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Experimental and Numerical Analysis of Out-of-Plane Behaviour of Dry-Joint Stone Historical Walls

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Abstract. Numerous dry-joint stone walls worldwide represent important historical and cultural heritage. These walls are particularly susceptible to out-of-plane loads, such as those due to seismic activity, which can lead to overturning failure. One way to evaluate these structures is to use numerical models, which must be validated with experimental data to ensure accuracy. For this reason, this research introduces a novel approach to evaluate the structural response of dry-joint stone construction, which uses rigid body numerical modelling, validated through experimental analysis of replicas that closely reproduce the properties of heritage structures. For this purpose, a reproduction of a dry-jointed stone wall of the Coricancha Temple in Cusco, Peru, was tested in order to validate the numerical models of the structure based on the finite element method and rigid-body mechanics. Three 1:2-scale concrete block walls were constructed on a tilting table. These walls were built with characteristics very similar to those of the original heritage structure and then tested under an out-of-plane overturning setup. The experimental tests provided key data, including pre-collapse drifts, collapse mechanisms, and collapse angles. The numerical simulations accurately predicted the collapse angles, replicated the experimentally observed failure patterns, and reproduced the average drift at various inclination angles. The close agreement between the numerical and experimental data validates the numerical models, demonstrating their accuracy in simulating the structural response of dry-joint stone historical walls.

Keywords: Out-of-plane, modelling, rigid body, block wall, collapse mechanism, Coricancha.

1. Introduction

Several heritage stone structures have endured to the present day globally. Many of these are dry-joint constructions that achieve stability solely through their weight (Costa et al. 2024) and friction between their stones. Majestic examples of such dry-joint stone structures include Angkor Wat in Cambodia, Asia (*Fig. 1a*); Great Zimbabwe in Zimbabwe, Africa (*Fig. 1b*); Machu Picchu in Peru, America (*Fig. 1c*); and the Segovia Aqueduct in Spain, Europe (*Fig. 1d*). Despite their impressive resistance to forces within their plane, they remain vulnerable to perpendicular loads, notably those generated by seismic activity (Ferreira et al. 2015) or foundation settlement.



(a)



(b)



Fig. 1 Dry-joint stone structures in the world: (a) Angkor Wat, Cambodia, (b) Great Zimbabwe, Zimbabwe, (c) Machupicchu, Peru and (d) Segovia, Spain

Source: Google Maps

An in-depth understanding of the structural behaviour of these constructions is crucial for effective intervention. However, this is notably challenging for heritage dry-joint structures due to their discontinuous nature and, more significantly, the nonlinear response of their inter-block joints (Smoljanović et al. 2018). Furthermore, the cultural significance of these structures generally precludes the use of invasive evaluation methods, such as destructive testing or sampling of representative materials (Macchi 1992). To address this problem, some authors have conducted non-invasive tests directly on the structures by using a variety of methods, including microtremors (Cuadra 2021, Özcan 2025), roughness measurements (Cuadra 2021), water leakage measurements (Fujii et al. 2018), sonic tests (Martini et al. 2023), and X-ray tests (Bevins et al. 2022; Bevins et al. 2024), among others. Consequently, other researchers simulate the behaviour of dry-joint stone structures using full- or reduced-scale experimental models designed to closely replicate the originals (Foti et al. 2018; Savalle et al. 2019; Santa-Cruz et al. 2021; Xu et al. 2023; Ita et al. 2023). While some models use similar natural stones (Santa-Cruz et al. 2021; Restrepo Velez et al. 2014; Nikolić et al. 2017; Ita et al. 2024), others employ alternative materials, such as clay (Bui et al. 2017), sugar (Bui et al. 2017), chalk (Foti et al. 2018), and concrete (Jaimes 2024), to simulate the stones. Among these cases, there is very little evaluation by authors regarding the vulnerability of dry-stone heritage (Naik and Menon 2022; Giardina et al. 2020; Manos et al. 2019; Gupta et al. 2025). A key challenge is that the specific physical properties of the blocks, including their geometry, density, and finish, are often not fully accounted for. The immense quantity and variety of these heritage structures make physical replication both costly and challenging, leading to a significant shortage of experimental tests.

Otherwise, dry-joint structures can be analysed for failure mechanisms using micro- and macro-modelling approaches for static or dynamic analysis (Portioli et al. 2023). In the macro-modelling approach, a dry-joint structure is treated as an equivalent homogeneous material (Cuadros-Rojas et al. 2024). Conversely, the micro-modelling approach is more detailed and accurate, though it demands significantly more computational time. The micro-modelling approach is performed using the Finite Element Method (FEM) or the Discrete Element Method (DEM). In FEM, discontinuities are represented by contact laws (Suárez 2021; Lipa et al. 2024). Ashlars are typically modelled as elastic or rigid materials, with contacts treated as inelastic interfaces governed by laws such as Coulomb's or Hertz's (Ferreira et al. 2015). Conversely, DEM analysis (Fukumoto et al. 2014; Santa-Cruz et al. 2021; Ita et al. 2023; Ita et al. 2024) idealises the structure as inherently discontinuous, in which joint behaviour (normal, tangential, and rotational) is defined by contact laws (Horabik and Molenda 2016). Integrating these two methods is also employed for structural assessment (Munjiza 2004; Smoljanović et al. 2018; Chen et al. 2023). Although numerical methods show great promise for simulating the behaviour of dry-joint block structures, they require validation through experimental testing to yield reliable results. The need for experimental data remains a significant challenge, primarily due to the scarcity of tests on these structures.

The evaluation of dry-joint stone in historical walls can be performed more simply and economically using concrete blocks, as they exhibit properties similar to those of natural stone. As shown in Table 1, the density and friction angle of various stones align closely with those of normal-weight concrete, which typically features a density of 21600–23500 N/m³ (Wilson et al., 2021) and a friction angle between 29.8° and 41.7° (Pul et al., 2017). By carefully controlling the concrete mix design, formwork geometry, and finish, it is

possible to replicate these properties. This technique produces full-scale or scaled-down stone replicas, avoiding the high costs associated with traditional stone carving. Through this approach, researchers can estimate the mechanical behaviour of dry-joint stone structures and validate numerical models to assess their performance under other load conditions.

