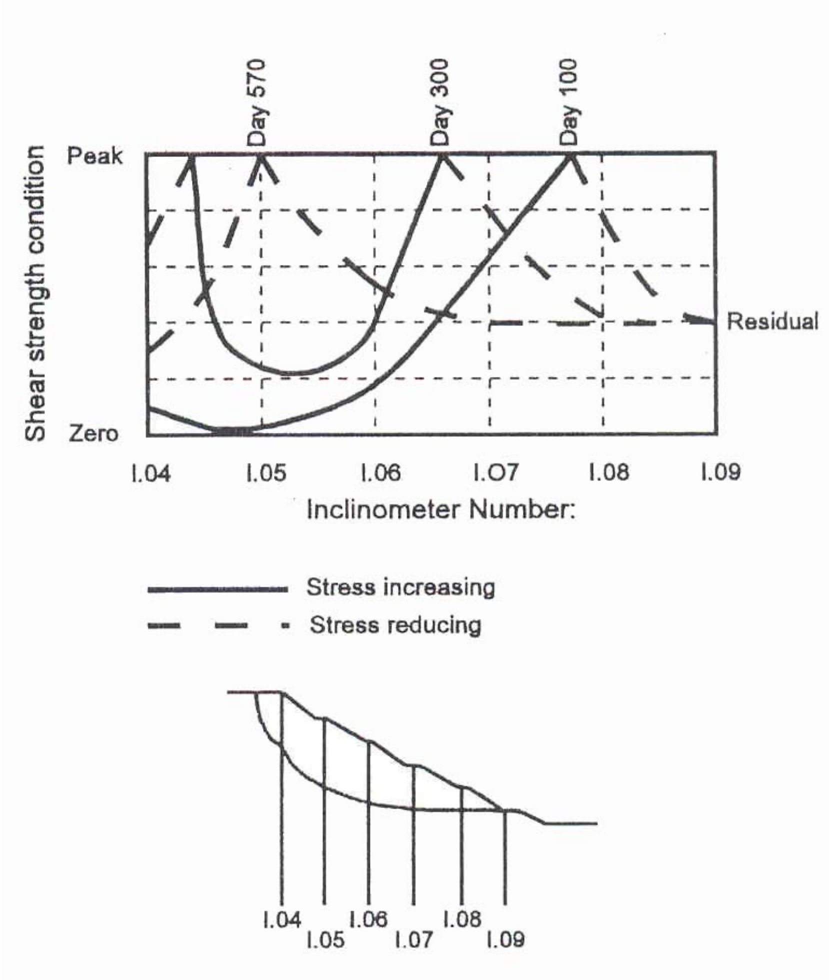
- Geological processes (tectonics, erosion, unloading, swelling…) lead to clay softening : water content increases, strength decreases.
- It was proposed that the strength for a full clay remoulding (« fully softened strength ») is a lower limit of the available strength

- "Fully softened strength" (FSS): Peak drained resistance of a reconstituted, normally consolidated clay.
  - "FSS" implies the loss of cohesion but maintains a random orientation of clay microstructure: The friction angle corresponding to FSS is similar to the peak friction angle of the clay mass

Additional factors leading to a loss of strength. Over consolidated clays and clay shales

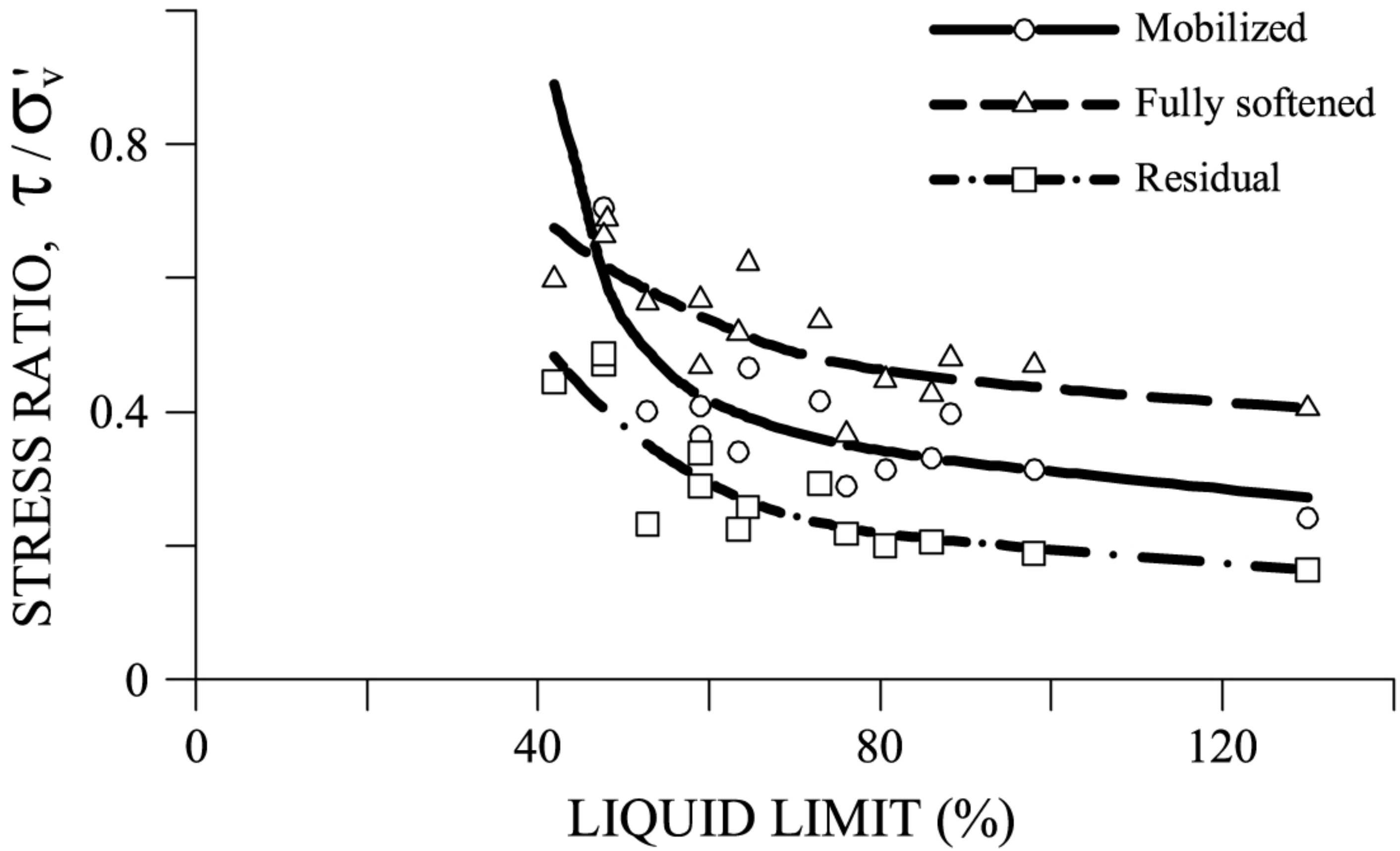
- Critical surfaces (fissures, discontinuity planes, sedimentation planes) are capable of accumulating relative shear displacements during geological processes
- Clay brittleness leads to **progressive failure** mechanisms during excavation or loading processes

Selborne slide



Estimated profiles of soil strength on the sliding surface. *(Cooper, 1996)*

Back analyses of mobilized strength in first time failures in hard and fissured clays *(Stark and Eid, 1994)*



(Note: Only Carsington and North Ridge dams are included in the cases analyzed)

- **Conclusion**: Mobilized strength is intermediate between FSS and the residual strength

Finite element analysis

«Residual Factor», R, defined by Skempton (1964) as :

$$R = (\overline{\tau}_P - \overline{\tau}) / (\overline{\tau}_P - \overline{\tau}_R)$$

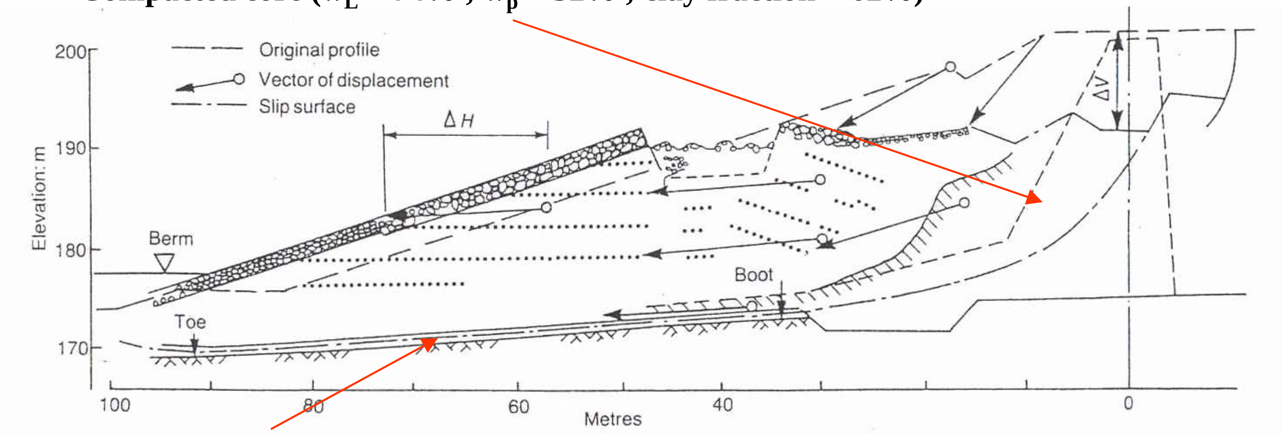
$\overline{\tau}_P$ $\overline{\tau}_R$ $\overline{\tau}$: Averaged values on the failure surface

- Carsington dam *(Potts et al, 1990)*

  R=0.42 (Core)

  R=0.52 (Yellow foundation clay)

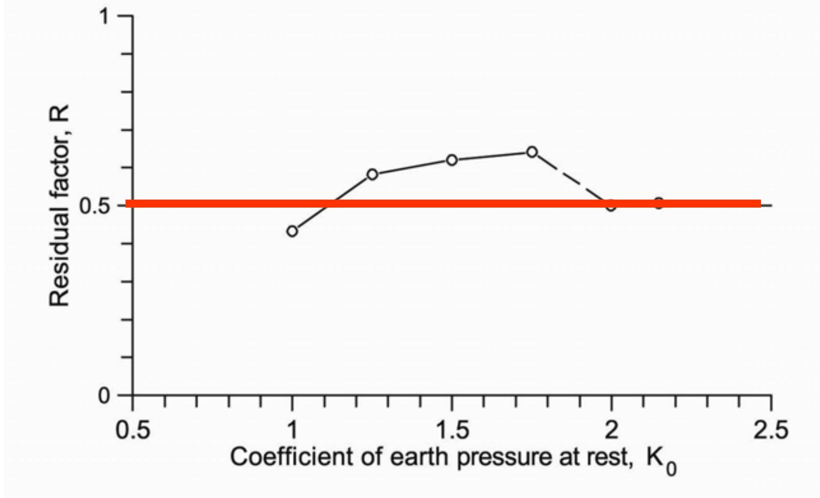
Compacted core (w_L = 74% ; w_p = 32% ; clay fraction = 62%)



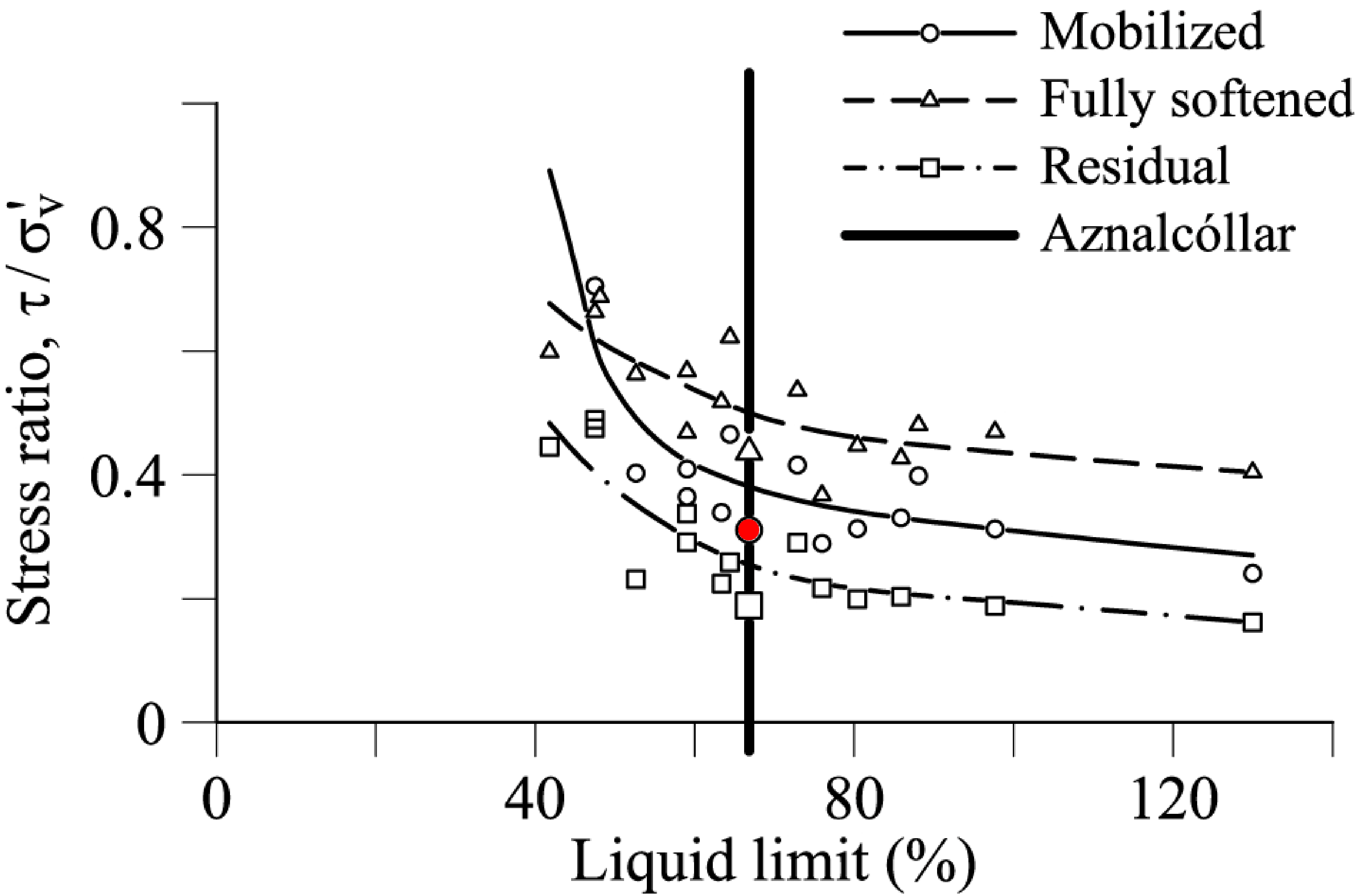
Foundation clay (Oxidized clay : w_L = 79% ; w_p = 31% ; clay fraction = 64%)

- Excavated slopes in hard clays *(Potts et al, 1998)*

Reference case: 10 m high excavated slope (3 :1)



Mobilised, fully softened and residual stress ratio for Aznalcóllar failure

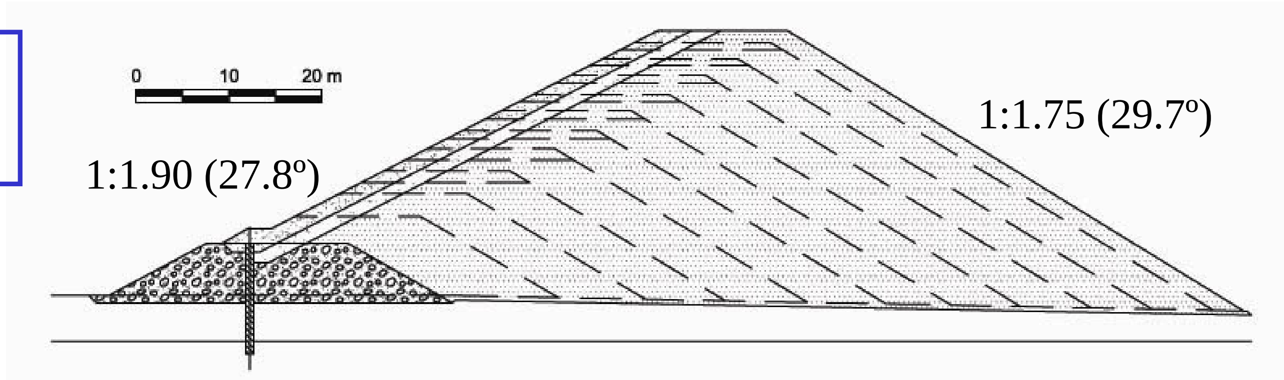


Stability calculations reported in the Project. Material parameters. Drained analysis

