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Brief insights: In situ testing of base friction angle in an old concrete dam

Enrique Asanza-Izquierdo, **Carlos Laina-Gómez**, Diego Escudero-Merino, Juan Antonio Díez-Torres
Laboratorio de Geotecnia, Centro de Estudios y Experimentación de Obras Públicas (CEDEX), Spain,
carlos.laina@cedex.es

ABSTRACT: In large Spanish dams, a safety review must be carried out every five years. Because of the re-evaluation of the structural safety, in some cases uncertainties appear in some parameter in which the foundation was characterised during the project or in the evolution of the materials over the years. This is the case of a gravity dam in the province of Alicante, where we wanted to re-evaluate the shear strength parameters in the foundation plane. For this purpose, a large scale in situ shear test was considered. These tests are unusual because require considerably more preparation and means than sampling and testing in the laboratory, but they are a way of determining the parameters governing the sliding of a concrete structure on a rock plane while minimising the scale factor. The test was carried out downstream on an outcrop like the foundation plane. Four 1x1x0.6 m concrete specimens were made above a rock plane, and the shear strength was obtained for each block subjected to different normal loads. In this way it was possible to determine the shear strength parameters of a concrete-rock contact. In this paper, the test set-up, the test procedure and the results obtained are described.

KEYWORDS: in situ test, shear strength, large scale, rock plane.

1 INTRODUCTION

1.1 *General scope of the Spanish dam*

Proper management of water resources provides a wide range of benefits to countries, irrespective of its degree of development, being reservoirs and dams key elements of any national hydraulic network. There are approximately 1300 dams in Spain, with as many as 500 (~38%) of them exceeding 50 years of age, with an average age of 45 years. Regarding the type, approximately 65 % of them are gravity dams. Most of the Spanish dams are State-owned and operated by nine National River Basin Authorities (Confederaciones Hidrográficas, CH) reporting to the Department for Environment. Thus, maintenance in Spain is an issue of major concern of the National Administration that relies on codes of inspection, monitoring and surveillance. The Centro de Estudios y Experimentación de Obras Públicas (CEDEX), a technical agency sponsored by both the Department for Transportation and the Department for Environment, provides institutional support and promotes R&D in civil engineering, and particularly in the scope of dams and water management.

The so-called “Normas Técnicas de Seguridad de Presas, RD 264/21” in Spain, that came into force in 2021, consolidated two previous national guidelines and codes. Such recent regulation enforces comprehensive inspections at specific year intervals of every dam, regardless of its ownership or use.

1.2 *Sources of uncertainties linked to sliding of gravity dams*

The shear strength parameters at the base of gravity dams, either along a discontinuity plane within the rock mass or at the rock-concrete interface, are the main source of uncertainty; in some cases, it may have not received due attention. Moreover, the critical sliding plane might have even undergone degradation, losing cementitious bonds or developing tensile cracks further inwards from the upstream toe.

It is well known, on the other hand, that decades ago water infrastructure used to be designed for climate conditions whose recurrence at present seems to depart from patterns assumed in the past, not only in semiarid countries like Spain; total rainfall events tend to decrease in volume (droughts) while increase in intensity, with a trend of reduction in the flood return period. Although hydrological failure is, by far, more severe in earth dams, the discharge flow rate of spillways of gravity dams needs re-evaluation and, subsequently, the effect of the increased hydrostatic thrust and uplift forces on the modes of failure, especially sliding.

1.3 *Background and purpose*

CEDEX was commissioned by the River Basin Authority (CH del Júcar) to carry out desk studies with an updated hydrological database from the Amadorio River Basin whereby the inferred flow rate at several return periods could be checked against the maximum discharge capacity of the spillway of Amadorio dam, a gravity dam close to the river mouth (near Benidorm, Mediterranean Sea). Such studies concluded that the dam crest should be raised 4 m, so that the spillway can discharge the 10000-year inflow design flood without exceeding the minimum free board. During such extreme events, due to the unfavourable increase of the hydrostatic and uplift forces (gravity forces remain unaltered), the safety factor against sliding drops. In this context, the geotechnical concern on the choice and reliability of the strength parameters inevitably arises.