Table 1 Density and friction angle of different stones
(National Research Council 1930; Barton and Choubey 1977)

Stone	Density (N/m ³)	Friction angle (°)
Granite	24525 – 26487	29 - 35
Andesite	21582 - 24525	26 - 35
Basalt	26487 - 30411	31 - 38
Sandstone	19620 - 25506	25 - 35
Limestone	17658 - 26487	27 - 37
Shale	19620 - 24525	27

This work aims to evaluate the out-of-plane structural behaviour of dry-joint historical stone walls using numerical models validated by experimental tests. The experiments were conducted on concrete-block replicas of the walls, specifically designed to replicate the original stone's characteristics. For this purpose, the study focused on a dry-joint Inca wall from the Coricancha Temple in Cusco, Peru. Three concrete block replicas of this wall were built and tested under an out-of-plane overturning setup. Simultaneously, numerical models were developed using the Finite Element Method (FEM), rigid bodies, and contact laws. These models were subsequently validated using data from the experimental pseudo-static tilting tests. The validation process involved comparing the relative block displacements, failure patterns, and collapse angles between the experimental and numerical models.

2. Construction of the specimens

The most prominent dry-joint stone constructions in Peru are commonly associated with the Inca culture (1440–1532; Canziani 2012) and are notable for their precise carving, which allows perfectly fitted blocks. Several Inca temples built with this stone assembly method are shown in *Fig. 2*.

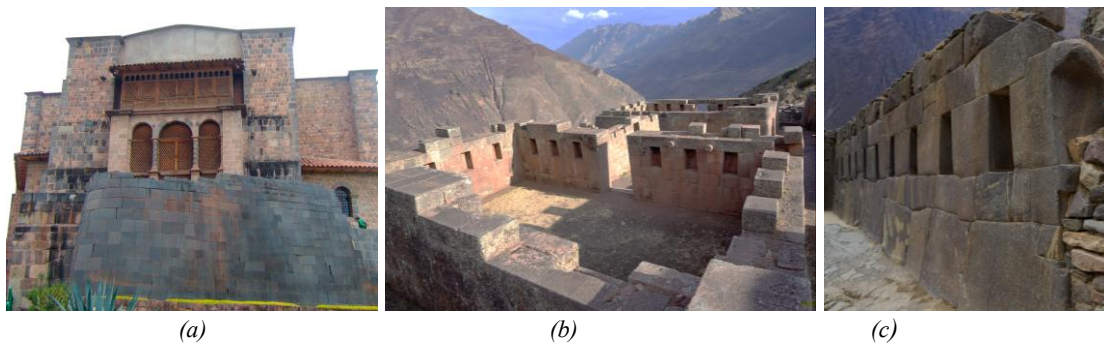


Fig. 2 Inca stone structures at various archaeological sites in Peru:
(a) Coricancha, (b) Pisac, and (c) Ollantaytambo

The Coricancha, also known as the Temple of the Sun, is situated in the heart of Cusco, Peru (Latitude: -13.52017, Longitude: -71.97521), and is widely regarded as the most significant temple of the Inca Empire (Castro 2020). During the colonial era, the site was modified to accommodate the construction of the Santo Domingo convent church (*Fig. 2a*). Despite these changes, several of the original Inca stone rooms and walls, distinguished by their exceptional craftsmanship, remain on site (*Fig. 3*).

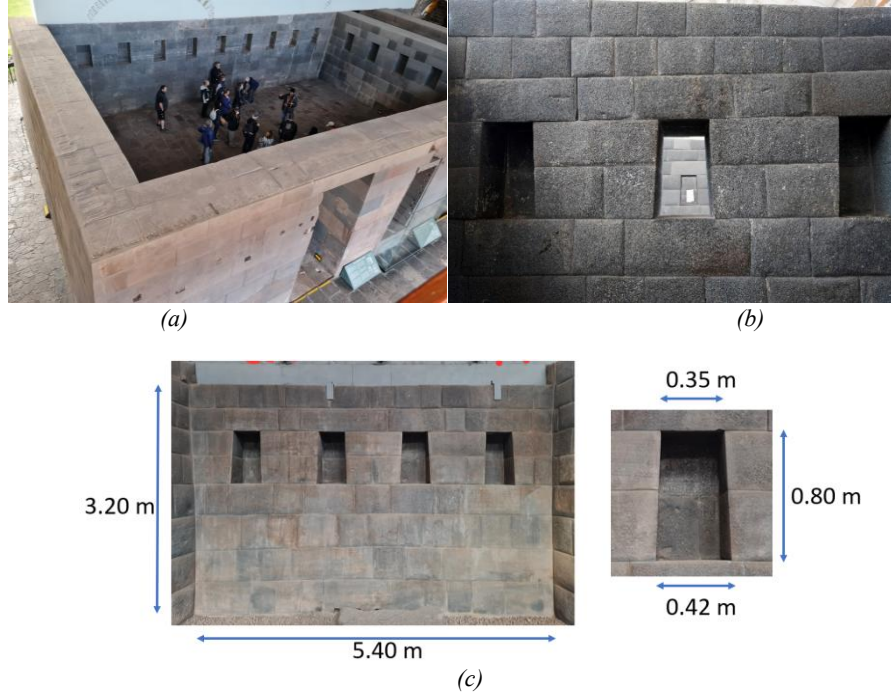


Fig. 3 Inca carved stone structures at the Coricancha Temple:
 (a) Top view, (b) Window and niches in one of the walls, and (c) Details of the selected Inca stone wall

2.1. Dry-joint historical wall evaluated

This research focuses on a wall from the Coricancha Temple as its subject of study, with dimensions and niches detailed in *Fig. 3c*. This two-leaf wall is constructed from dry-joint carved andesite stone of varying sizes (0.2 to 1.6 m) in a coursed ashlar pattern (Huamani and Yapura 2021; Lipa et al. 2025). With an estimated average thickness of 0.80 m, typical of adjacent walls, the block layout shows horizontal alignment but an irregular vertical stacking pattern. Additionally, individual blocks have irregular front faces (rectangular, trapezoidal, or parallelogram; Tovar 2023). Crucially, it remains unknown which blocks span both leaves of the wall, further complicating the creation of a fully geometrically representative experimental model.

To estimate the out-of-plane behaviour of this stone wall, concrete block models were built. Due to the lack of available data on the properties of the andesite stone from the Coricancha Temple, a study on two types of andesite from Inca structures at the archaeological site of Pisac in Cusco, Peru, was selected as a reference. This study indicated a density range of 2020 to 2600 kg/m³ (19816 to 25506 N/m³) and a friction angle range of 31.2 to 44.72° (Ancori 2023). The concrete blocks used in this research were designed to have characteristics similar to those of andesite from Pisac (section 2.3).

As the angles of the visible faces of the Coricancha site's stones range from 85° to 93° (Tovar 2023), different formwork for each concrete block would be required for their exact reproduction. For simplification, two simplified models were developed: Model A (assuming vertical joint continuity) and Model B (assuming no vertical joint continuity), with one wall constructed for Model A (wall A-1) and two for Model B (walls B-1 and B-2). Wall A-1 was built as a pilot wall to define the variables for the overturning tests.