Soil	Original Project, 1978				Revised Project, 1996		
	c' (kPa)	φ' (º)	Relative specific weight (γ/γ_w)	Water	c' (kPa)	φ' (º)	Relative specific weight (γ/γ_w)
Tailings	0	0	**2.95**	saturated	0	0	**2.97**
Upstream red clay mantle	0	26	2.15	saturated	10	27	2.17
Filter	0	35	1.85	dry	0	35	1.85
Rock fill (schist's)	0	35	1.95	dry	0	40	1.95
Alluvial terrace	0	35	2.15	saturated	0	35	2.15
Blue marly clay	**0**	**25**	1.90	saturated	**20**	**22**	1.98

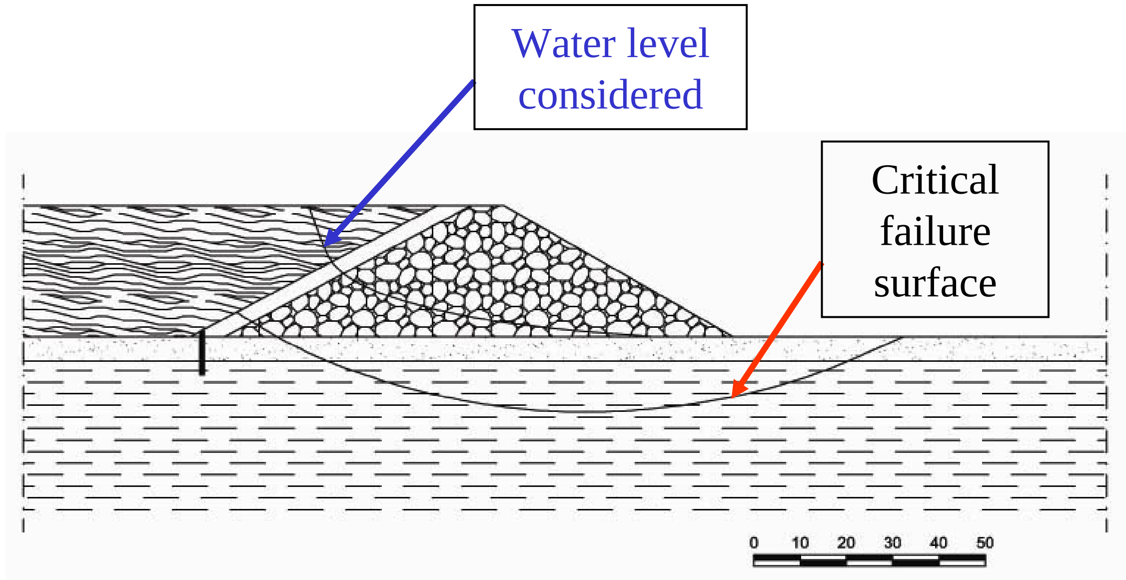
Stability calculations during the design phase

- Original Projet . Advancing geometry

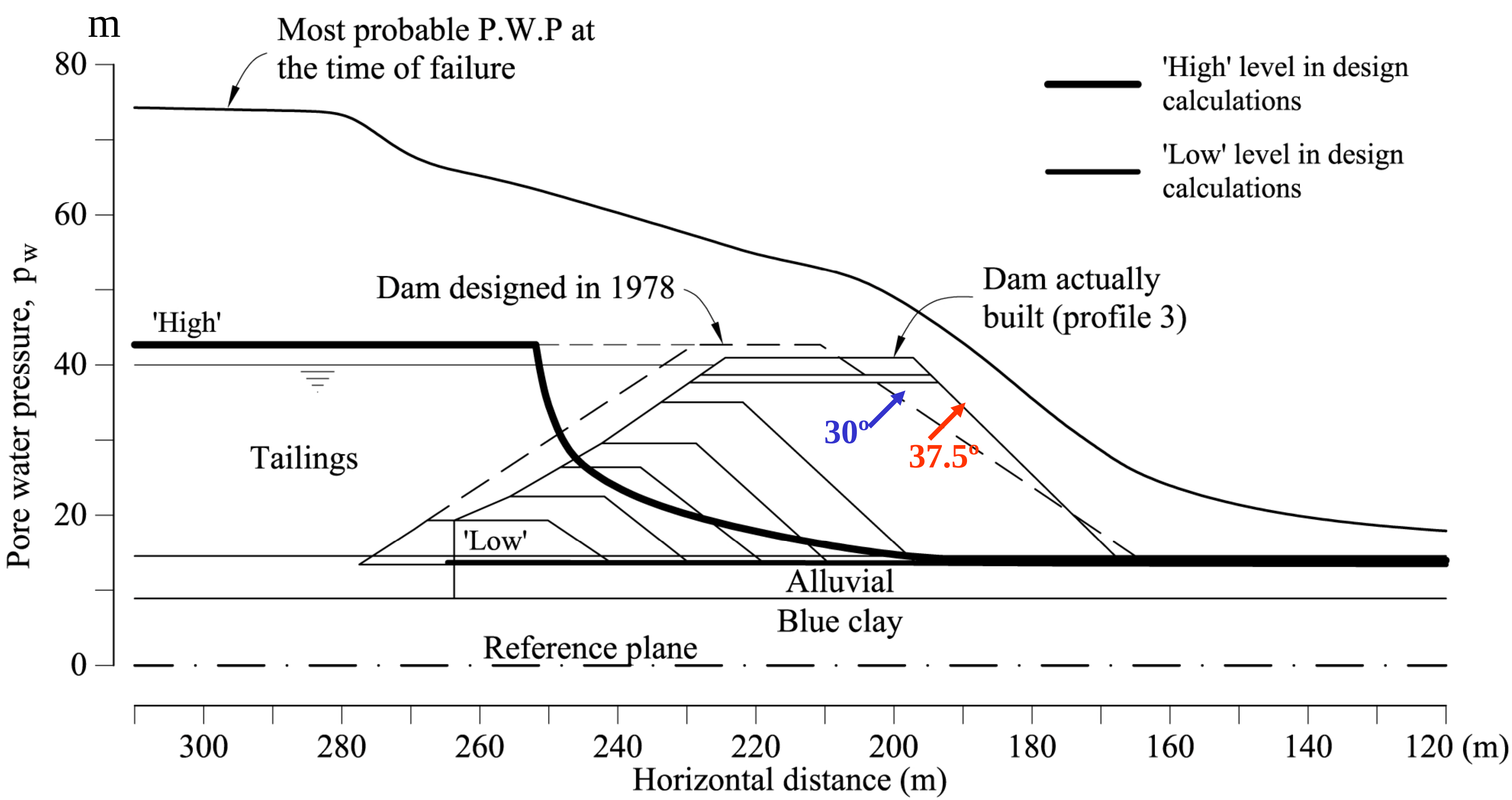


- Hypothesis:
  - Liquefied tailings
  - Seismic acceleration ah=0.048g (MSK=7) av=0.776g
  - Water level: “Stationary Conditions”
  - Calculation method: Morgenstern-Price

→ **(FS=1.3)**



Water pressures on failure plane



GENERAL. HARD CLAYS AND CLAYSTONES

- Undrained analysis goes often against safety

- The behaviour is controlled by singular surfaces. Safety analysis must be drained: $(c',\phi')_{?}$ + p_w

- Singular surfaces are often damaged

$(c',\phi')_{initial} < (c',\phi')_{peak}$

AZNALCÓLLAR

- c_u = 100-225 kPa; F.S.>2

- Failure was controlled by quasi-horizontal sedimentation planes ($i=2^o$ following the motion direction)

- If c'_p = 65 kPa; ϕ'_p =24.1º, (peak strength parameters of clay matrix in horizontal shear tests) the failure envelope is not reached

11. Final remarks

GENERAL. HARD CLAYS AND CLAYSTONES

- Progressive failure reduces the available strength

Order: $c'_p \rightarrow 0$

Failure

$\phi'_p \rightarrow \phi'_{pp} \rightarrow \phi'_{res}$

(Fully softened?)

- Brittleness and the associated evolution towards residual strength implies:
  - No warning signs (value of field instrumentation?)
  - Accelerated motion

AZNALCÓLLAR

- Failure is explained by $c' = 0$ and $\phi' = 17^o$-19^o. (There was a reduction of available strength between 1988 and 1998).
- Residual Factor R = 0.5
- Brittleness Index, I_f =0.7- 0.8

$\phi'_{res} = 11^o$ (high plasticity clay)

Maximum acceleration : 0.14 g

Maximum velocity : 20 km/h

GENERAL. HARD CLAYS AND CLAYSTONES

• Pore water pressures are controlled by "in situ" permeability. A consolidation analysis is in general required

• Stationary flow conditions are not relevant. Its use to estimate stability conditions may go against safety

AZNALCÓLLAR

• Permeability was derived from "in situ" measurements of p_w . It was small and homogeneous:

$K = 2\text{-}7 \times 10^{-9}$ cm/s

($c_v = 10^{-3}$ cm^2/s)

Laboratory tests provided similar estimates

Degree of dissipation of p_w after 20 yrs of increasing load:

15%-20% of maximum

$U = 0.15 - 0.20$

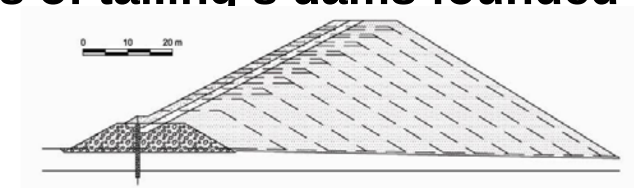
• Project designers accepted stationary conditions

AZNALCÓLLAR FAILURE

LESSONS LEARNED:

- **The difficulty to interpret, in practice, the behaviour of hard clayey soils/soft clay rocks having:**
  - **High plasticity**
  - **Marked brittleness**
  - **Low residual friction**
  - **Low permeability**
  - **A well developed system of discontinuities**

- **The risk of some construction procedures of tailing's dams founded on brittle clays**



- **The relevance of correctly estimating at the design stage of pore water pressures. Standard hypothesis (stationary flow) goes against safety**

ACKNOWLEDGEMENTS:

Antonio Gens (UPC)

Jordi Alcoverro (UPC)

José Moya (UPC)

Carlos López (UPC)

Antonio Lloret (UPC)

Josep Rius (UPC)

Jean Vaunat (UPC)

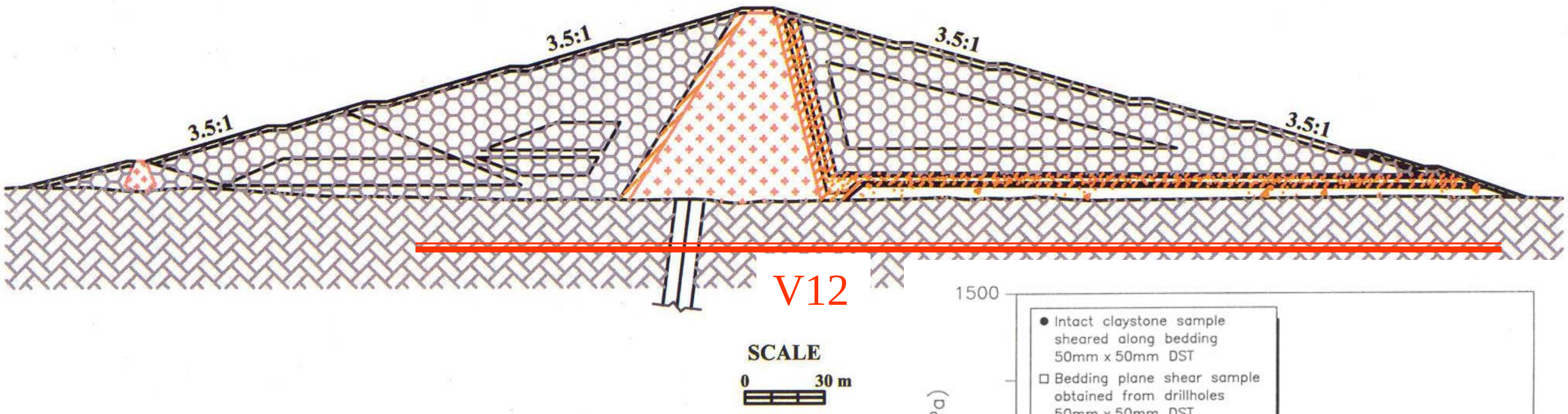
Mar Obrador (UPC)

IGTE: Instituto Geotecnológico de España (E. Custodio, P. Gumiel, J. Grime, M.Mejías)

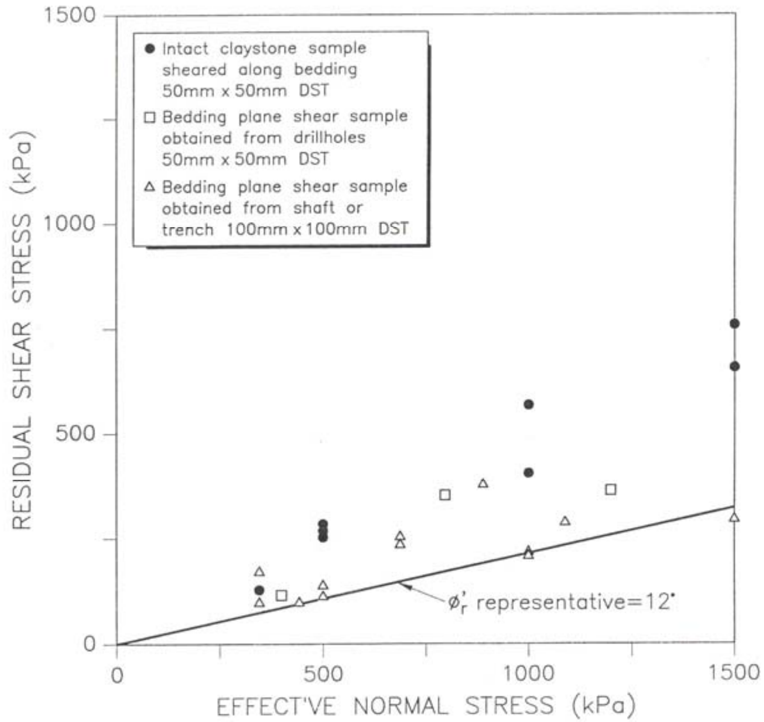
RODIO España

Examples of dams founded on clays

Oldman River dam, Canada. *(Davachi et al, 1990)*



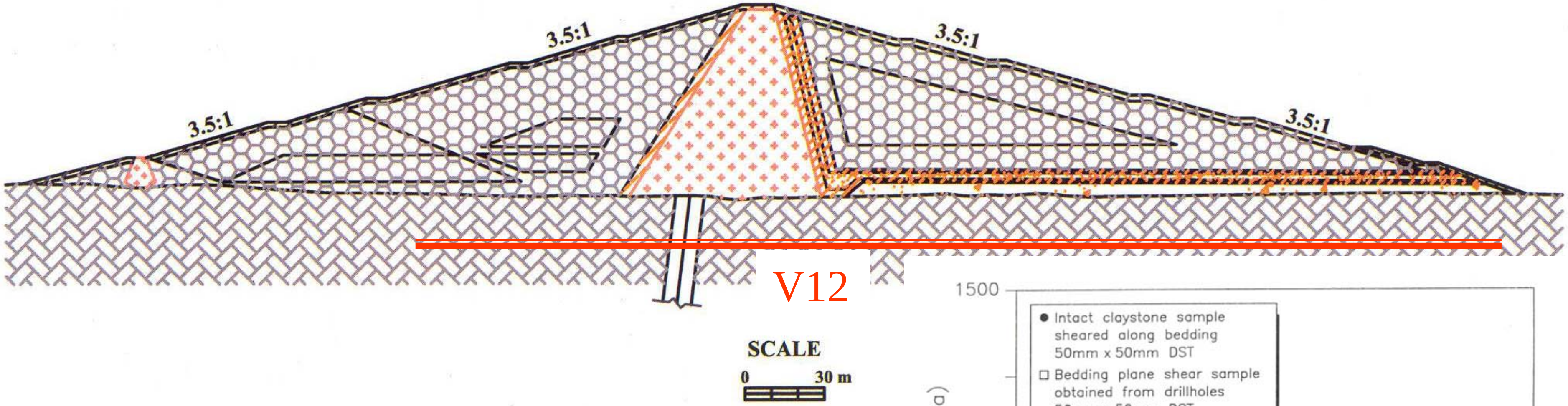
- Foundation: Hard claystones. Horizontal stratification. Frequent shear planes.
- Shoulder slopes (13.7º equivalent to 4.7:1) determined by a drained analysis. Friction adopted: 12º on critical planes