This paper addresses the practice and experience gained in Spain over decades on large-scale in situ shear tests at the rock/concrete interface of dams, and how it was advantageously put into practice for the reassessment of the strength parameters regarding the sliding stability of Amadorio Dam.

2 LARGE-SCALE IN SITU SHEAR TESTS

2.1 *Principles of the direct shear tests*

Direct shear tests are aimed at measuring the mobilised shear strength (τ_{mob}) at the interface of two bodies in contact, due to both frictional forces (being "φ " the angle of friction), which is proportional to the normal stress (σ_n) , and adhesion forces or effective cohesion "(c')", following the Coulomb failure criteria:

$$\tau_{mob} = c' + \sigma_n \cdot \tan\varphi \qquad (1)$$

When dealing with particulate material (soils), which is not the case, samples are placed in a box, horizontally split into two halves. The standard laboratory direct shear apparatus has a 20mm-thick box of internal dimensions 60×60mm. Larger boxes are uncommon and costly and, therefore, they can only be found in certain research centres. The Laboratorio de Geotecnia (LG-CEDEX) has two shear apparatuses with boxes 15x30x30cm in size, suitable for coarse granular soils; LG-

CEDEX holds a much larger box as well, 120x100x100cm in size (Figure 1), which has been intensively used to test the shear strength of rockfills for dams and other earthen structures (Muñiz-Menéndez and Estaire, 2021).

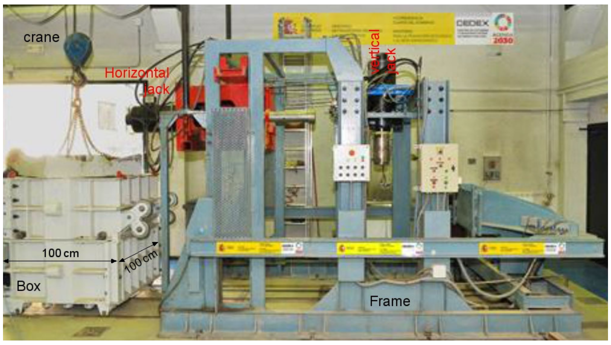


Figure 1. Large box (120x100x100 cm) at Laboratorio de Geotecnia (CEDEX).

Once a vertical stress is applied onto a distribution plate resting on the sample, the test is carried out by shearing the two box halves, relative to each other, by means of a motor-driven ram acting upon the bottom half at a programmed constant displacement rate. The opposing shear force that gradually develops within the tested plane is measured throughout the test; vertical displacement is likewise measured, as it accounts for dilation, which yields extra strength due to grain interlocking or large-scale roughness in the case of rock discontinuities (saw-toothed model (Patton, 1966), see Figure 2).

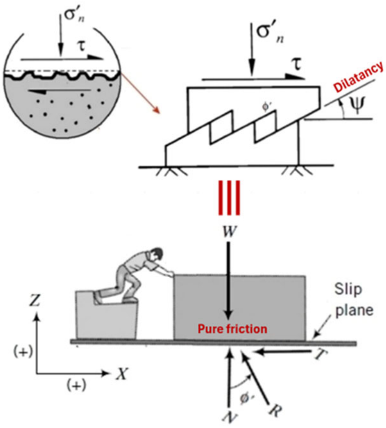


Figure 2. Dilatancy envisaged as a saw-toothed model (modified from (Budhu, 2010).

2.2 *Difficulties facing large-scale in situ shear tests*

Distinction should be made when comparing standard laboratory and large-scale in situ shear tests, with samples usually between 0,5 and 1m. Despite their likeness, the high level of forces applied, the space that it takes up and the budget constraints of the latter prevent proper compliance with a laboratory standard shear test. Four basic requirements are exceedingly hard to attain when planning a large-scale in situ shear test, so it must be born in mind at the engineer's desk.