The three walls were constructed sequentially on a monotonic tilting table at the San Ignacio de Loyola University (Lima, Peru). This table (*Fig. 4*) can support structures up to 4 m long and 1.5 m thick, with variable height. The tilting mechanism allows rotation up to 45° about its axis and supports a maximum load of 30 tons (Ita et al. 2023). Constructed from metal profiles and tilted by two 15-ton-capacity jacks, the table also incorporates a fixed metal bar to prevent the first course of the walls from sliding during overturning tests. Due to the table's dimensions, the walls were constructed at a 1:2 scale.

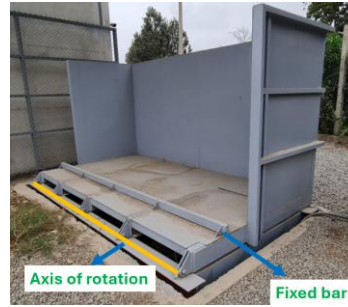


Fig. 4 Monotonic tilting table

2.2. Models A and B

Model A is a 1:2 scale representation of the evaluated wall (Total blocks: 101). This initial model measured 2.4 m in length, 1.6 m in height, and 0.4 m in thickness, and was designed as a two-leaf wall with regularly shaped blocks (*Fig. 5*). In model A, each block course was 0.20 m high, and each leaf was 0.20 m thick. The characteristics of the blocks illustrated in *Fig. 5* are listed in *Table 2*. Pink blocks have variable dimensions to match the niche dimensions shown in *Fig. 3c* at a 1:2 scale.

Notably, *Fig. 5a* and *Fig. 5b* show regions with running joints or vertical joint alignment. As this structural characteristic does not exist in the original wall and could introduce a failure plane, the model was subsequently modified to create Model B.

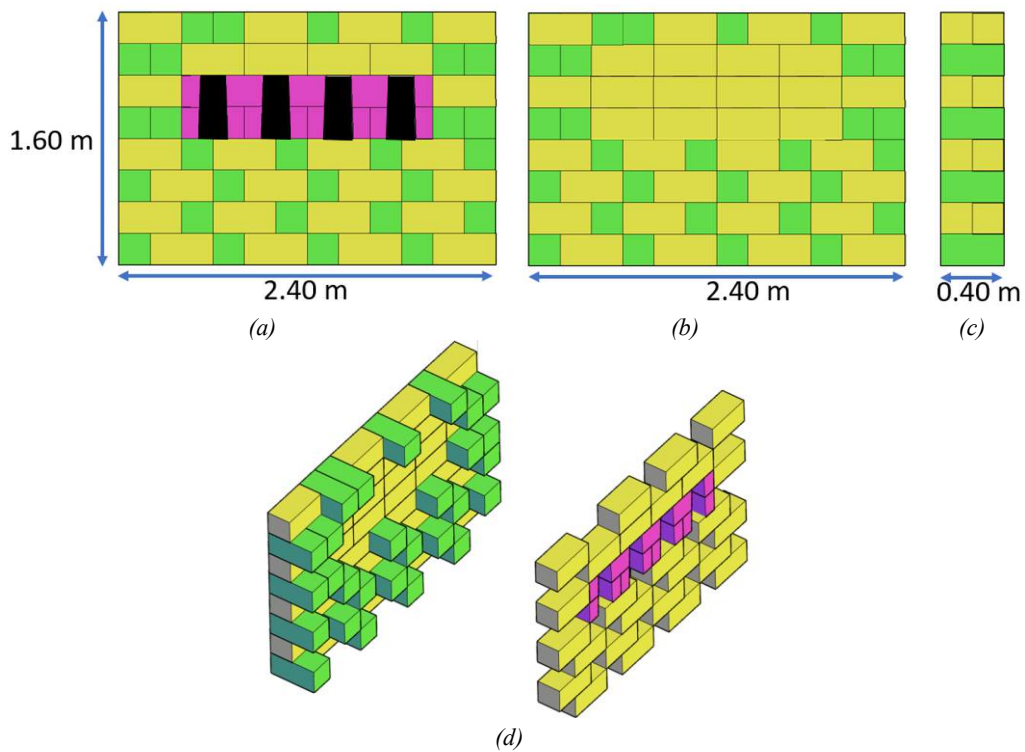
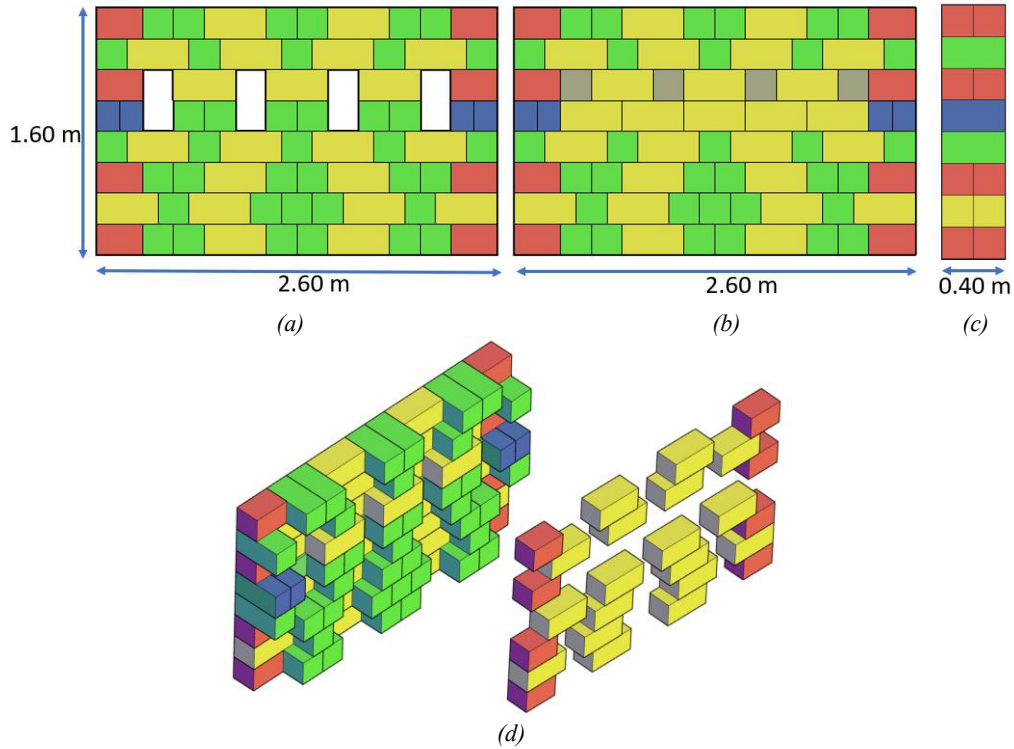