Shear strength of stratification plane V12

Examples of dams founded on clays

Oldman River dam, Canada. *(Davachi et al, 1990)*



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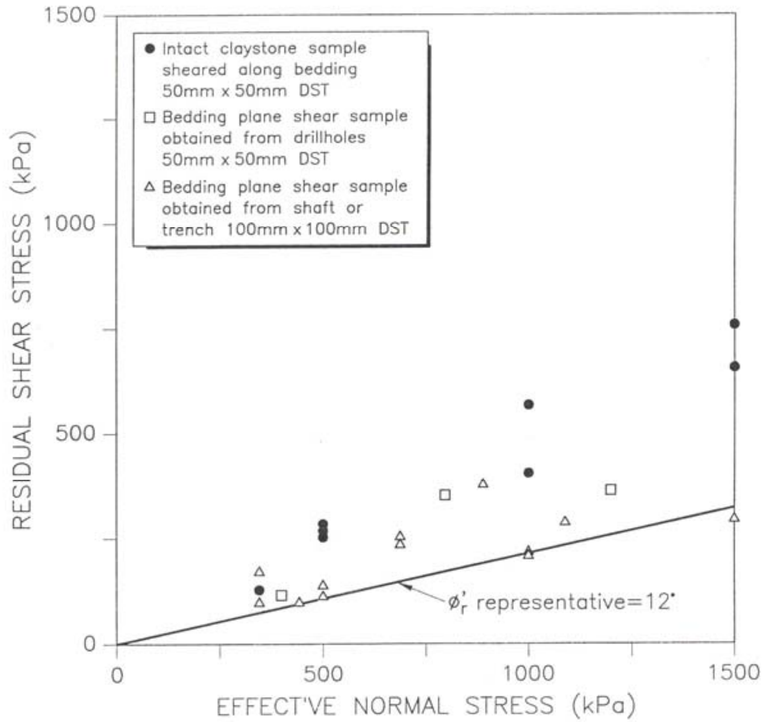
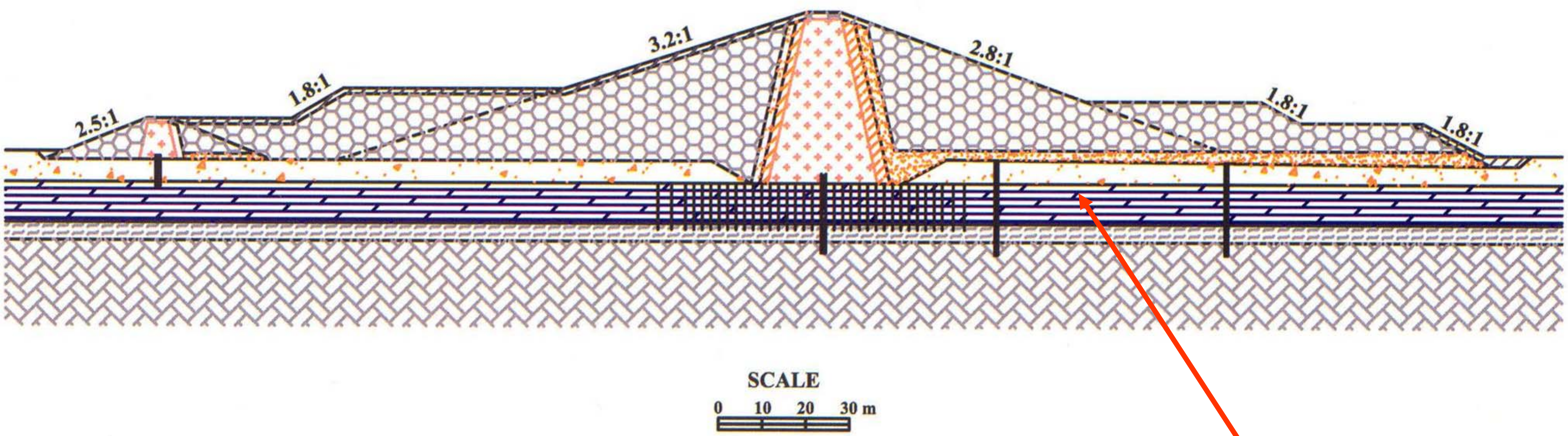


Shear strength of stratification plane V12

Examples of dams founded on clays

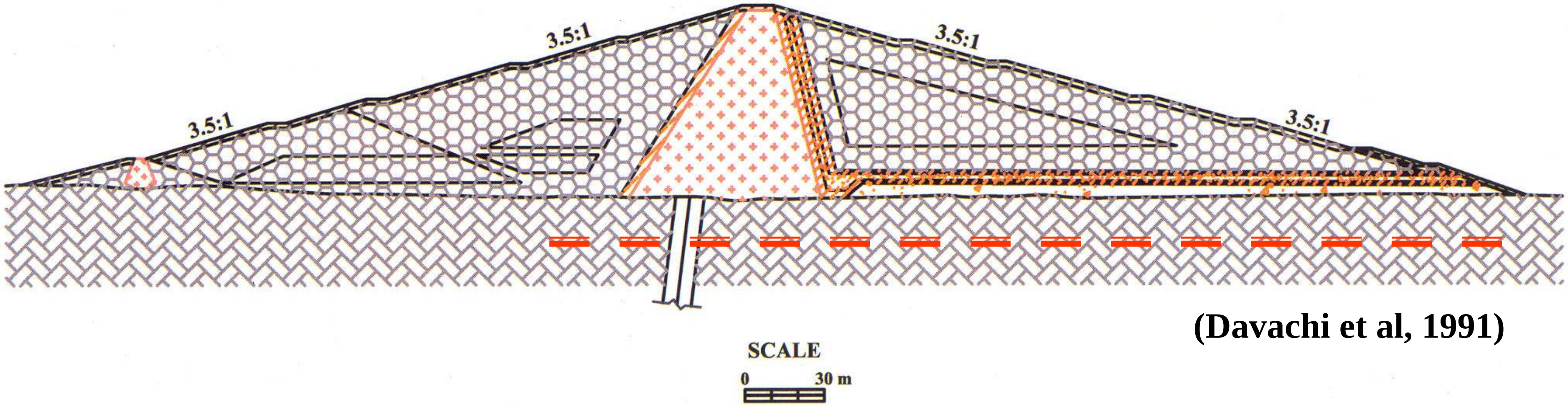
Lechago dam, Teruel, Spain. Under construction (2007)



- Dam profile is determined by the undrained resistance, very small, of the foundation soft deltaic clay

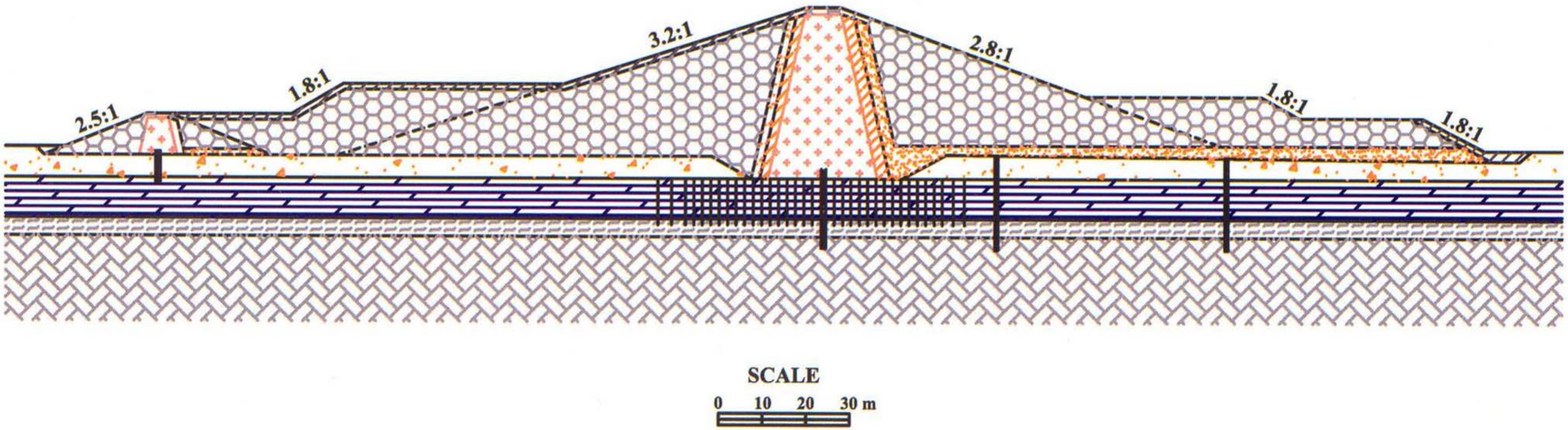
- Undrained strength of the soft clay: $0.22\sigma'_v$
- Equivalent « undrained » friction angle: 12.5º

Oldman River



(Davachi et al, 1991)

Lechago

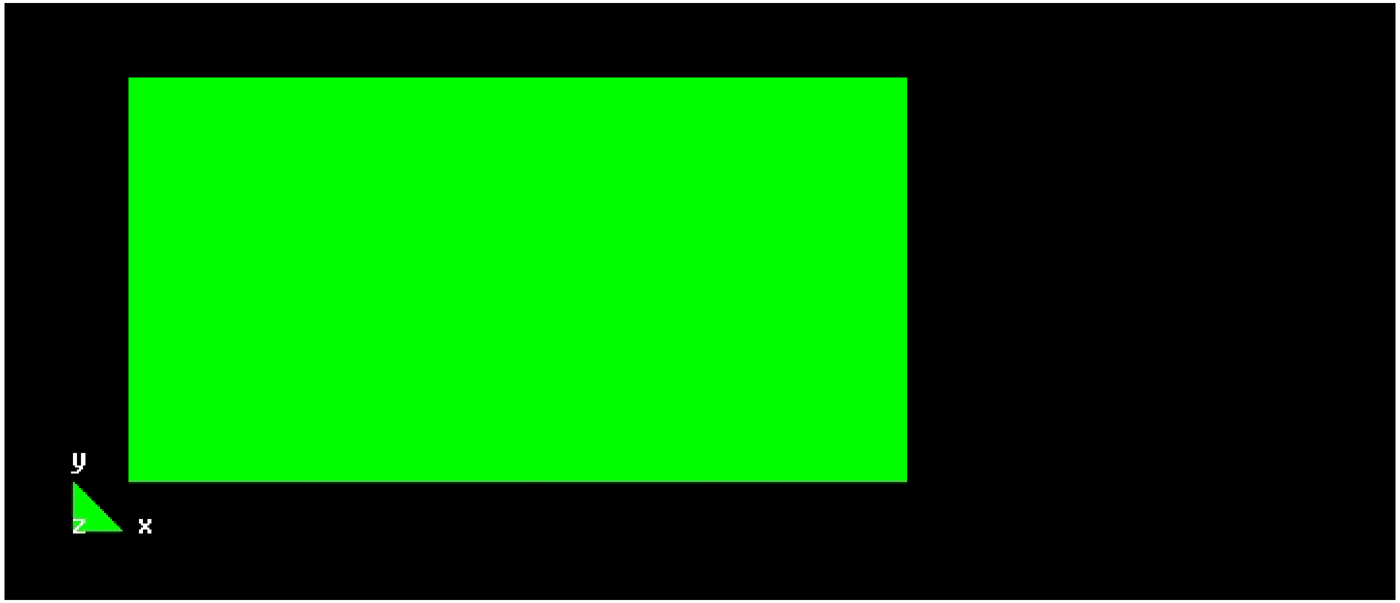


GRACIAS POR SU ATENCIÓN

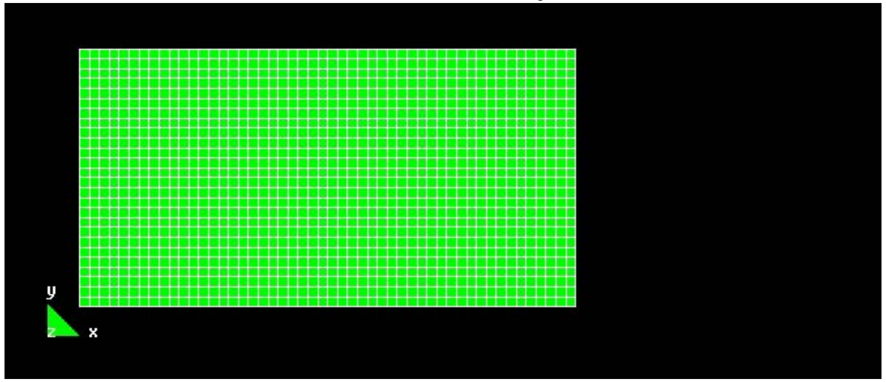
Simulación de un ensayo de corte directo

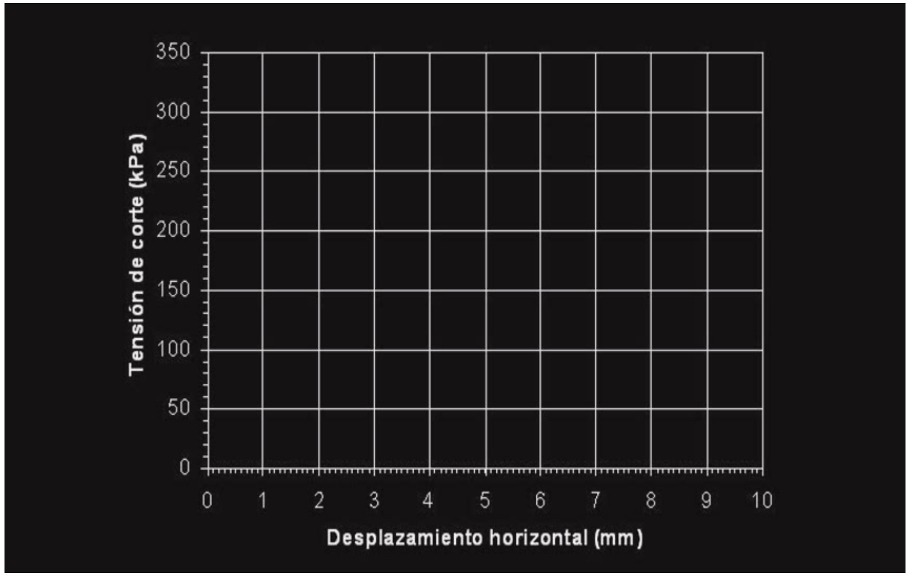
Suelo elastoplástico + daño inducido por acumulación de deformaciones irreversibles de corte. TAMAÑO DE MALLA: 1mm x 1mm

Deformación plástica de corte



Esfuerzo de corte, τ_{xy}



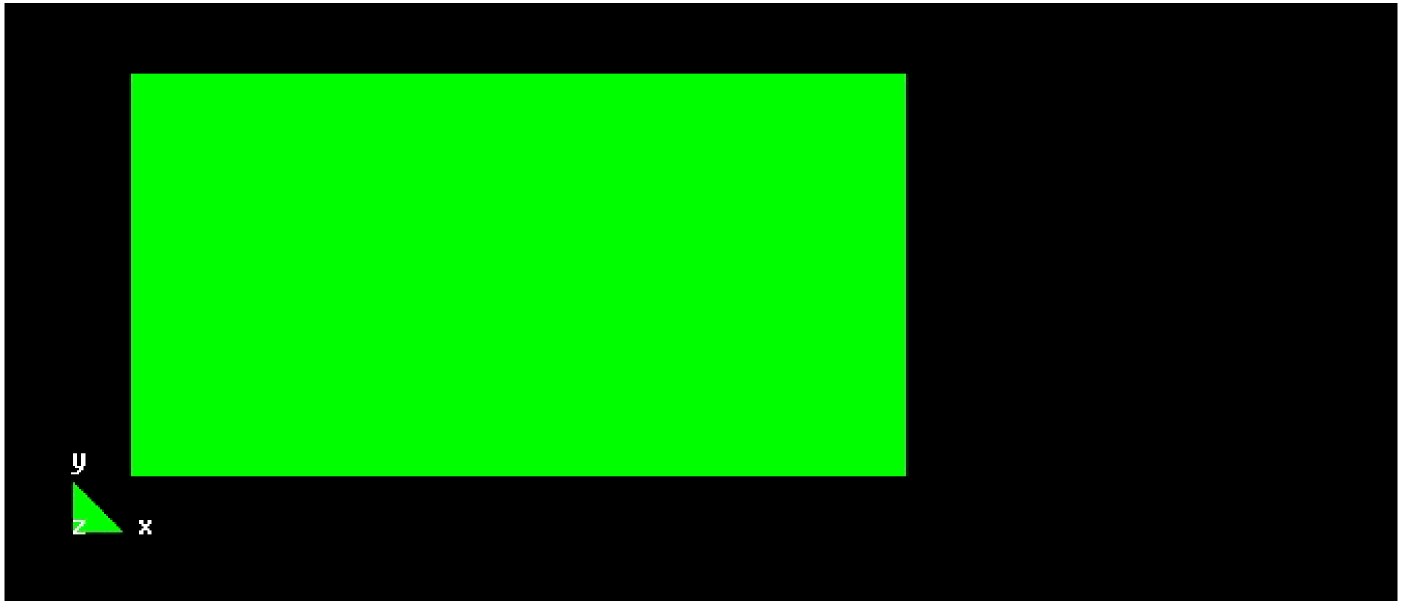


Tensión de corte vs. despl. rel.