1. Representativeness: the location of the test is of utmost importance, as it has to fulfil accessibility and representativity of the rock mass: Two basic location are the choices: inside one of the dam galleries, free of lining; or outside, on an outcrop of rock of the same nature and conditions, downstream in the vicinity of the dam (Figure 3).

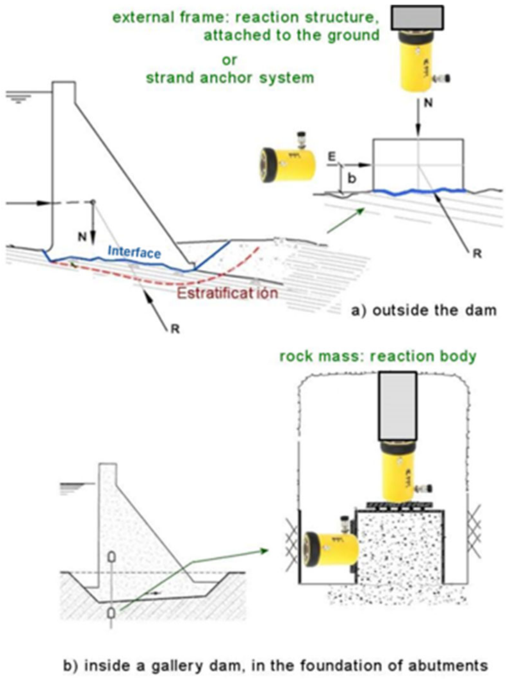


Figure 3. The two basic location of a large-scale shear tests.

2. Running the shearing force at a constant displacement rate; such testing mode allows a controlled post-peak failure and, therefore, detailed information throughout the test. Explanations of the advantages of failure tests at constant strain rate can be found in (Serrano, 1998) and (Hudson et al., 1972), so, whenever possible, it is the preferable mode in the laboratory. However, when handling large loads (from hundreds to thousands of kilonewtons) constant strain rate turns unaffordable in the field (setting up high-performance servo-controlled motor would be needed); a sequence of load increments by a hand-operated hydraulic pump that supplies pressure to a hydraulic ram is used instead, although the control is partially lost. This procedure is accepted in suggested methods and standards (ASTM, 2002, ISRM, 2007). At the peak resistance threshold, however, as the hydraulic pump cannot restore right away the loss pressure caused by sudden displacements, it results in a gap of relevant information. In fact, this is what happened in the tests shown here. However, the loss of information has been minimized by increasing the sampling frequency, yet at the expense of processing sizeable log files.
3. Maintaining the vertical load strictly constant during the shear. Unlike standard shear tests apparatuses, which apply them by means of dead weights on a hanger attached to a lever, large-scale in situ test designs often resort as well to a hydraulic ram, fitted to an external rigid reaction body: rock mass in a gallery or a reaction frame (Figure 3). However, such rigidity prevents the free and (expected) vertical movements associated to dilation, unless the hydraulic ram releases the excess pressure on the go, so as to counterbalance the subsequent increase of vertical load over the target value. Such operations seem unpractical with hand-operated hydraulic pump; for this reason, alternatives with strand anchor systems (Figure 4), due to their high flexibility (compared with bar anchors) have been successfully used in Spain. Some drawbacks, however, have be argued on the grounds of occupational hazards, stress concentration and costs when no anchoring machinery is at hand.

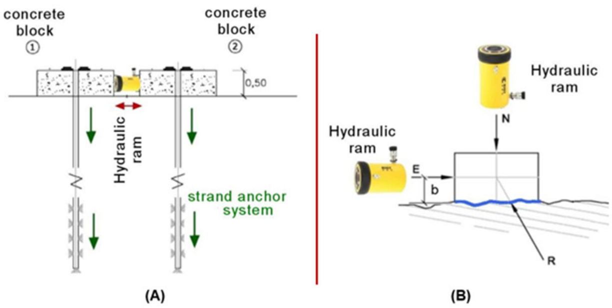
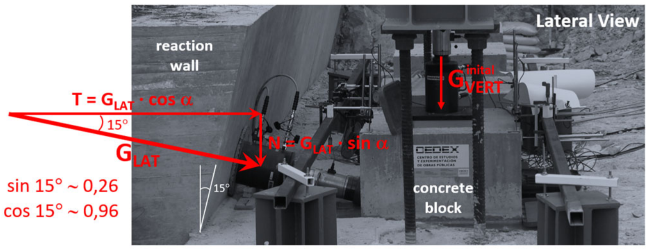