Fig. 5 Model A: (a) Front view, (b) Back view, (c) Side view, and (d) 3D view of the stone leaves

Table 2 Block characteristics

Block colour	Dimensions	¿Span both leaves (through stones)?	Quantity (Model A)	Quantity (Model B)
Green	0.20 m × 0.20 m × 0.40 m	Yes	28	39
Yellow	0.20 m × 0.20 m × 0.40 m	No	60	47
Pink	Variable	No	13	-
Red	0.20 m × 0.20 m × 0.30 m	No	-	16
Grey	0.20 m × 0.20 m × 0.20 m	No	-	4
Blue	0.15 m × 0.40 m × 0.20 m	Yes	-	4
Total			101	110

Fig. 6 illustrates Model B, without running joints, which was also constructed at a 1:2 scale to represent better the wall under evaluation (Total blocks: 110). The characteristics of the blocks illustrated in Fig. 6 are also listed in Table 2. In this new model, the niches are rectangular. Notably, this Model B was 0.20 m wider than Model A to represent the actual wall length better and avoid running joints. In fact, Model B dimensions were 2.6 m × 1.6 m × 0.4 m.

**Fig. 6** Model B: (a) Front view, (b) Back view, (c) Side view, and (d) 3D view of the leaves

2.3. Concrete blocks

Concrete blocks were made from local materials in Lima, Peru. The properties of the cement and aggregates were determined through experimental tests conducted in accordance with ASTM standards. Table 3 presents these results along with the specific ASTM standards employed. Type I cement was used, in accordance with ASTM C150 (2024). Additionally, a plasticising admixture was incorporated to enhance the flowability of the fresh concrete mix and to achieve smoother block faces, enabling the blocks to fit together effectively.

Table 3 Properties of the cement and aggregates for the construction of the blocks

Material	Property	Result	Standard
Cement	Density	29921 N/m ³	ASTM C188 (2023)
Coarse aggregate	Nominal maximum size	1"	ASTM C33 (2024a)
	Size number	5	ASTM C33 (2024a)
	Absorption	0.67%	ASTM C127 (2024)
	Total evaporable moisture content	0.32%	ASTM C566 (2019)
	Specific gravity	26585 N/m ³	ASTM C127 (2024)
	Bulk density (loose condition)	14421 N/m ³	ASTM C29 (2023)
	Bulk density (compacted condition)	15598 N/m ³	ASTM C29 (2023)
Fine aggregate	Nominal maximum size	#4	ASTM C33 (2024a)
	Absorption	0.98%	ASTM C128 (2022)
	Total evaporable moisture content	0.61%	ASTM C566 (2019)
	Specific gravity	25604 N/m ³	ASTM C128 (2022)
	Bulk density (loose condition)	14715 N/m ³	ASTM C29 (2023)
	Bulk density (compacted condition)	17070 N/m ³	ASTM C29 (2023)

The concrete mix was designed with a water-cement ratio of 0.5 and a sand-aggregate ratio of 0.48 based on absolute volume. The concrete blocks were prepared in accordance with ASTM C192 (2024). Additionally, cylindrical concrete specimens measuring 6 inches (0.1524 m) in diameter and 12 inches (0.3048 m) in height were cast to evaluate the concrete's compressive strength. Concrete blocks were formed using 18-mm-thick phenolic-panel wooden moulds (*Fig. 7a*). They were then cured for 28 days in a water pool at ambient temperature (*Fig. 7b*). The properties of fresh and hardened concrete are summarised in *Table 4*.

Table 4 Properties of the concrete manufactured for the construction of the blocks

Material	Property	Result	Standards
Freshly mixed concrete	Temperature	28.5°C	ASTM C1064 (2023)
	Slump	10" (0.254 m)	ASTM C143 (2020)
	Density	23190.8 N/m ³	ASTM C138 (2024a)
	Air	3.0%	ASTM C138 (2024a)
Hardened concrete	Compressive strength (9 days)	15.21 MPa	ASTM C39 (2024)
	Compressive strength (28 days)	24.33 MPa	ASTM C39 (2024)

**Fig. 7** Concrete blocks: (a) Making blocks, and (b) Curing of concrete

After curing, the concrete blocks were seasoned outdoors to air-dry. Subsequently, five randomly chosen blocks were evaluated, yielding an average density of 2364 kg/m³ (23190.8 N/m³). The coefficient of friction between concrete blocks was then evaluated using a tilting table (*Fig. 8*), following the procedure

described by Ita et al. (2024). Six blocks were placed on top of another six similar blocks ($0.20 \text{ m} \times 0.20 \text{ m} \times 0.40 \text{ m}$) on the rotating platform. The boundary condition was that the lower blocks were immobilised by a metal bar integrated on the platform (*Fig. 8*). The critical angles of rotation at which sliding of the upper blocks commenced (representing the angle of friction) were recorded as 33° , 33° , 34° , 34° , 37° , and 37° , with a calculated average of 34.67° . This resulted in an average coefficient of friction (μ) of $\tan 34.67^\circ \approx 0.7$. The average density and friction angle of this concrete (23190.8 N/m^3 and 34.67° , respectively) show a strong similarity to the reported properties of andesite stones used in Inca constructions at the Písac archaeological site in Cusco, Peru (density: $19816.2\text{-}25506 \text{ N/m}^3$; friction angle: $31.2^\circ - 44.72^\circ$; Ancori 2023).

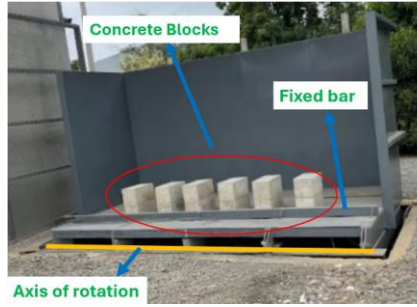


Fig. 8 Test to determine the coefficient of friction of concrete blocks

2.4. Construction of the walls

Fig. 9a and *Fig. 9b* illustrate the construction of Wall A-1. To mitigate sliding during overturning tests, the initial course of each specimen was placed on a horizontal surface adjacent to the fixed bar (*Fig. 9a*). Verticality was controlled after each course was placed using a plumb bob. *Fig. 9c* and *Fig. 9d* show the completed configurations of Wall A-1 and Wall B-1, respectively. Construction proceeded sequentially: Wall A-1, Wall B-1, and Wall B-2. Note that some blocks from Wall A-1 were reused to construct Walls B-1 and B-2. One significant finding was that, despite the construction blocks being intended to be as regular as possible, they did not achieve a perfect fit during wall construction.