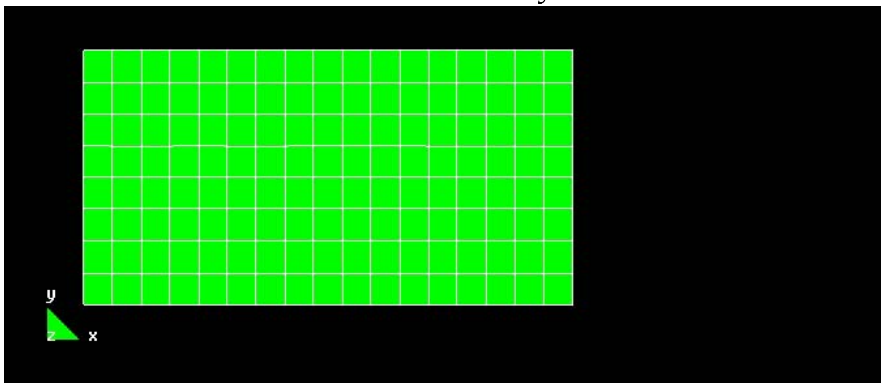
Simulación de un ensayo de corte directo

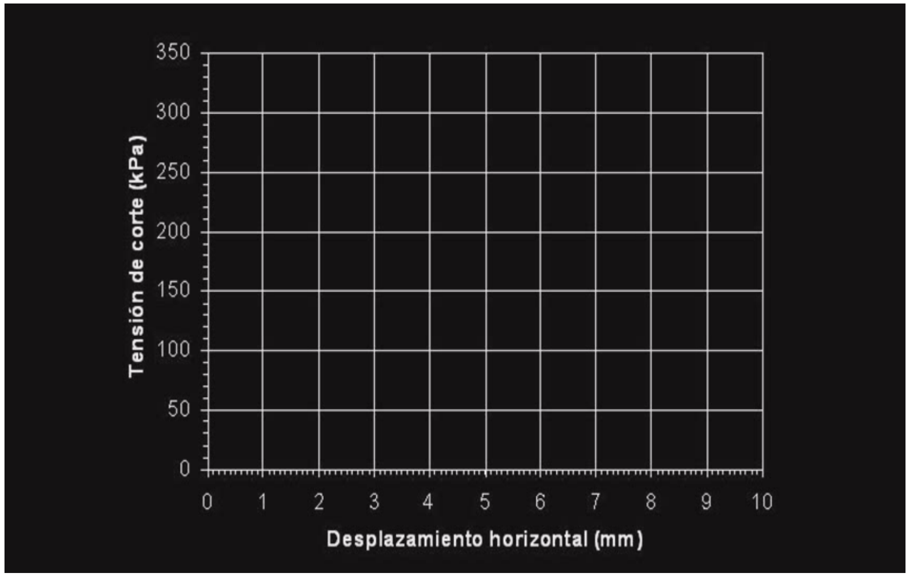
Suelo elastoplástico + daño inducido por acumulación de deformaciones irreversibles de corte. TAMAÑO DE MALLA: 3mm x 3mm

Deformación plástica de corte



Esfuerzo de corte, τ_{xy}

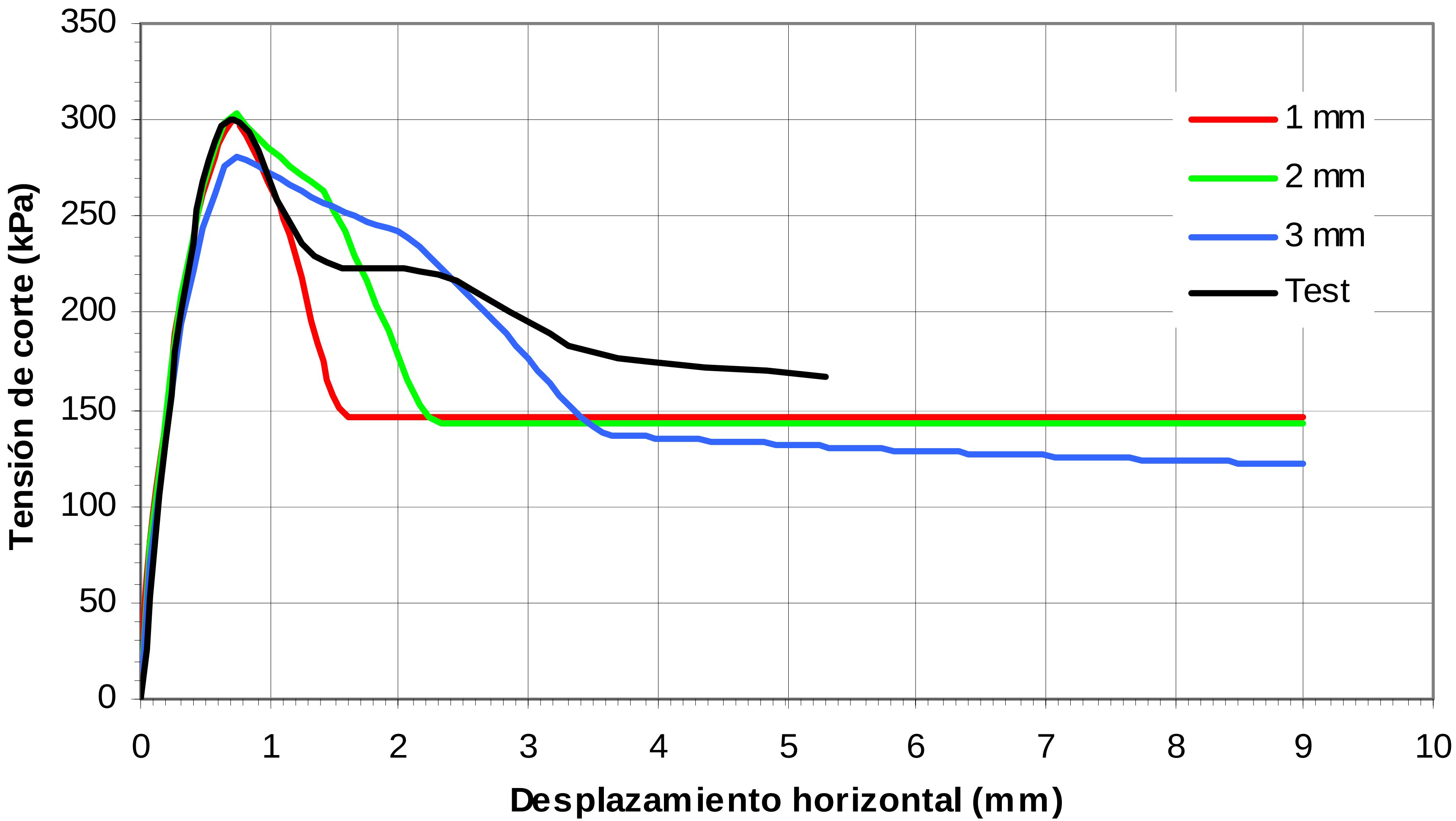




Tensión de corte vs. despl. rel.

EFECTO DEL TAMAÑO DE LA MALLA

Ensayo simulado de corte directo sobre la arcilla de cimentación de Aznalcóllar



RESUMEN DE LOS PARÁMETROS RESISTENTES EN EL CASO DE LA ROTURA DE AZNALCÓLLAR (MARGAS AZULES DEL GUADALQUIVIR)

Condiciones	Cohesión	A. fricción	Despl. relativo
Pico	65 kPa	24º	0
Post Pico	0	24º	1 mm
Post Pico	0	18º-20º	6 mm
Residual	0	11º	Centímetros

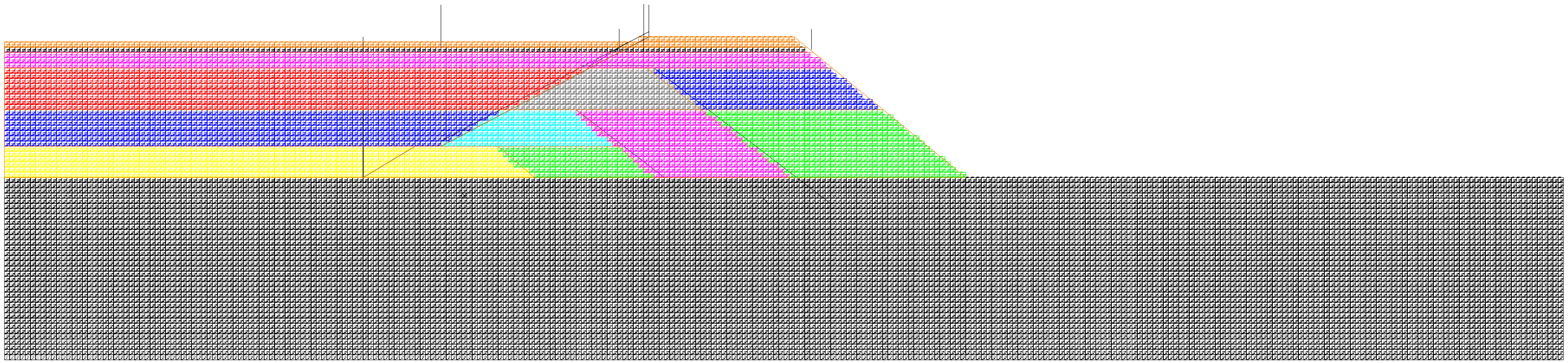
La rotura se explica con c' = 0 y ϕ' = 17º (en equilibrio límite, es decir , en condiciones medias)

WORK IN PROGRESS. Simulation of Aznalcóllar failure using the "Material Point Method" (F. Zabala, U. de San Juan, Argentina/UPC)

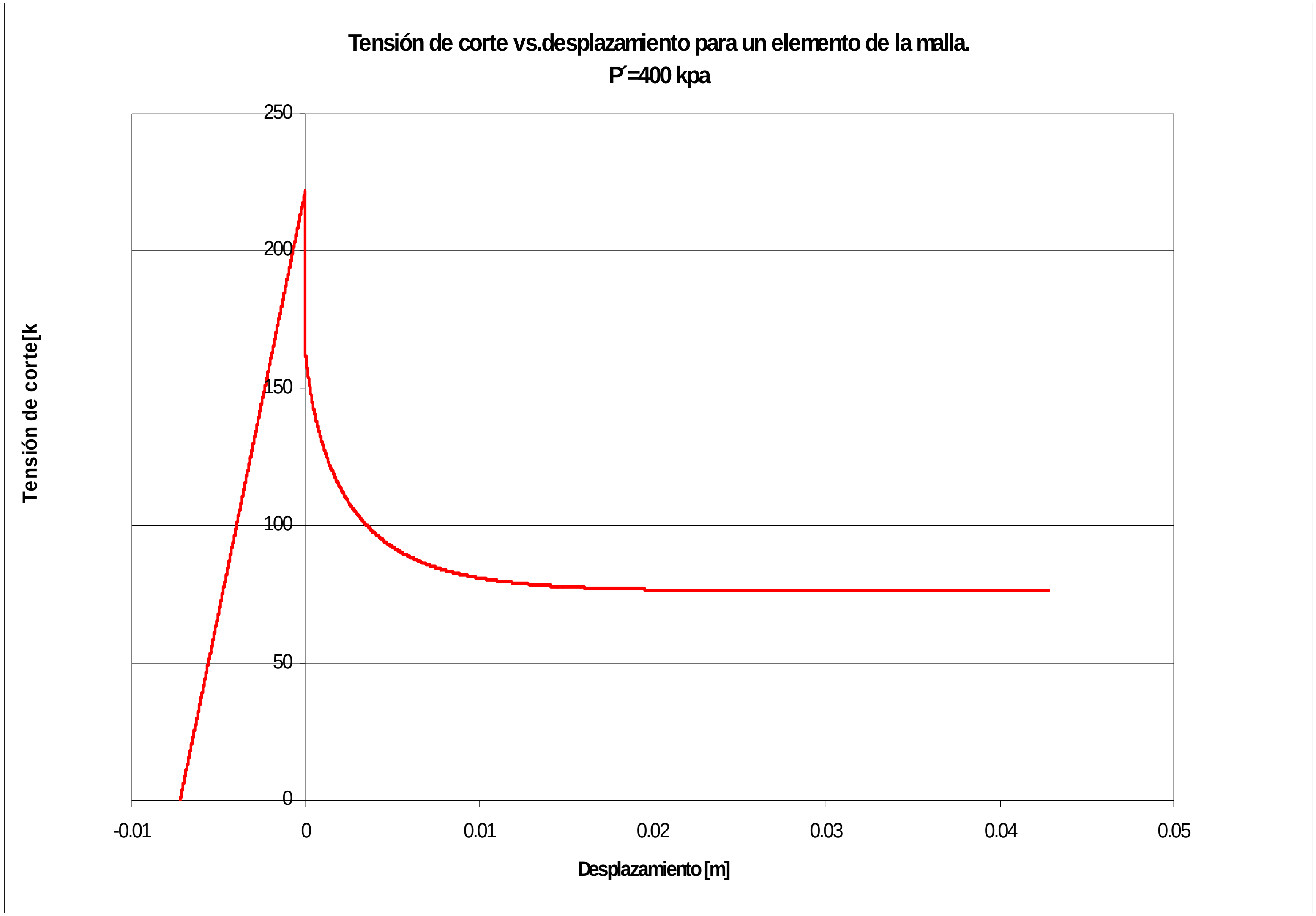
(The initial idea: *Sulsky D., Schreyer H. L., & Zhou S-J, "Application of a Particle-in-Cell Method to Solid Mechanics", Computer Physics Communications, vol. 87, pp. 236-252, 1995)*

Mass Points+ Auxiliary Mesh	• Mass, velocity, deformation and stress are assigned to points • Conservation equations (interaction between particles) are solved in the nodes of an auxiliary mesh

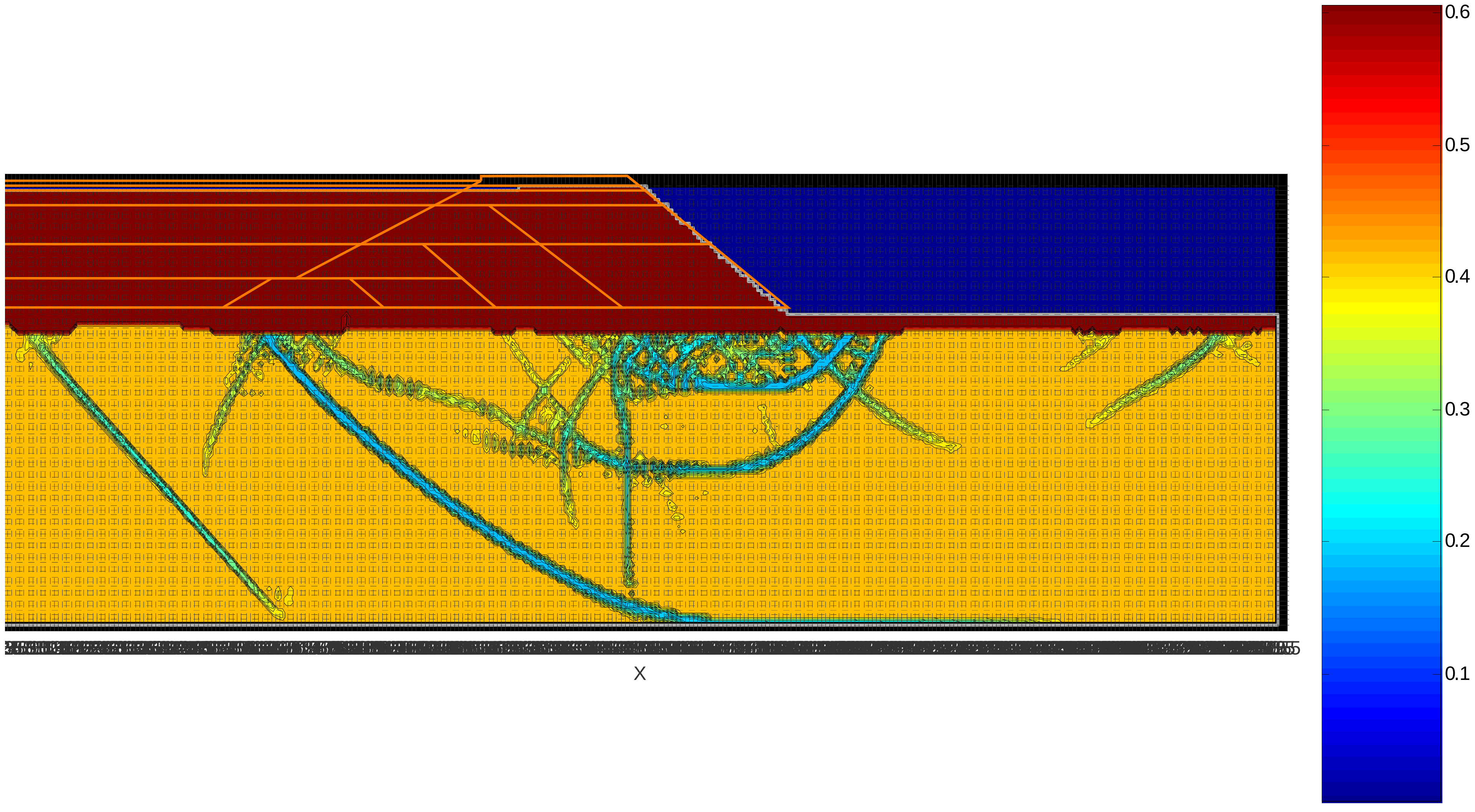
- 23.000 particles
- Fixed rectangular mesh. 27600 elements. Size of element: 1mx1m, 300 columns and 92 rows: 27600 elements