Figure 4. Alternatives for applying vertical load.

4. Keeping the shear force coplanar to the shearing plane, which is solved with standard laboratory shear tests boxes. If the situ shear test is arranged such that the lateral hydraulic ram is placed horizontally, an overturning moment is unavoidable and might distort the results (Soriano, 1997). Recommended methods and guides suggest (ISRM, 2007, Brady and Brown, 2004) tilting the lateral shearing load slightly (α ~ 15°) with the resultant line passing through the centre of the sheared plane, so that the load is evenly distributed at the base of the block. Such restriction certainly complicates the setting-up completely, especially when it is carried out outside the dam, as a reaction wall with overhanging face is needed. Leaving aside by now the increase of vertical load when using a rigid reaction body, a tilted lateral load (GLAT) itself entails an increase of the initial vertical load as the test progresses, hence the correction of the true shear load and the total vertical load (Figure 5):



$$N_{total} = G_{VERT}^{inital} + G_{LAT} \cdot \sin \alpha$$

Figure 5. Lateral shearing force (tilted 15º): decomposition into horizontal and vertical forces. Setting-up: tilted lateral force.

Figure 5 and Figure 6 shows the setting-up of the large-scale in situ shear test at Amadorio dam, which follows such arrangement.

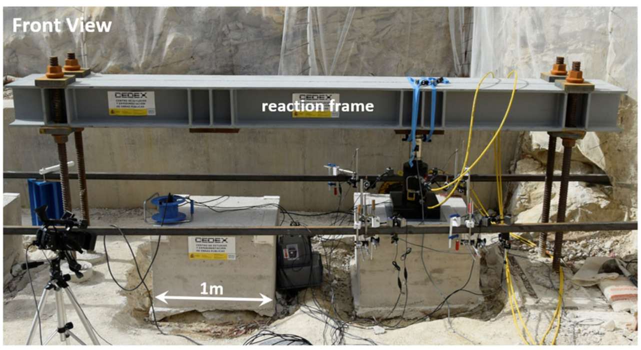


Figure 6. Large-scale shear test downstream Amadorio dam. Setting-up: rigid frame.

3 AN OVERVIEW OF THE SPANISH EXPERIENCE

In Spain, one of the two basic arrangements depicted in Figure 4 are generally followed in the scope of dam engineering. According to the authors´ experience gained, the advantages and disadvantages of both arrangements are listed in Table 1.

Table 1. In situ shear test in Spain (outside the dam). Advantages and disadvantages.

	Advantages	Disadvantages
With External rigid frame	Stresses evenly distributed Free of overturning moments	Complex Costly Vertical load varies during the test
With strand anchors	Due to the anchor flexibility, vertical load nearly constant Useful if bearing capacity is to be assessed; Relatively inexpensive Simple	Anchor head, require distribution load Overturning moments Vertical load prefixed, not adjustable during the test

Practice in Spain concerning the choice of the strength parameters at the concrete/rock interface has mostly relied on expertise judgement of the excavated fresh rock and inspection pits by comparable experience. Note that laboratory tests on rock cores fall short of the determination of strength parameters. As a result, in situ large-scale shear tests have been successfully carried out as well for a number of concrete dams.