Fig. 9 Construction of the walls: (a) Placing the first course, (b) Arranging the upper blocks, (c) Wall A-1 completed (model A), and (d) Wall B-1 completed (model B)

3. Experimental Tests

To document the overturning tests, four cameras were strategically set up to record the wall's behaviour from different perspectives: front (Lumix GH5S⁽¹⁾ camera), side (Lumix GH5S⁽²⁾ camera), diagonal (Sony a7RIV camera), and top (webcam), capturing each stage of rotation (*Fig. 10a*). All cameras were mounted 1 m above the ground. Additionally, a total station, positioned near the front camera, recorded the displacement of key points on the walls after each incremental rotation. *Fig. 10b* illustrates the points measured in Wall A-1, while *Fig. 10c* shows those in Wall B-1 and Wall B-2 (divided into vertical columns A-E). A reference point external to the loading platform was also monitored to ensure the integrity of the displacement measurements.

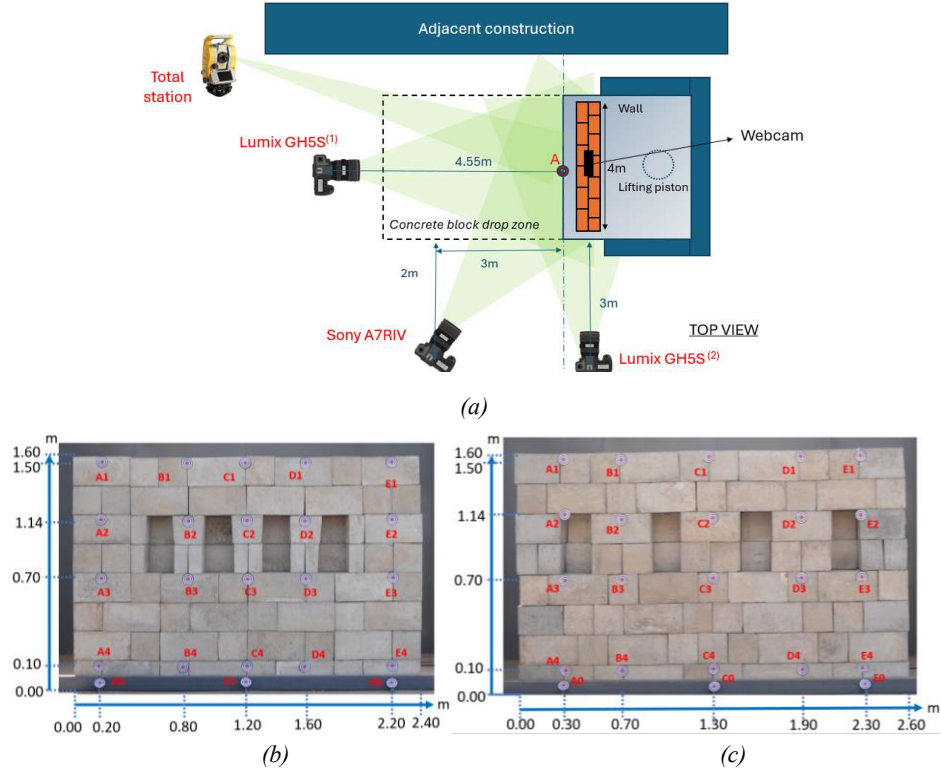


Fig. 10 Control points: (a) Relative Positioning of Cameras and Total Station (Top View), (b) Wall A-1, and (c) Wall B-1 and B-2

Before overturning, the control points were surveyed using the total station. The platform was then rotated incrementally, and the control points were re-surveyed at each new angle until the structure collapsed. For Wall A-1, angles of 0°, 5°, and 7°, plus the collapse angle, were evaluated. Based on these results, a finer angular resolution was deemed necessary for Walls B-1 and B-2, leading to the evaluation of 0°, 2°, 4°, 5°, 6°, 7°, 8°, 9°, and 10°, in addition to the collapse angle. The platform's angular velocity was maintained at approximately 0.2°/s throughout each rotation phase. For each angle of inclination (α), the weight of the wall (W) generates a force, F_h , acting in the direction out of the wall's plane, which is equal to $W \times \sin \alpha$.

3.1. Wall A-1

The wall's condition at 5° and 7° rotation is illustrated in *Fig. 11*. At 5°, the frontal and side views showed no changes; however, the diagonal view (*Fig. 11c*) revealed a minor separation of the upper blocks, highlighted in red. At 7°, no additional joint openings were observed.

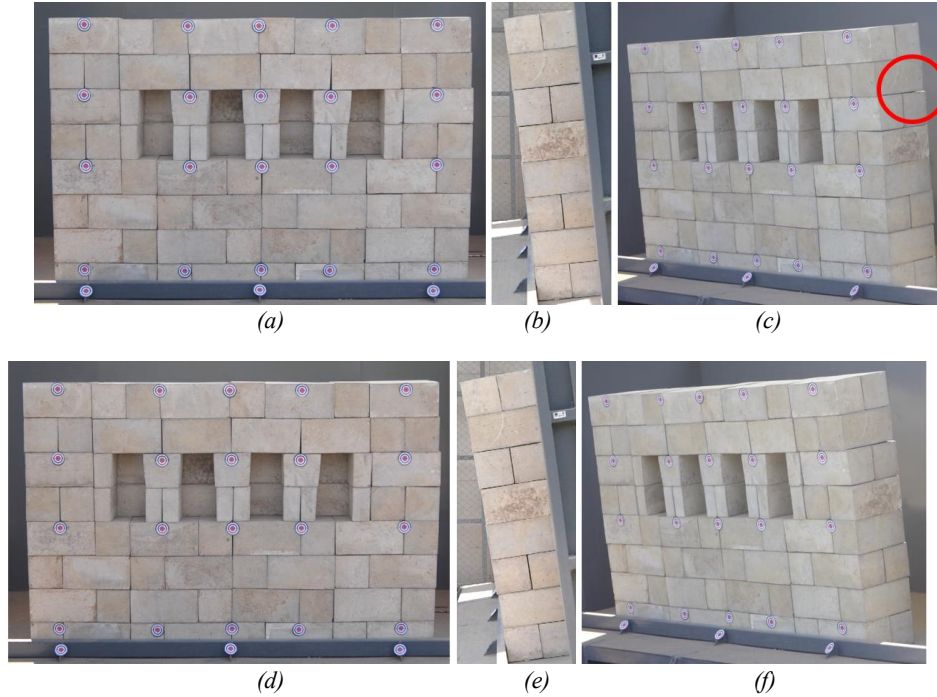


Fig. 11 Views of Wall A-1: (a) Front view (5°), (b) Side view (5°), (c) Diagonal view (5°), (d) Front view (7°), (e) Side view (7°), and (f) Diagonal view (7°)

Fig. 12 illustrates the wall's condition just before reaching 10° of rotation, with failure initiating around 9.8° due to overturning. As seen in *Fig. 12a*, the upper section of the structure begins to overturn from the 3rd course, and a running joint (red contours) develops along a vertical joint alignment on one of the niches. At approximately 9.85° of rotation (*Fig. 12b*), multiple failure planes form along the vertical joints, leading to collapse. *Fig. 12c* depicts the remaining structure at 10° of rotation. Additionally, *Fig. 12d* (approximately 9.85° of rotation) shows a failure at the rear of the wall, along a vertical joint midway up the structure.