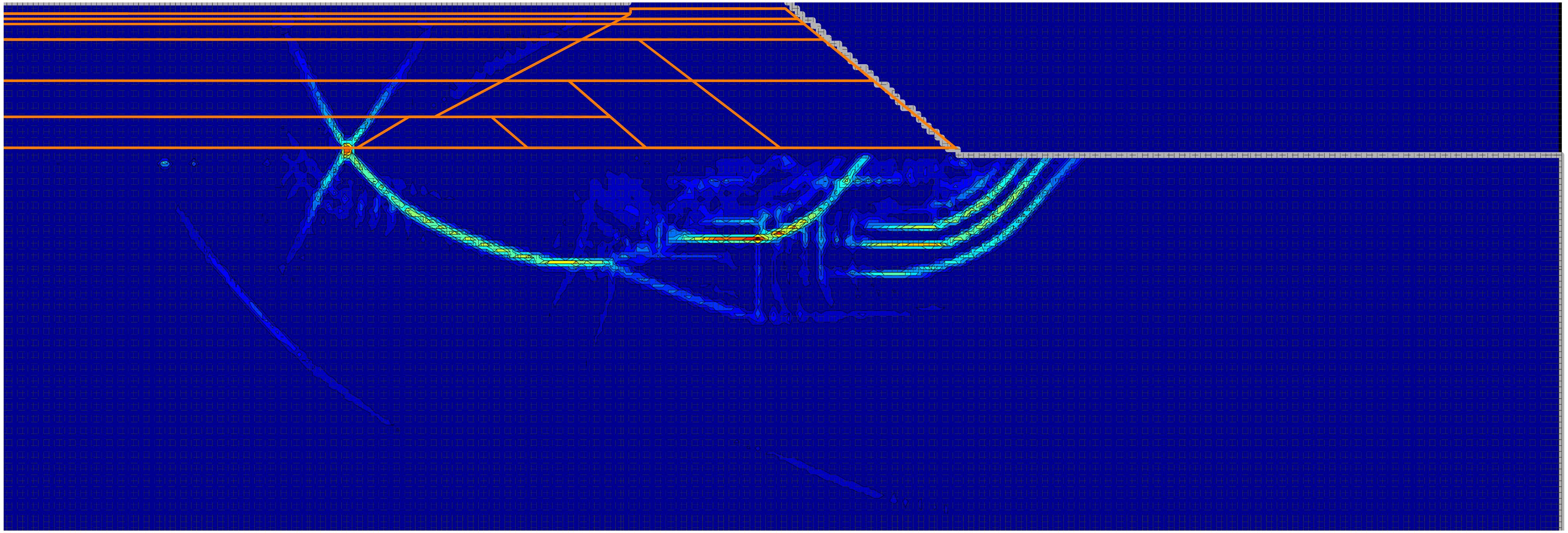
Simulation of a simple shear test of a 1x1 m of the mesh



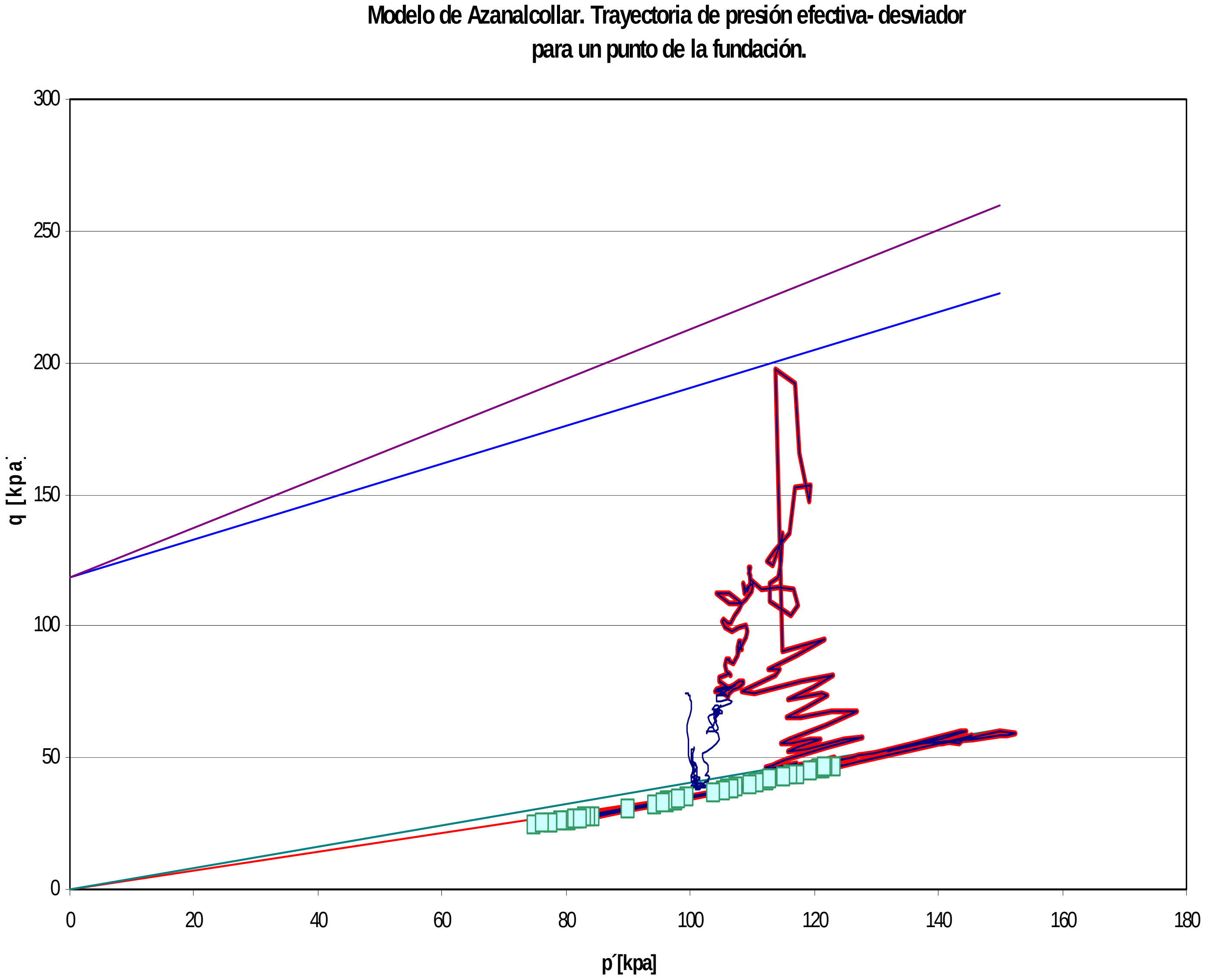
Contours of mobilized friction angle (rad)



PLASTIC SHEAR CONTOURS



Stress path in a failed point of the foundation



2. The geometry of the failure

Slickensided plane found in boring S-2.1 at the position of the failure surface

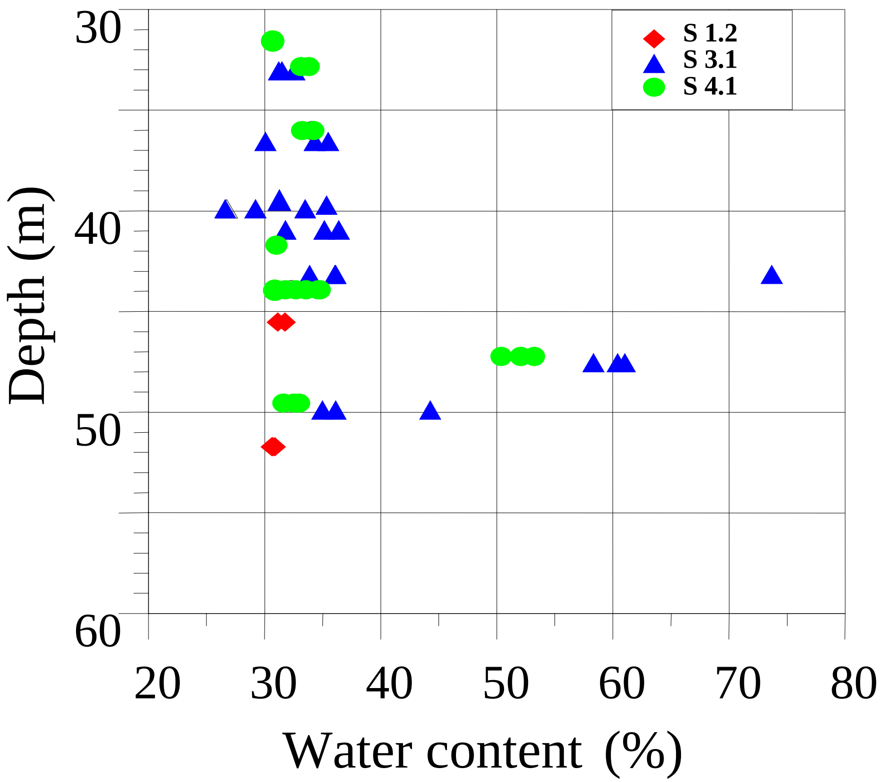
Boring S-2.1

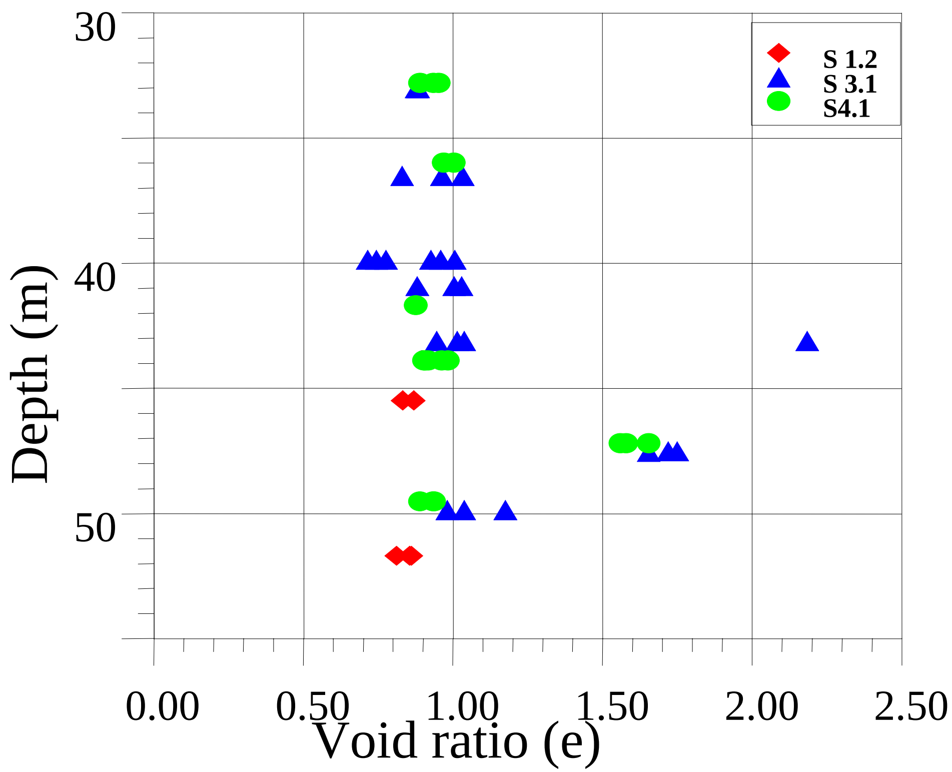
Depth: 33.8 m

4. Geotechnical characteristics of foundation clay

Basic identification

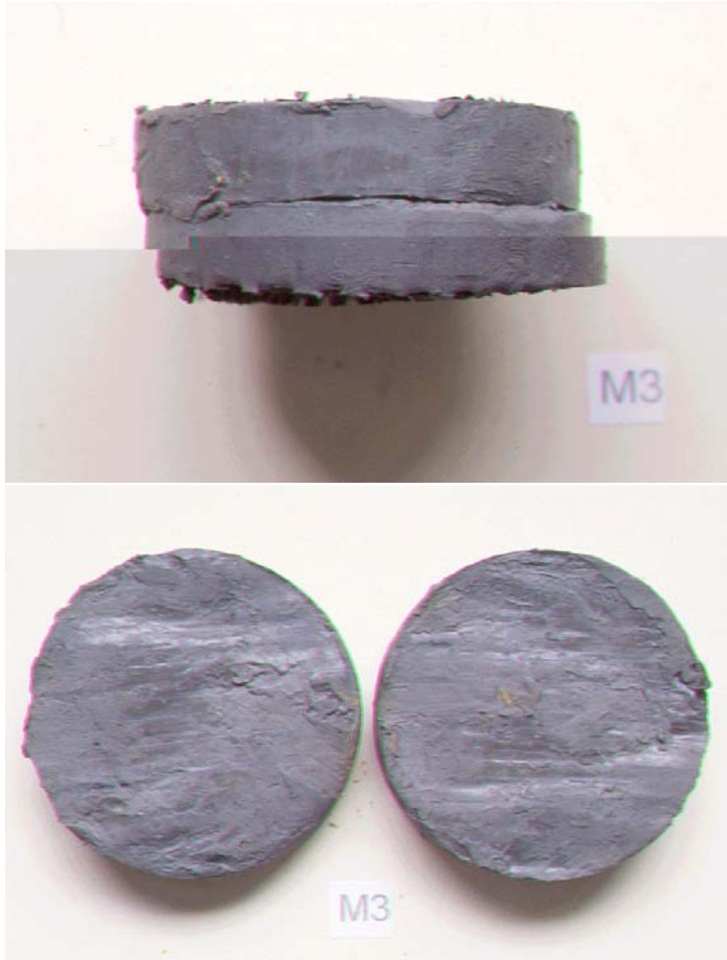
- Slight variation of density, water content and void ratio with depth (w = 30-35%; γ_{nat}= 1.90-1.98 g/cc; e = 0.8 - 1.0)
- Percentage of : Fines: >98% ; Clay: 47-58%
- Plasticity: w_l = 63 - 67% ; IP = 32 – 35%
- Classification: MH or CH

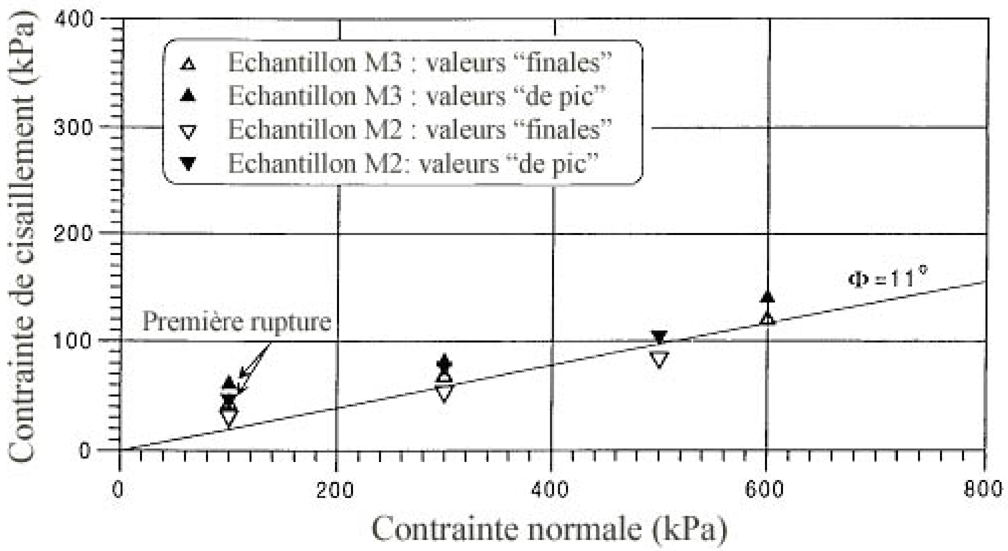




4. Geotechnical characteristics of foundation clay

Direct shear tests on natural discontinuities

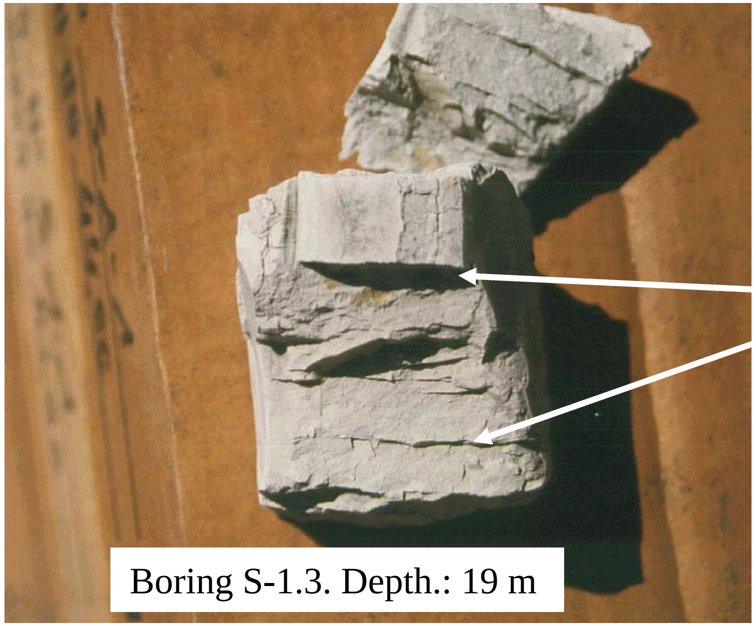




- Strength parameters:

  c'=0; ϕ'= 11°

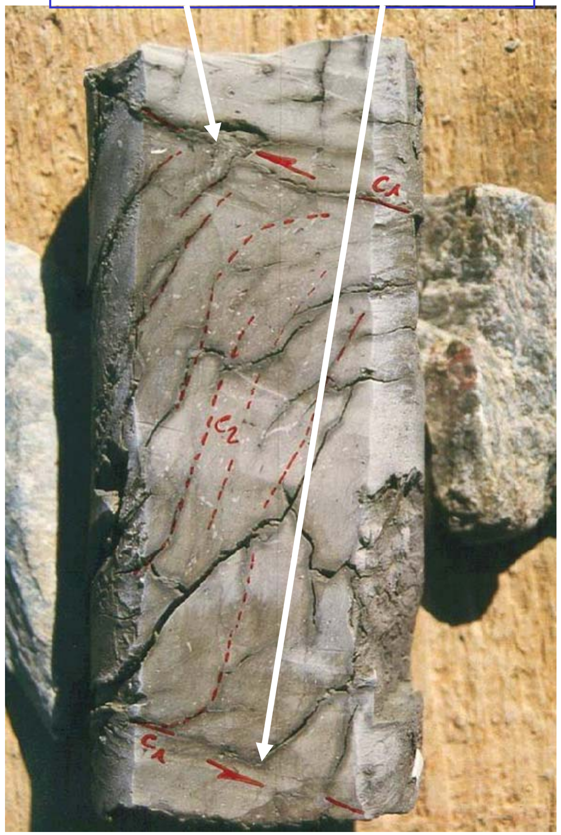
Observations in cores

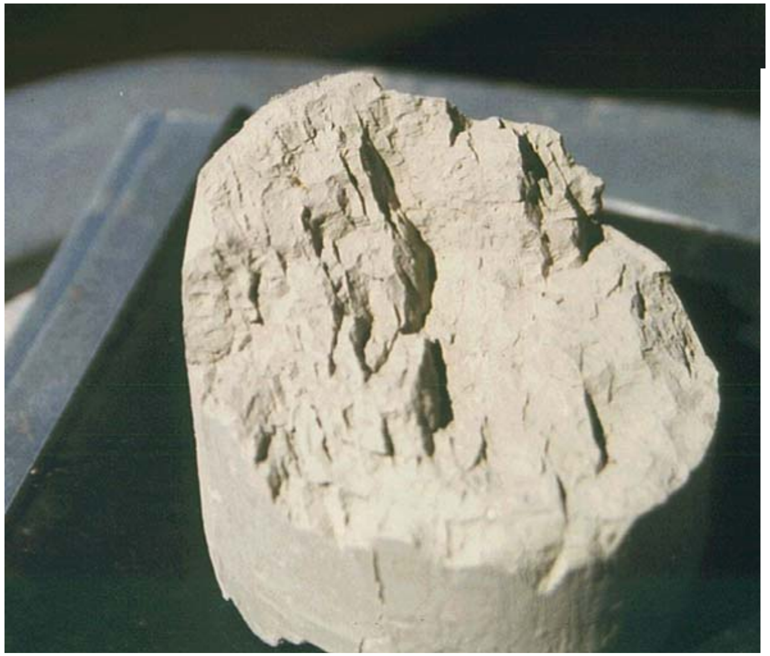


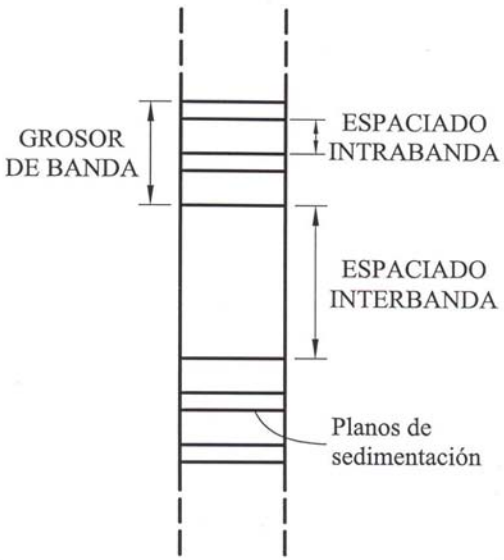
Boring S-1.3. Depth.: 19 m

Stratification Joints
(spacing: 10-15 mm)

Shear planes

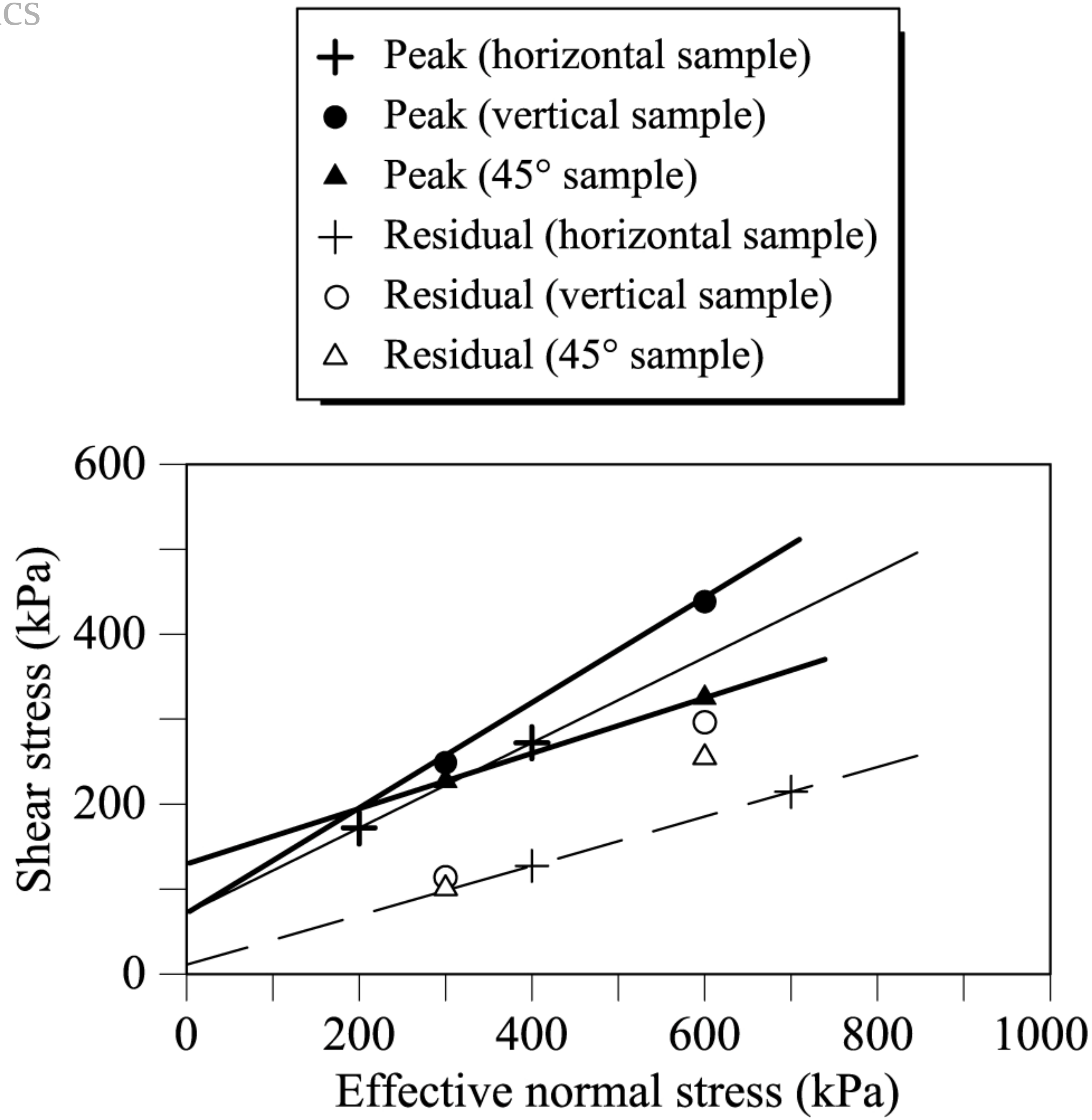






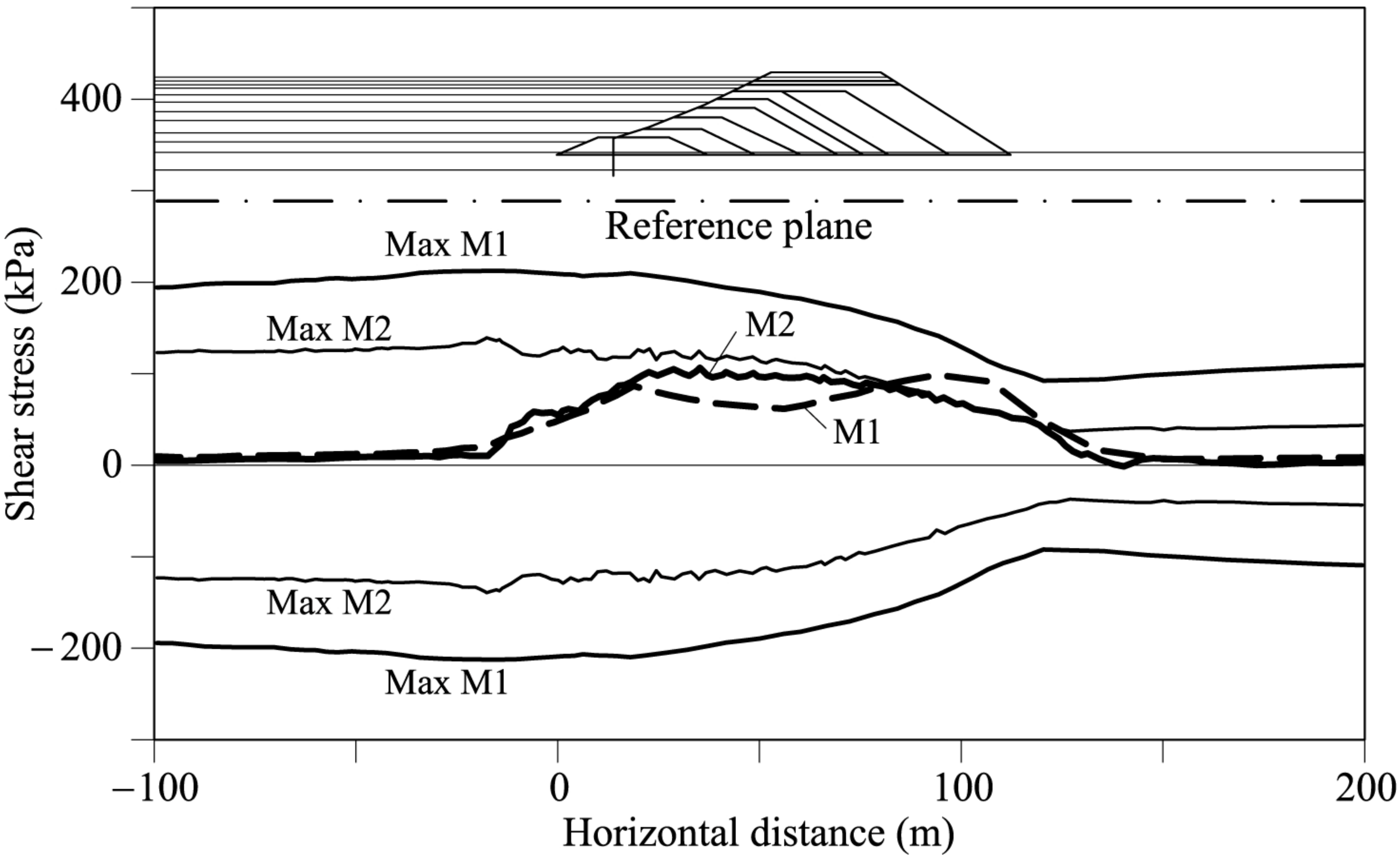
Clay matrix.

Drained direct shear

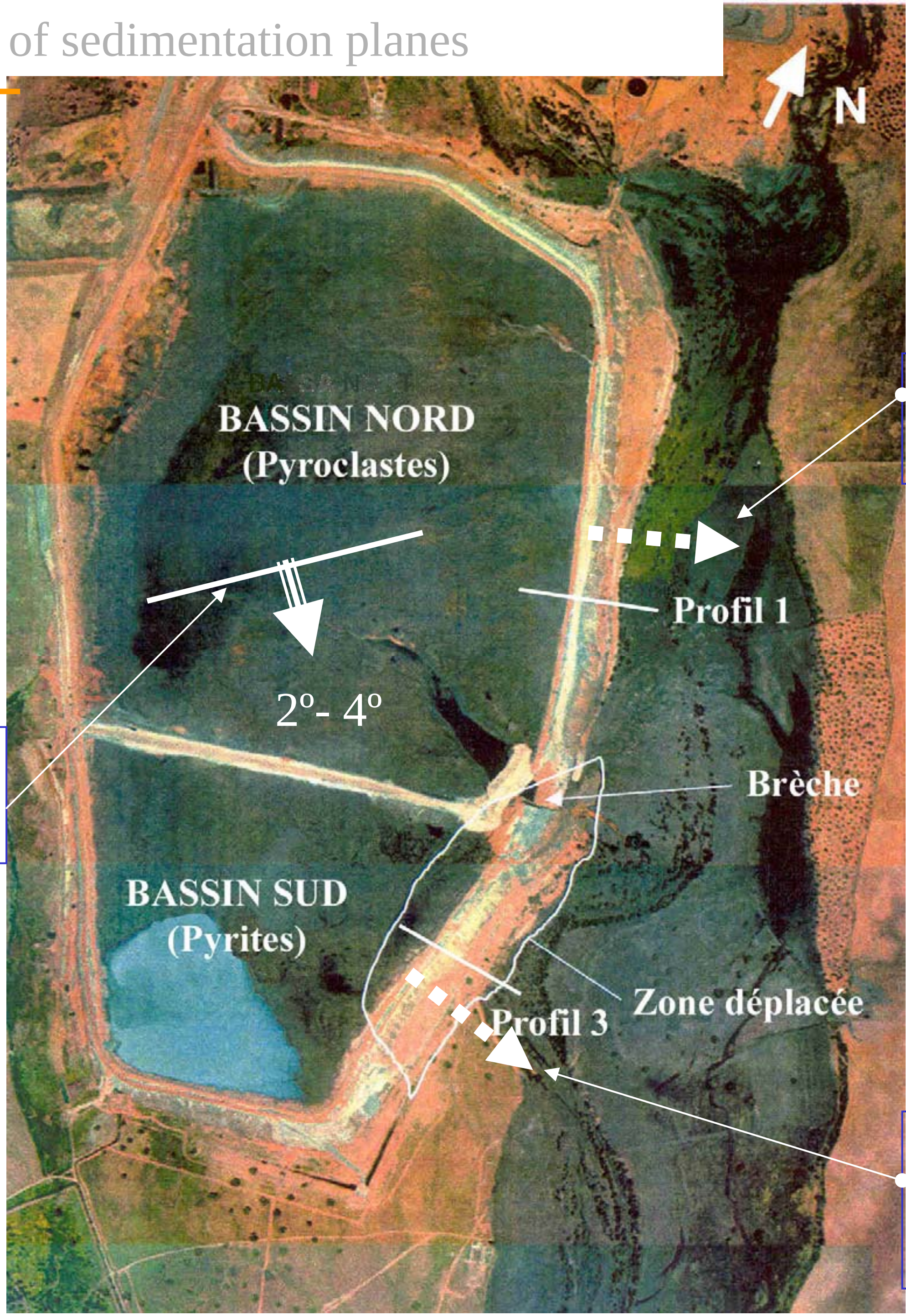


- Limited strength anisotropy

Distribution of shear stresses on a horizontal plane located 10 m below the upper surface of blue clay