4 THE LARGE-SCALE IN SITU TEST AT AMADORIO DAM

4.1 *Main features of the dam and purpose*

The Amadorio dam, constructed between 1948 and 1960, is located in Villajoyosa (Alicante), on the course of the Amadorio River, a tributary of Júcar River, with a reservoir of around 16 hm³. It is a gravity dam with a straight profile, approximately 65 meters high from its foundations and a crest length of about 335 meters. Due to the scarcity at that time (Spanish post-war period) it was built with lean concrete (UCS~10 MPa) poorly mixed. Figure 7 shows a cross section of the dam.

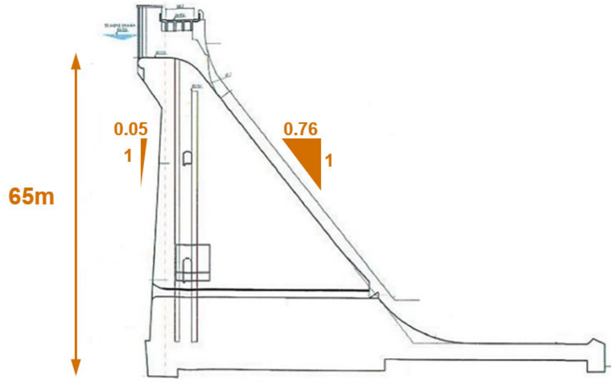


Figure 7. Cross section of Amadorio dam.

The dam foundation consists of alternated layers of limestone, marly limestones and calcarenites with thickness ranging from centimetres to decimetres. The average dip there is 20° downstream. In general, according to the scarce information available, the rock mass is not weathered, especially on the right bank. Hardly any other information can be found concerning the foundation condition prior to pouring the concrete.

As a result of the recent safety inspections, the issue of the proper choice of the strength parameters at the dam foundation was raised, as some disagreement had been found in the three inspection reports at hand. Table 2 shows the strength parameters (values unaffected by reduction factors) proposed by the three reports:

Table 2. Strength parameters suggested by the three reports.

Report	Angle of friction (º)	Cohesion (kPa)	Remarks
Consultant A (2002)	39	500	
Consultant B (2018)	34	250	
Consultant C (2021)	36,5	350	(1)

(1) Recommended a large-scale shear test; C, previously engaged in such tests for other dams

4.2 *Basic arrangement of the test*

The test area was located downstream the dam (Figure 8), on the right bank where representative massive marly limestone outcrops at the base of the slope (Figure 9). Minor excavation work and ground levelling was needed prior to prepare the test area, where an array of four cast-in-place concrete reinforced blocks 1x1x0,6m in size were programmed to be tested.

Figure 8. Location of the test, downstream the dam. The blue rectangle is a canopy for shading.

(A) (B)

Figure 9. (A) Another view of the test site before executing the civil works. (B) Outcrop of the rock at the contact plane of one of the blocks.

The test requires two reaction structures (Figure 5 and Figure 6):

- a passive reinforced single-face (workform overhanging 15º) cast-in-place concrete wall, 4 m long and 1,40 m high, densely reinforced; the opposite side was an open rock excavation, previously examined, as all the passive thrust relies on it;
- and a reaction frame: consisted of 2 twin welded steel HEB-340 beams 5 m long, spanning two concrete blocks; the twin beams are pinned at their ends on four threaded steel bars (Ø63,5 mm St670/800) with flat bearing plates and a nut on each side. The twin beams are detachable and movable to cover the span of the other two concrete blocks with another pair of bars. All six bars, which absorb the uplift force of the vertical hydraulic ram, are grouted 8 m into the rock mass. Figure 10 shows the design drawings of the reaction frame and the cast-in-place wall.