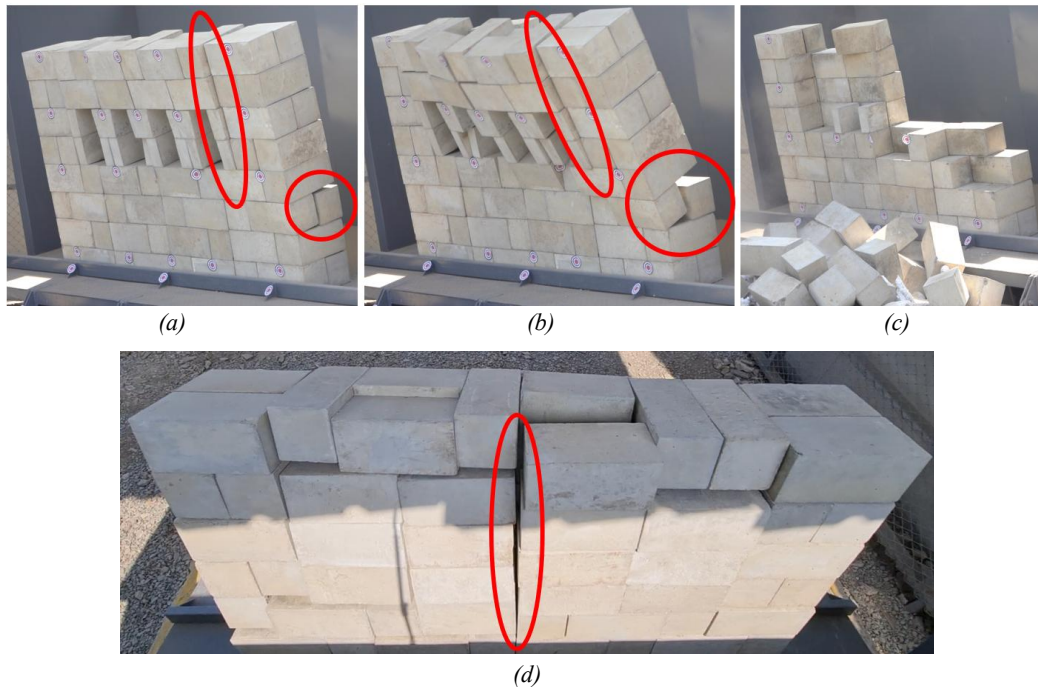


Fig. 12 Views of Wall A-1 at different inclination angles: (a) 9.8°, (b) 9.85°, (c) 10°, and (d) 9.85°

The test concluded with the tilting table rotated to 12° . *Fig. 13* illustrates the condition of Wall A-1 at various rotation angles. Specifically, *Fig. 13a* shows that, again due to a running joint, a portion of the wall collapses along a shear plane. *Fig. 13b* reveals only a remaining tower of blocks on the left side. Finally, *Fig. 13c* illustrates the collapse of this tower, with only blocks from the 1st to the 4th courses remaining on the platform.

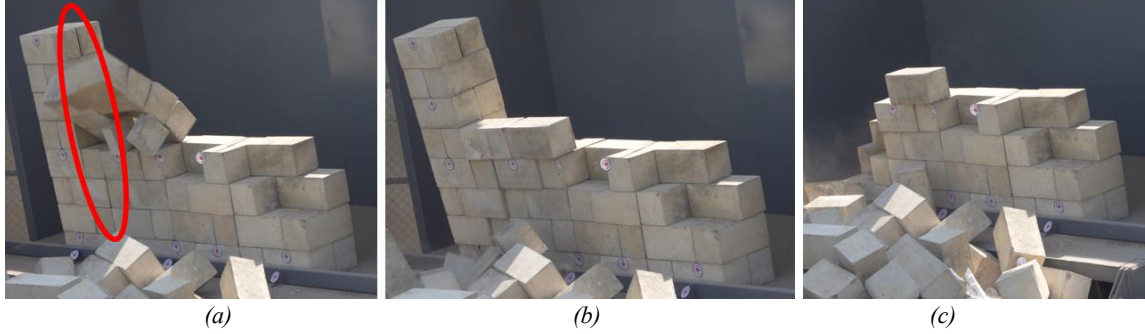


Fig. 13 Views of Wall A-1: (a) Approximately 10.8° , (b) Approximately 10.9° , and (c) 12°

3.2. Wall B-1

The wall's condition at a 10° rotation is illustrated in *Fig. 14*. Visual inspection at this stage revealed no relative displacements.

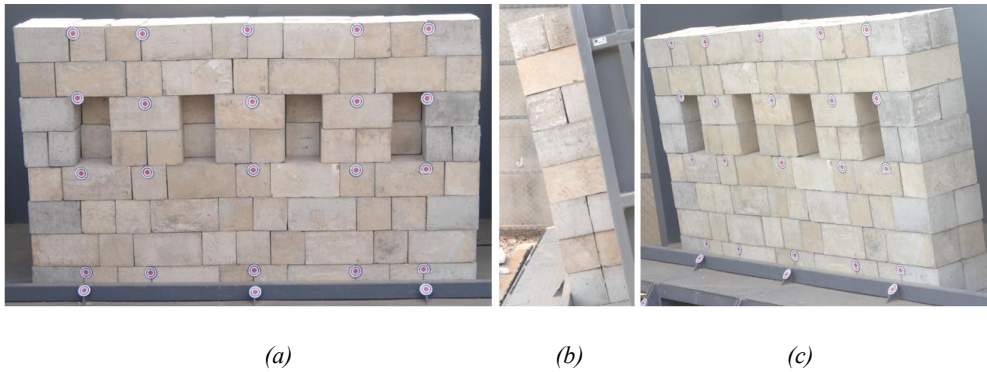


Fig. 14 Views of Wall B-1 at 10° rotation: (a) Front view, (b) Side view, and (c) Diagonal view

Fig. 15 illustrates the wall's condition just before reaching 11° , at which point failure initiates between the 2nd and 4th courses, leading to wall rotation. An interesting observation is that the wall's global collapse and the failure zone (red contours in *Fig. 15a* and *Fig. 15b*) match the initial failure point in Wall A-1. This collapse mode is consistent with Jaimes' findings (2024) from 1:4-scale concrete block wall studies.