8. Influence of dip of sedimentation planes



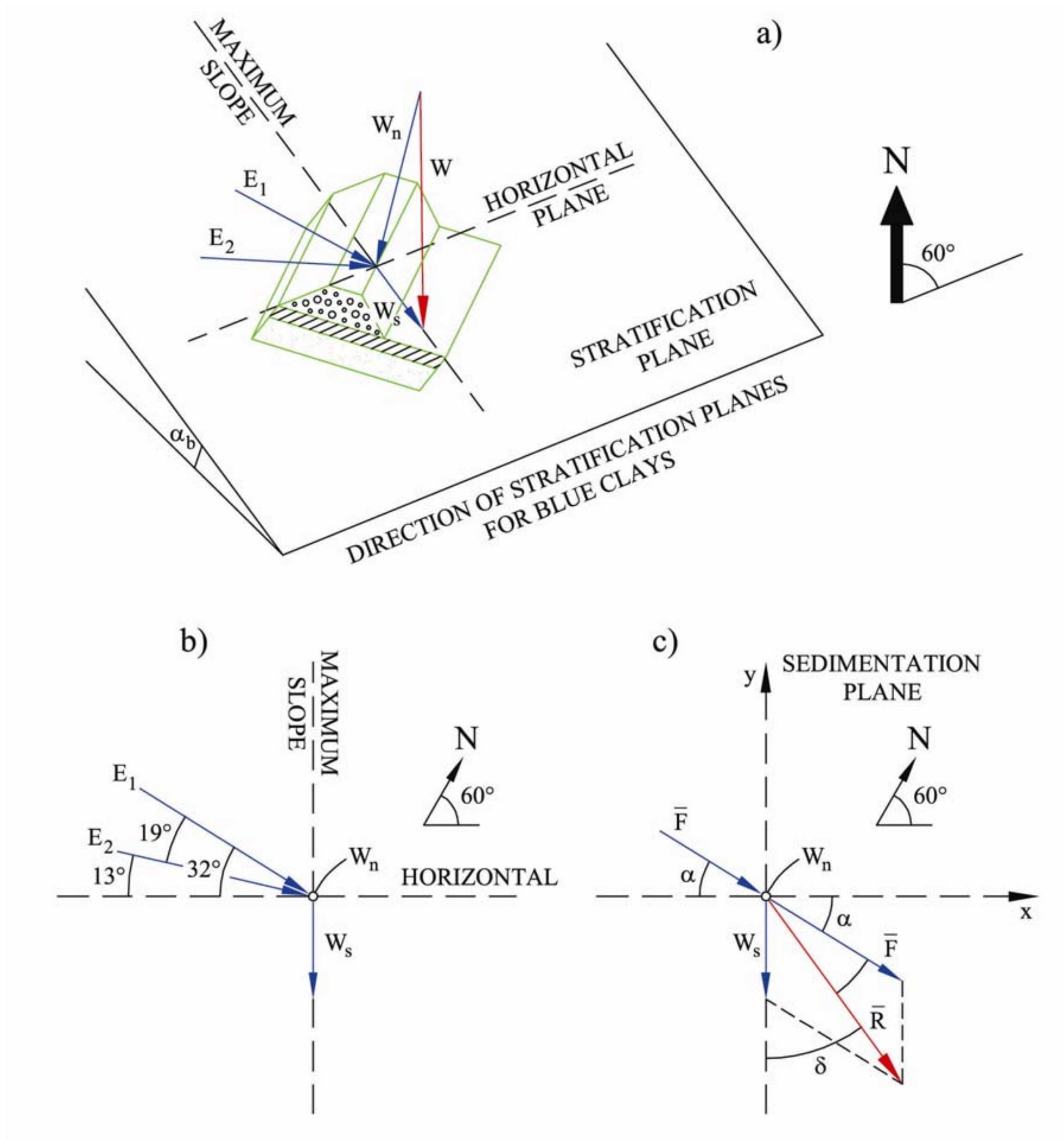
Stratification of blue clay

Direction of thrust. North Basin

Direction of thrust. South Basin

8. Influence of dip of sedimentation planes

- E_1: Tailings thrust.
  South Basin
- E_2: North Basin
- W: Weight of dyke and of the associated foundation slab
- W_s: Weight component in the direction of maximum slope ($W.\sin\alpha_b$)
- W_n: Weight component in the normal direction to the sedimentation planes ($W.\cos\alpha_b$)
- α: Direction of tailings thrust
- δ: Direction of total pushing force (motion of dyke)



Unstabilizing force (tailings thrust) and motion direction

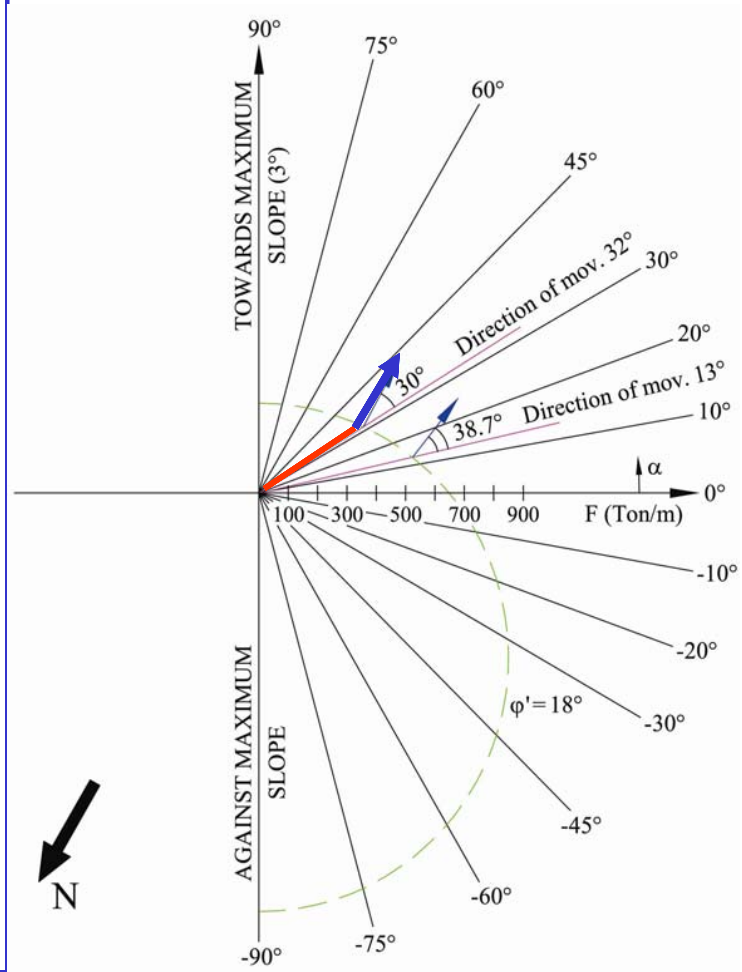
- Equilibrium

$$W_n' \tan\varphi' = \left|\bar{F} + \bar{W}_s\right|$$

$$F^2 + 2FW \,\mathrm{sen}\,\alpha_b \mathrm{sen}\,\alpha + W^2\,\mathrm{sen}^2\,\alpha_b - W_n'^2 \tan^2\varphi' = 0$$

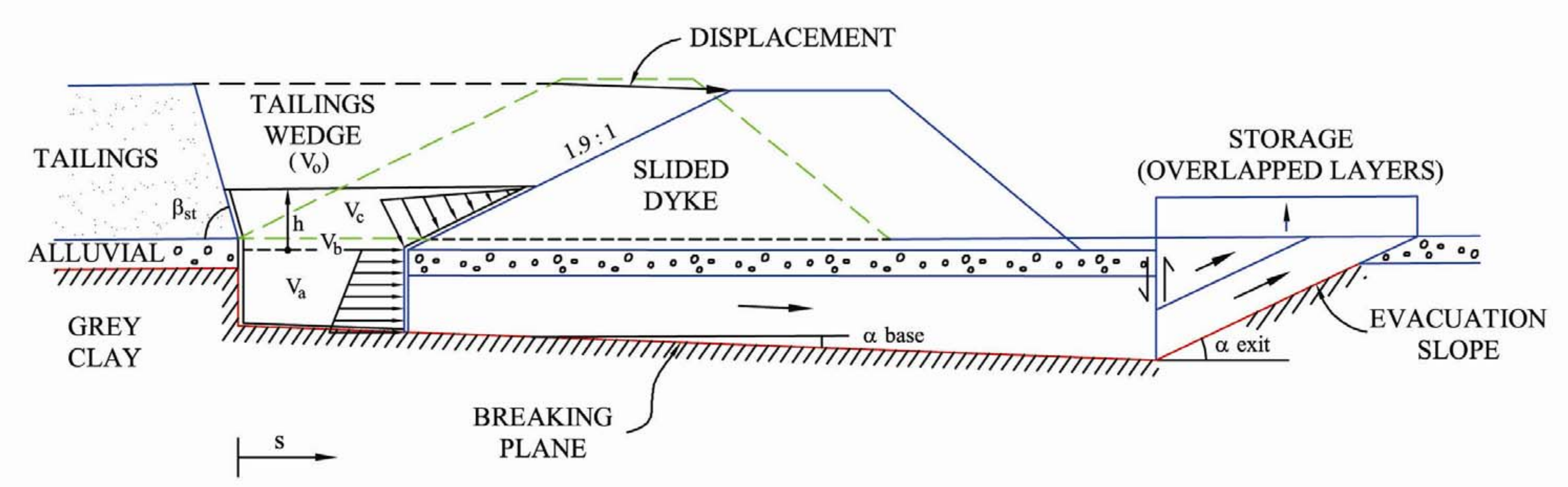
- Force to displace the dyke:
  - North Basin : 538 Ton/m
  - South Basin : 420 Ton/m

- NOTE that motion direction is different from thrust direction (consistent with “in situ” observations)



Main ideas

- The solid rigid motion of the dyke provides an opportunity for a simple analysis
- The total displacement of the dyke (known by field observations) allows calibration of the model
- Unknown aspects (velocity, acceleration) may be derived



Conservation of the volume of liquefied tailings: $V_0 = V_a + V_b + V_c \rightarrow h = h(s)$

Forces against the moving mass

$$\sum \overline{F} = \sum \overline{F}_d + \sum \overline{F}_R = M\overline{a} \qquad (1)$$

$\overline{F}_d$: Acting forces (s) $\overline{F}_R$: Resistant forces (s)

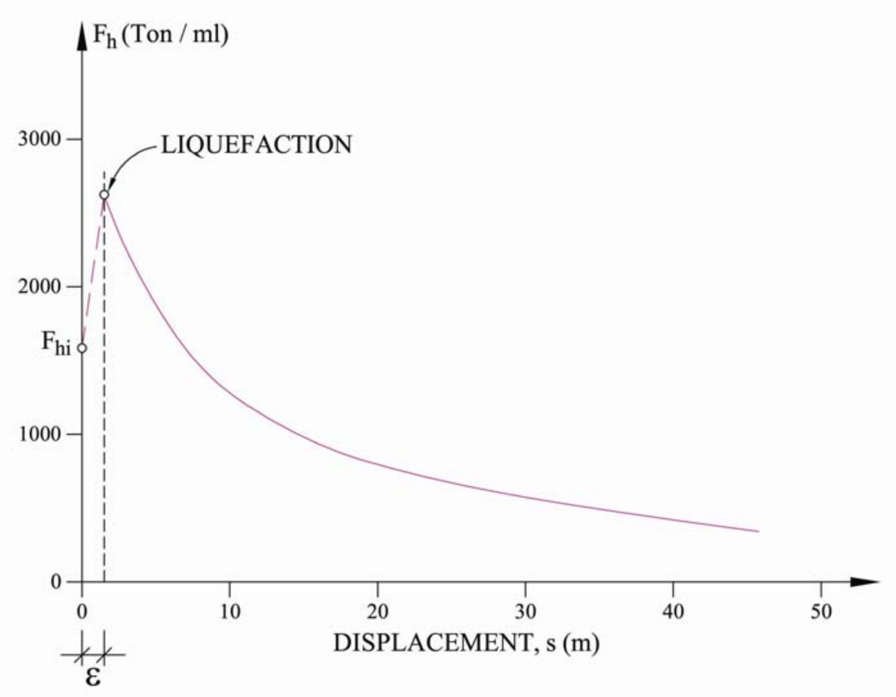
- Displacement, s, depends on time: s(t). Nonlinear differential equation (1) may be integrated, step by step, to obtain the history of motion

Thrust of tailings

- Force by liquefied tailings:

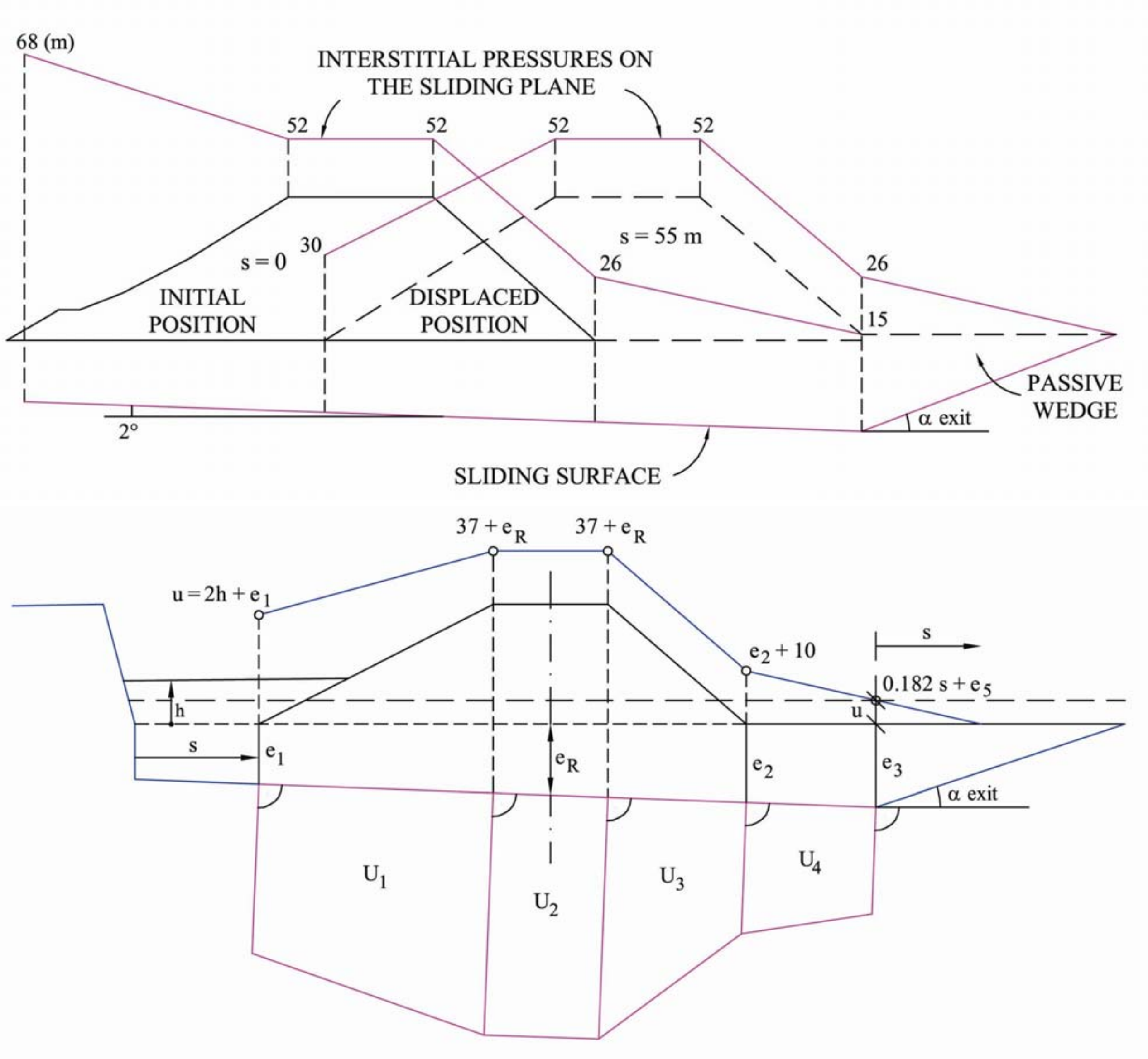
$F_h = e_1 \gamma_e (h + e_1/2) + h^2 \gamma_e/2 \quad (\gamma_e = 3.1t/m3)$

- F initial (K_0=0.5): $F_{hi} \cong 1403 \quad Ton/ml$
- If liquefaction: $F_{hl} \cong 2605 \quad Ton/ml$
- ε : Model parameter