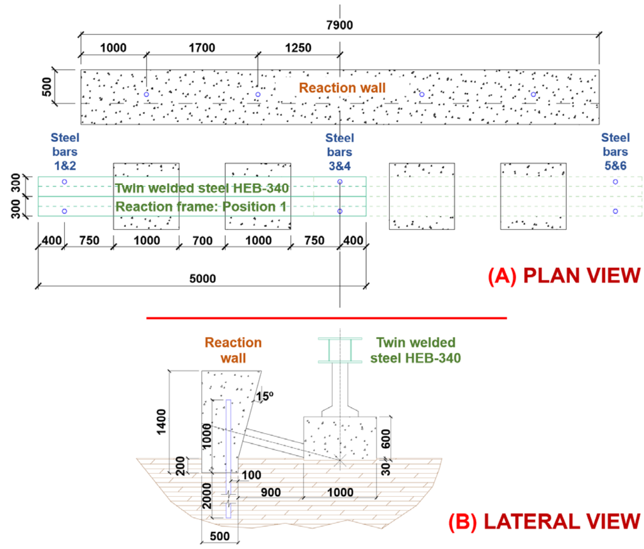


Figure 10. Design drawings of the reaction frame and the concrete wall: (A) Plan view, (B) Lateral View.

4.3 *Load and monitoring system*

The increments of lateral load were applied with a hydraulic ram assisted by a hand-operated hydraulic pump with a capacity of 2,680 kN, whereas the vertical load, with another one of 1,680 kN. Reaction pads and plates to avoid concentrated loads were used. In addition, special efforts were made to create a smooth surface between the top face of the block and the load distribution plate, as the lack of vertical alignment might damage the hydraulic ram; on the top of the block, it proved to work successfully with the placement of a thin aluminium plate, meant to absorb lack of flatness and grain indentation, under a "sandwich" of two thick distribution plates with viscous grease in between. Nonetheless, both hydraulic rams are equipped with a ball-and-socket joint to allow for minor adjustment.

Although during the test the loads can be obtained by the pressures displayed at both pump gauges, electronic load cells were attached between the ram and the distribution plates and connected to the data acquisition system. With regard to the displacement readings, six regular dial gauges (0,01 mm accuracy; stroke 100 mm) were installed at the tested block: four vertically, at each edge of its top face (dilatancy and tilting); and two horizontally, at the front face near its top corners, to obtain an averaged displacement. All six dial gauges were paired to webcams which, in turn, were connected to a closed-circuit television (CCTV) and recorded (Figure 11). Load values and dial gauges readings were noted down at intervals throughout the test as back-up. All six webcams, displayed altogether on a screen provide useful information to track the test (Figure 12) and spare operators from any hazards, as reading are taken off the testing site. In addition, another set of six LVDT were as well installed next to each sister dial gauges and connected to the data acquisition system with a high reading frequency, so as not to miss out uncontrolled brittle failures. The tip of the LVDTs and dial gauges plungers rest on pieces of china tiles glued to the block on avoidance of surface roughness.

4.4 *Commented test procedure*

The vertical loads applied to the four concrete blocks should cover the range of average effective stress at the base of a 65 m dam at its worst scenarios. Although the desired target vertical loads were aimed at 250, 400, 550 y 700 kN (effective stress: ~250,400, 500 and 700 kPa), the true vertical load at failure can be neither known nor closely narrowed down in advance: on one hand, the inclined lateral load contributes unavoidably with a vertical component (see Figure 5); and, on the other, the frame itself causes unpredictable extra vertical loads once the adhesive/cohesive forces have been broken after minor shear displacements, since a minimal uplift movement along its saw-toothed surface is partly frustrated in proportion to the frame stiffness.

Figure 11. Arrangement of the monitoring system

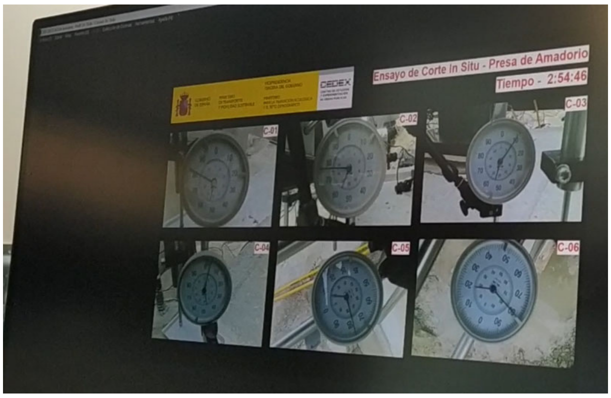


Figure 12. All six dial gauges displayed on a PC screen at a safety distance from the test.