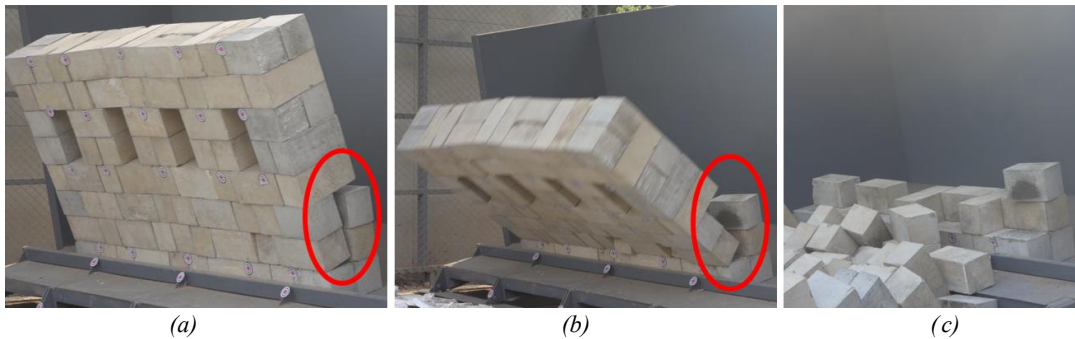


Fig. 15 Views of Wall B-1 between 10 and 11° of rotation: (a) Approximately 10.65° , (b) Approximately 10.75° , and (c) 11°

3.3. Wall B-2

The wall at 7° and 10° rotation is shown in *Fig. 16*. At 7°, a minor separation between blocks is visible in the same failure-prone area observed in previous walls (red contours in *Fig. 16b*). At 10°, *Fig. 16e* and *Fig. 16f* reveal a slight opening within the red-outlined area.

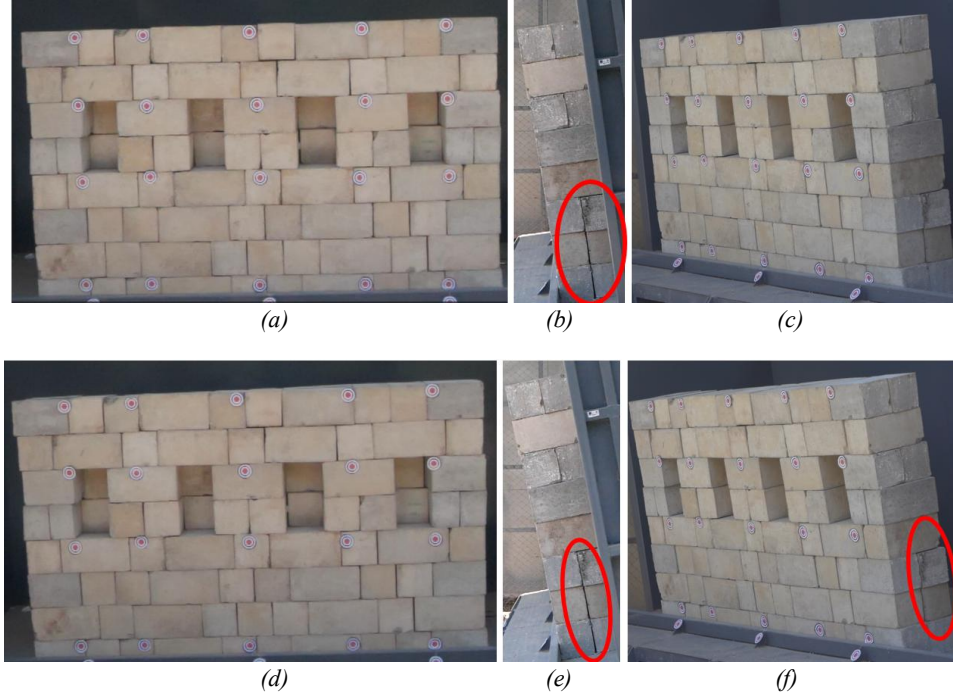
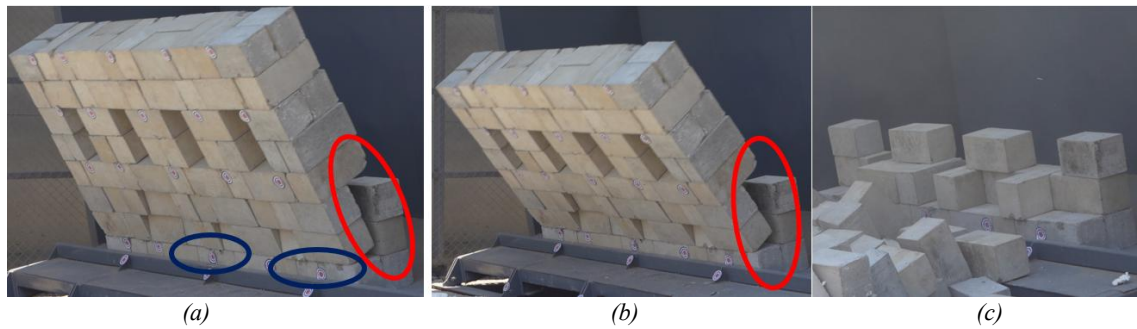


Fig. 16 Views of Wall B-2: (a) Front view (7°), (b) Side view (7°), (c) Diagonal view (7°), (d) Front view (10°), (e) Side view (10°), and (f) Diagonal view (10°)

Fig. 17 illustrates the wall just before reaching 11°. Wall B-2 exhibits a failure mechanism comparable to Wall B-1, characterised by a global collapse of the superstructure above the 1st course. *Fig. 17a* highlights (with blue outlines) minor crushing damage that occurred during the collapse, specifically along the upper edge of several first-course blocks. *Fig. 17c* shows the final, nearly symmetrical state of the structure, closely resembling the post-collapse condition of Wall B-1. Finally, *Fig. 17d* depicts the almost symmetrical collapse of the wall's rear section.