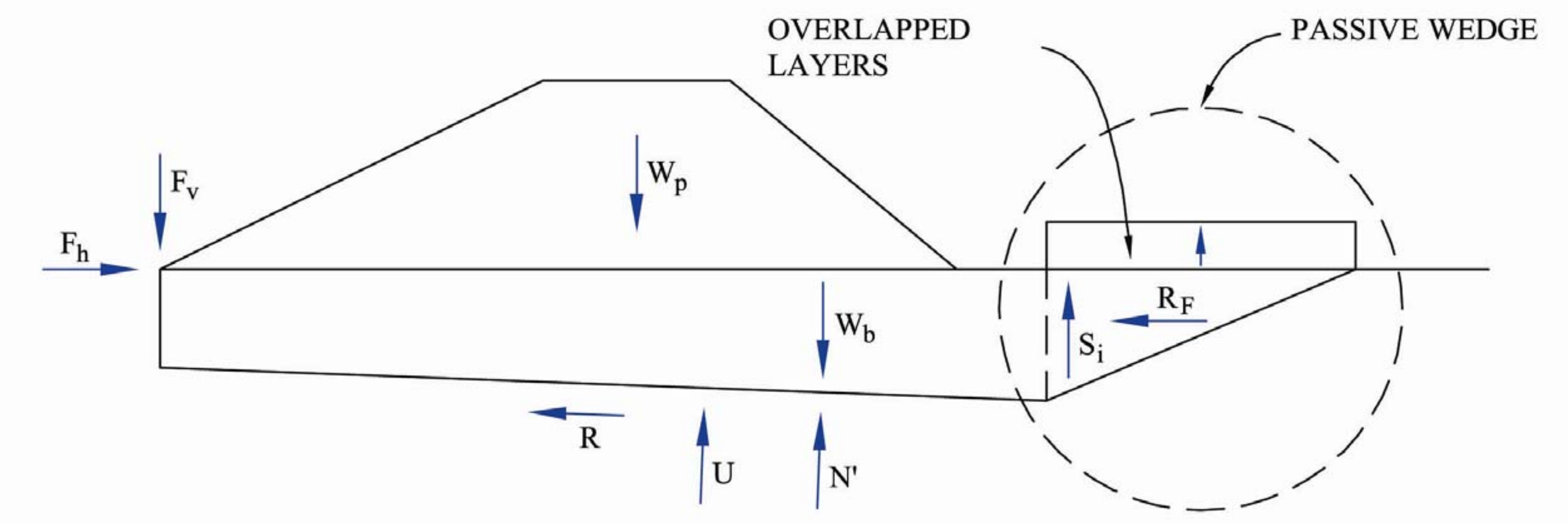


Resistant forces. Pore water pressures

- Rapid displacement of the dyke induces an undrained loading
- Central and downstream parts: Pwp controlled by dyke weight
- Upstream: Pwp controlled by height of tailings



Resistance to sliding. Equilibrium. Basal surface

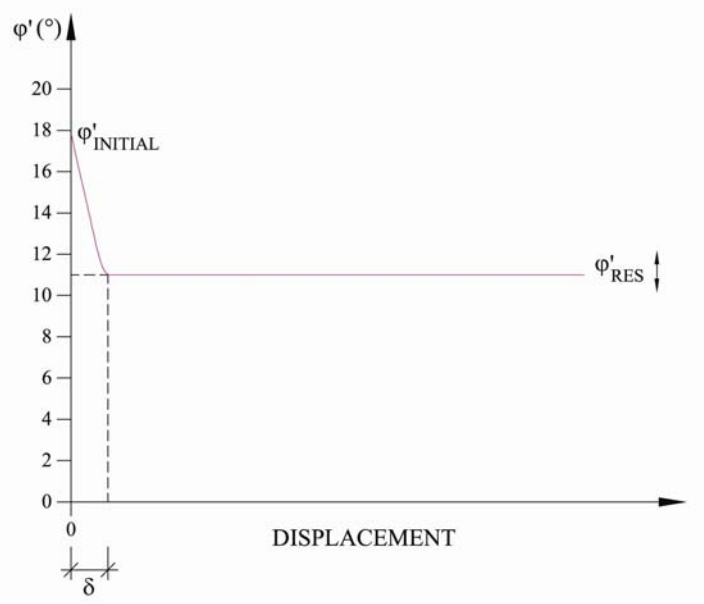


Equilibrium in vertical direction: $N' = \left(F_v + W_p + W_b - S_i - U \cos \alpha_b\right) / \left(\cos \alpha_b + \text{sen } \alpha_b \tan \varphi'\right)$

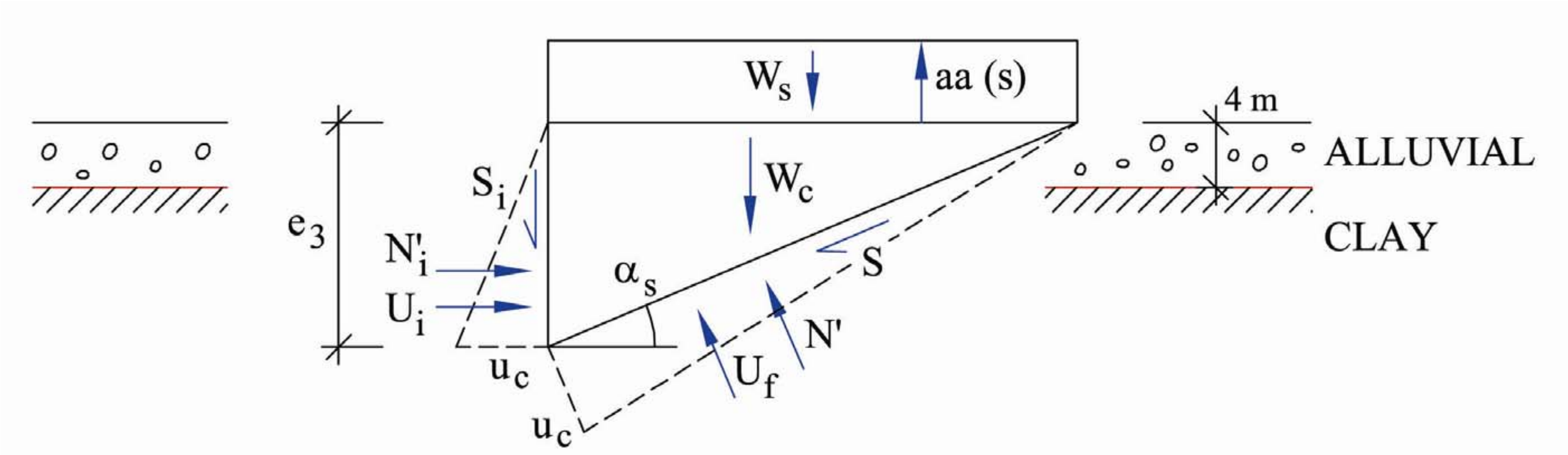
$$R = N' \tan \varphi'$$

Variable

- Angle ϕ' decreases from the initial to the residual value. δ: Model Parameter



Resistance to sliding. Passive wedge



- Force equilibrium in vertical and horizontal directions:

$$N'_i = \frac{(W_s + W_c)\left(\tan\alpha_s + \tan\varphi'_m\right)}{1 - 2\tan\varphi'_m \tan\alpha_s - \tan^2\varphi'_m}$$

- Horizontal thrust: $(N_i')_{crit}$ = min (N_i') (on α_s) ➡ $(\alpha_s)_{critical} = 17.4^o$

- "In situ" observations : $\alpha_s = 20^o$

- Passive resistance: $R_f = N_i' + U_i$

Model parameters

(*): Very small influence on results

Blue clay

- $\phi'_{initial}$: Mean friction angle on the failure surface. It is around 18º. (*)
- ϕ'_{res}: Residual friction angle: Varies between 10º and 12º.
- δ: Necessary distance to mobilize the residual friction angle. Relative displacements of several decimetres are required. (*)

Tailings

- ε: Displacement to get liquefied tailings (1 m) (*)
- γ_e: Natural specific weight of liquefied tailings. (31 kN/m3).
- F_{hi}: Initial horizontal thrust mobilized against the dyke (and the upper slice of clay). It is estimated at 14000 kN/m if K_0 =0.5, and at 11000 kN/m if active conditions prevail. (*)

Model parameters

Geometry

- β_{st}: Dip of the upstream scar within the tailings deposit (70º à 90º)
- e_R: Depth of failure surface under the center of the dyke, (14-15 m)
- α_b: Apparent slope of failure surface (2º)
- aa_{max}: Maximum height of soil accumulated over passive wedge (12 m)
- α_s: Exit angle of failure surface (20º)

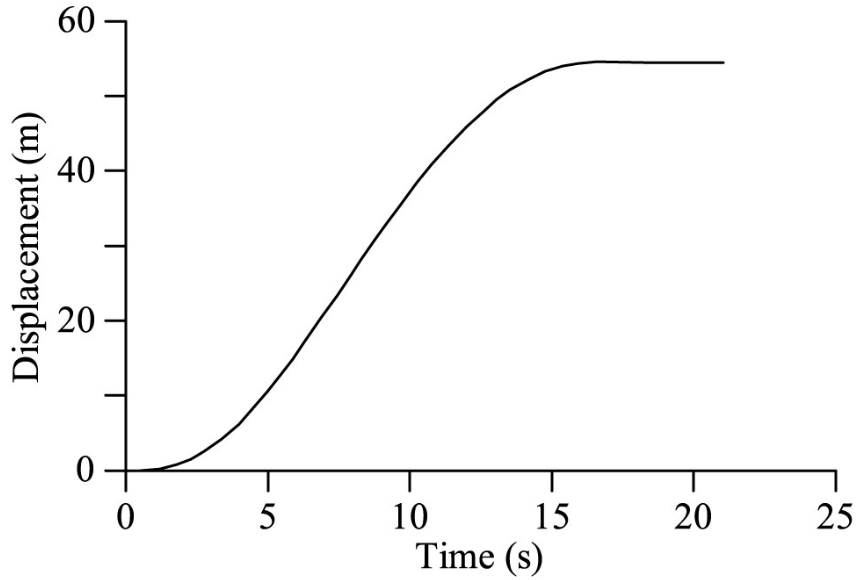
Numerical

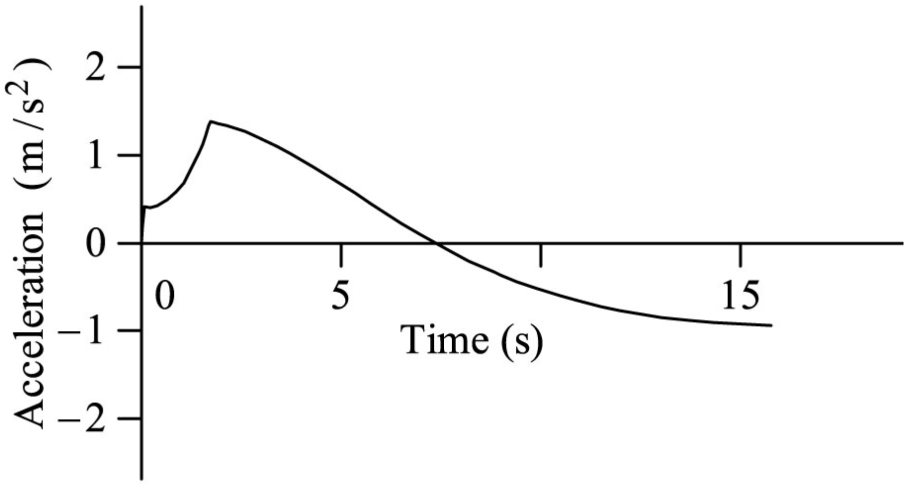
- Δt: Time increment for iterative calculation. Negligible error if $\Delta t < 0.1$ s

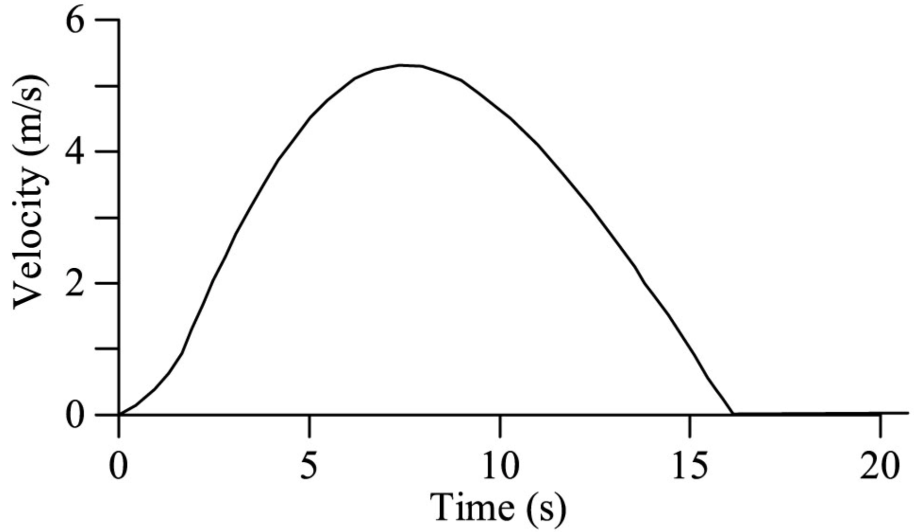
Results

- Time duration of motion: 14.5 s
- Maximum speed: 5.5 m/s (20km/h)
- Maximum acceleration: 0.14g

(Equivalent intensity: 7-8 MKS)

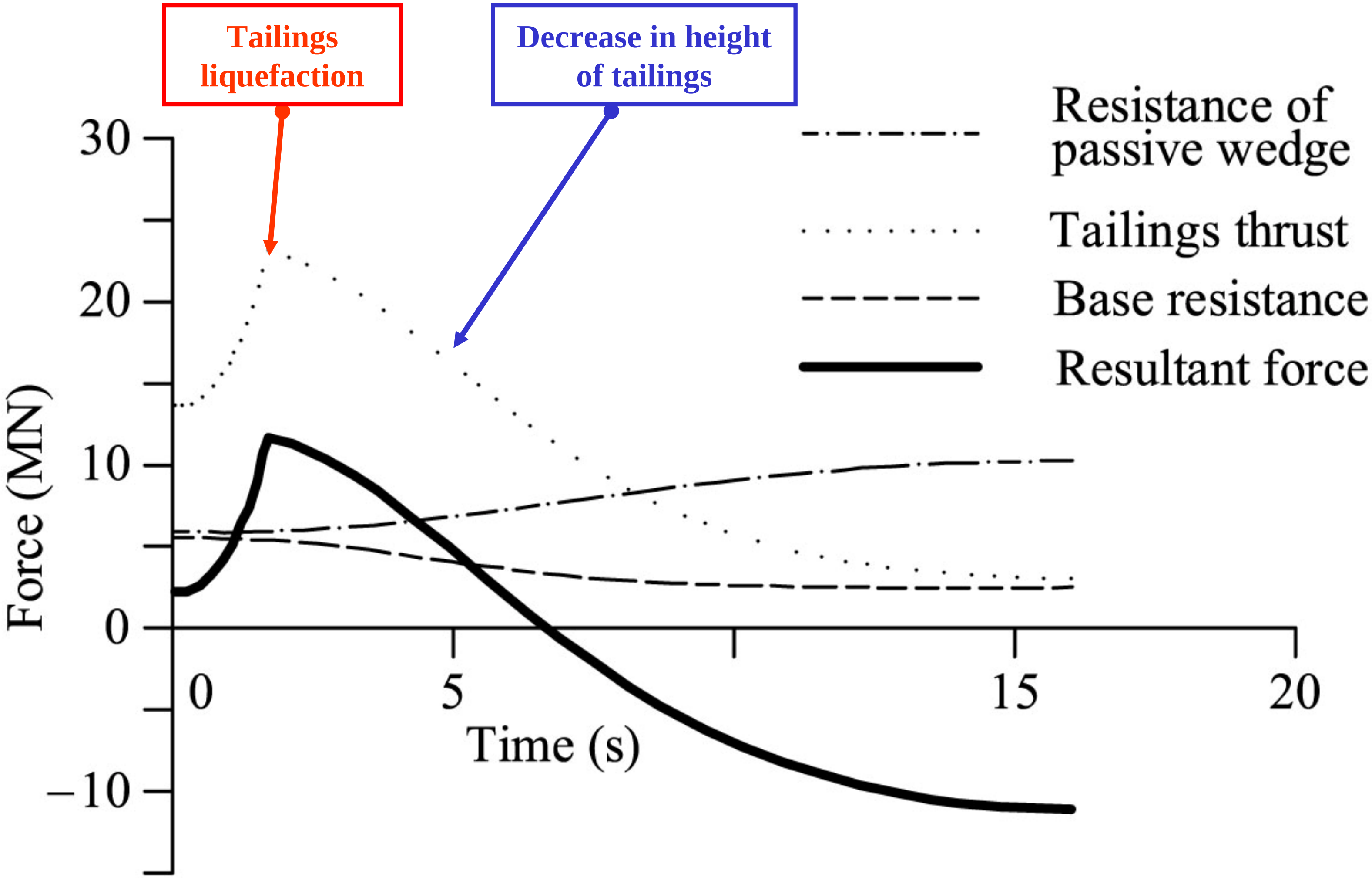






Results

- Summary of forces: Thrust; Base resistance; Passive wedge



Sensitivity analysis. Effect on total dyke displacement

- Slope of sedimentation planes

α_b	s_{max} (m)
2º	52
1º	47.8
0º	43.8
-1º	39.6

- Dip of upstream scar

β_{est}	s_{max} (m)
80º	48.4
90º	45

- Residual friction angle of clay

ϕ_{res}	s_{max} (m)
10º	57
12º	47

Sensitivity analysis. Effect on total dyke displacement

- Exit angle of passive wedge

α_s	s_{max} (m)
10º	41
15º	48
25º	53
30º	53

- Maximum height of accumulated soil on passive wedge

aa_{max} (m)	s_{max} (m)
8	55
12	49

Conclusions of dynamic analysis

- The ability of the model to reproduce the dyke displacement gives confidence to hypothesis made and parameters selected
- The motion was fast:
  - Total time: 15 sec
  - Maximum speed: 20 km/h
- Maximum acceleration (0.14 g) was fast attained. Instability and level reduction of liquefied tailings upstream
- The motion stopped because the height of liquefied tailings decreased. Passive wedge played a marginal role.