In fact, it seems hard to set clearly apart cohesive and frictional forces. Even so, basic rationale suggests that, as mass concrete yields at 0,2 % approximately, the cohesive/adhesive forces developed by the concrete at the calcareous rock surface might well reach such range of distortion. Hence, after 1% of relative horizontal distortion (10mm, along a 1m-long stiff concrete block) such forces should have vanished completely, being the remaining forces at the interface purely of frictional nature. In this regard, the tests were intended to reach at least a horizontal displacement of 10 mm and, if possible, 25 mm, which eventually was accomplished. Apparently, the friction angle, due to a (somehow) grinding effect, decays towards a "residual" value. Altarejos-García (2009) made insightful research in this regard, which is in good agreement with the previous reasoning and the results of the present tests, as shown later.

5 RESULTS AND ANALYSIS

5.1 *Results. Peak values*

Table 3 summarises basic conditions and results of the four tests, while Figure 13 (left) plots the total force paths (vertical vs horizontal) followed by the 4 tests beyond the peak failure, as well as the peak strength envelope (right). Nonetheless, such "peak" (and brittle as well) envelope, obtained through linear fit, overestimates the true strength parameters (it yields c'~1 MPa; ϕ_{peak}~55º) (see Figure 13 right), as the cementitious bonds at the dam base may have decayed or been broken by tensile cracks near the upstream toe. Besides, the block is made of concrete of a much higher quality than the old dam itself. So, the true adhesion/cohesion, if any, would have reached values far lower than 1 MPa.

As shown in Table 3, shear strains at peak strength ranges from 0,2 to 0,7 %, as previously predicted; thus, further displacements occur with total lack of adhesion/cohesion forces while dilative strength develops (see Figure 2). Test with Block 1, as shown in Figure 14 (vertical vs horizontal displacements), seems to follow such behaviour. In fact, the response of the other 3 blocks do not differ substantially from Figure 14. The dilation angle is obtained from the ratio vertical/horizontal.

Table 3. Summary of test characteristics and observations.

Test	Initial vertical Load (1) (kN)	Horizontal force at peak failure (kN)	Displacement at peak failure (mm)	Remarks
Block 1	25	~2,050	2	Brittle failure
Block 2	50	~1,760	4	Brittle failure
Block 3	25	~2,150	7	
Block 4	25	~1,940	5	Prior, test of rock mass deformability

(1) Self-weight of the concrete block included

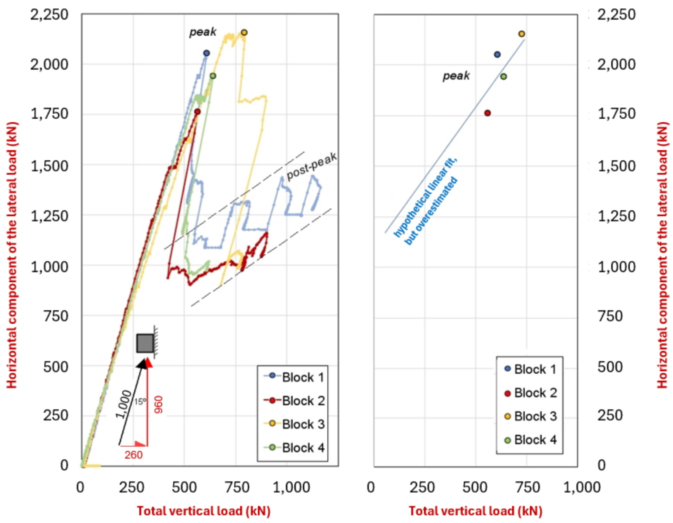


Figure 13. Load paths (vertical vs horizontal) followed by the 4 blocks and peak envelope.

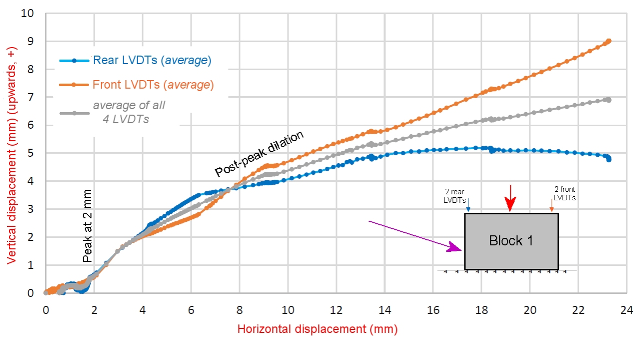


Figure 14. Relationship between vertical and horizontal displacements in the test with Block 1.

5.2 *Results. Post-peak strength*

The post-peak strength was analysed with: Taylor's model (Taylor, 1948), based on energy balance; the Barton-Bandis model (Barton and Bandis, 1990), which is more appropriate for natural discontinuities; and finally, the saw-toothed model (Figure 2), which seems the most suitable. Taylor's model is highly conceptual, and rather aimed at granular soils, whose dilative response is sensitive to density. Ultimately, the saw-toothed model can be reduced to the contribution of two additive angles:

$$\phi_{mov} = \psi + \phi \qquad (2)$$

where ψ is Dilation angle; ϕ, Friction true (or "basic" defined by Barton/Bandis) angle; and ϕ_{mov}, Mobilised angle.

Theoretically, the true angle should not vary as the shear continues, whereas the dilation angle should decay due to the grinding effect, unless the rock matrix is hard enough. Figure 15 shows the evolution of the three angles throughout the test with block 1. The mobilised angle and the dilation angle are obtained with the ratios shown in Figure 15 and, therefore, the (true) friction angle is readily inferred as a subtraction. Degraded angles could be roughly derived if large shear displacements have been achieved (>20 mm).

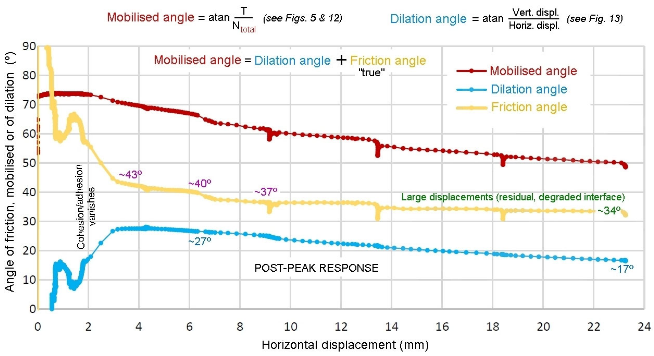


Figure 15. Relationship between angle of friction, mobilised, of dilation and horizontal displacements in the test with Block 1.

Table 4 summarises the angles obtained with the four blocks. The true or basic friction angle, as seen as a "plain" angle, free of any geometrical contribution, is a good estimation for design purposes in the safe side. It seems sensible, due to uncertainties, to neglect cohesive/adhesive force. Undoubtedly, irregularities during the foundation clearing and certain cohesive forces are likely to contribute to the sliding stability, yet only a rough guess can be attempted.

Table 4. Angles (º) obtained from the large-scale shear test.

Test	ϕ	$\phi_{degraded}$	ψ	$\psi_{degraded}$
Block 1	40-43	34	27	17
Block 2	38-48 (1)	34	28	19
Block 3	38	32	29	22
Block 4	45	<35 (2)	23	(2)

(1) Vague results; (2) limited displacement;

6 CONCLUSIONS

Practice in Spain concerning the choice of the strength parameters at the concrete/rock interface has mostly relied on expertise judgement of the excavated fresh rock and inspection pits by comparable experience, yet for decades large-scale in situ test have been successfully carried out to narrow down uncertainties. Details during the setting-up is crucial, as discussed in the paper.

7 ACKNOWLEDGEMENTS

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