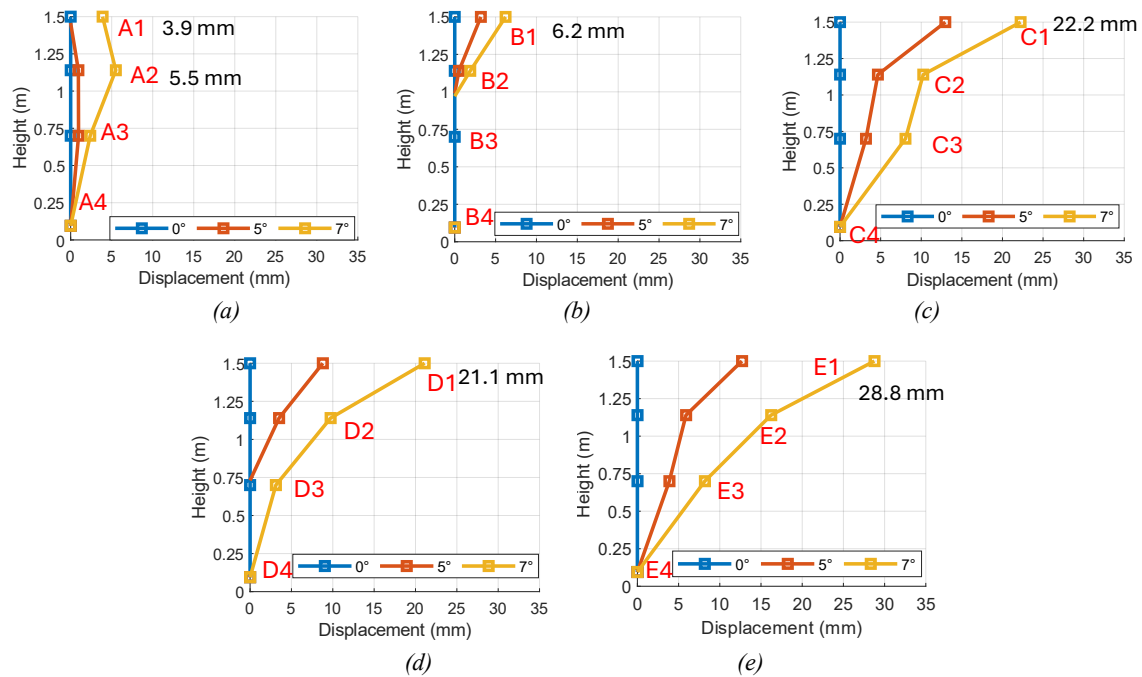
(d)

Fig. 17 Views of Wall B-2 between 10 and 11° of rotation:

(a) Approximately 10.55°, (b) Approximately 10.65°, (c) 11°, and (d) Approximately 10.65°

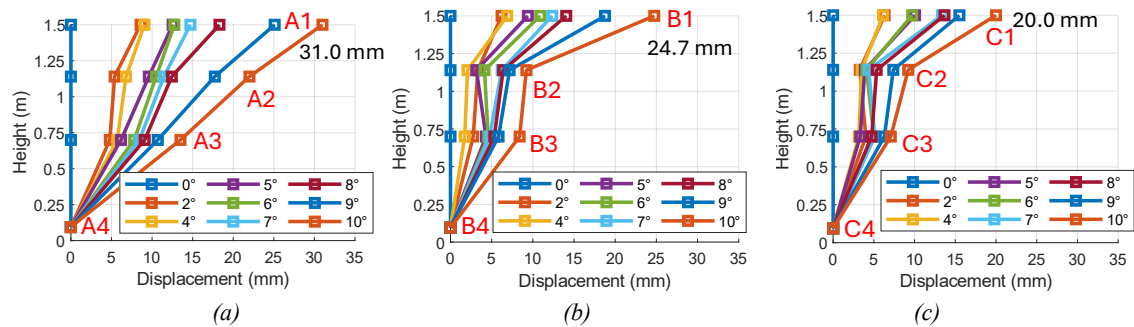
4. Analysis of Experimental Results

Fig. 18 illustrates the relative displacements of Wall A-1's control points (indicated in Fig. 10b for columns A-E) measured by the Total Station with respect to its base at each rotation increment. These displacements were measured consistently perpendicular to the wall plane at each evaluated rotation. In this case, only three rotation angles were assessed. As previously mentioned, this was because this specimen served as a pilot wall. As shown, the control points for column E exhibit the most significant relative displacements, corresponding to the collapse initiation zone at 9.8° of rotation (Fig. 12). Conversely, the minimum displacements are observed in column A, the zone that remained stable at 9.8° (Fig. 12). The maximum displacement recorded at 7° was 28.8 mm (point E1), resulting in a drift of 1.92%.

**Fig. 18 Relative displacement of Wall A-1:**

(a) Column A, (b) Column B, (c) Column C, (d) Column D, and (e) Column E

Fig. 19 illustrates the relative displacements of the control points on Wall B-1 (indicated in Fig. 10c for columns A-E) with respect to its base at each rotation increment. In this case, the control points for column A exhibit a slightly larger relative displacement compared to the other columns. However, the collapse of Wall B-1 at 10.65° of rotation involved the entire upper section with a global overturning mechanism (Fig. 15). The maximum displacement recorded at 10° was 31.0 mm (point A1), resulting in a drift of 2.07%.



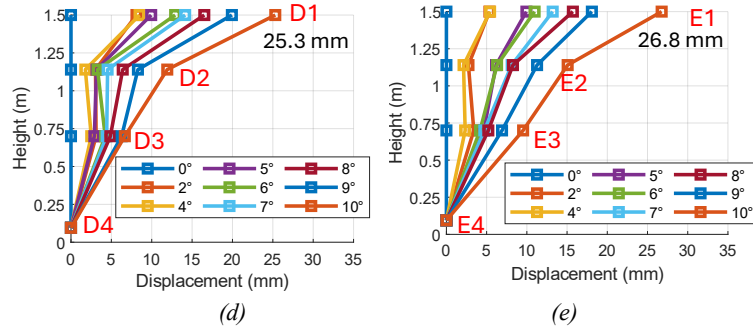


Fig. 19 Relative displacement of Wall B-1:

(a) Column A, (b) Column B, (c) Column C, (d) Column D, and (e) Column E

Fig. 20 illustrates the relative displacements of the control points on Wall B-2 (indicated in Fig. 10c for columns A-E) with respect to its base at each rotation increment. The control points in column C exhibit a larger relative displacement than those in columns A and E. Similar to Wall B-1, the collapse of Wall B-2 at 10.55° of rotation involved the entire upper section with a global mechanism (Fig. 17). Notably, the displacements in this wall are slightly smaller than those observed in Wall B-1. The maximum displacement recorded at 10° was 18.5 mm (point C1), resulting in a drift of 1.23%.

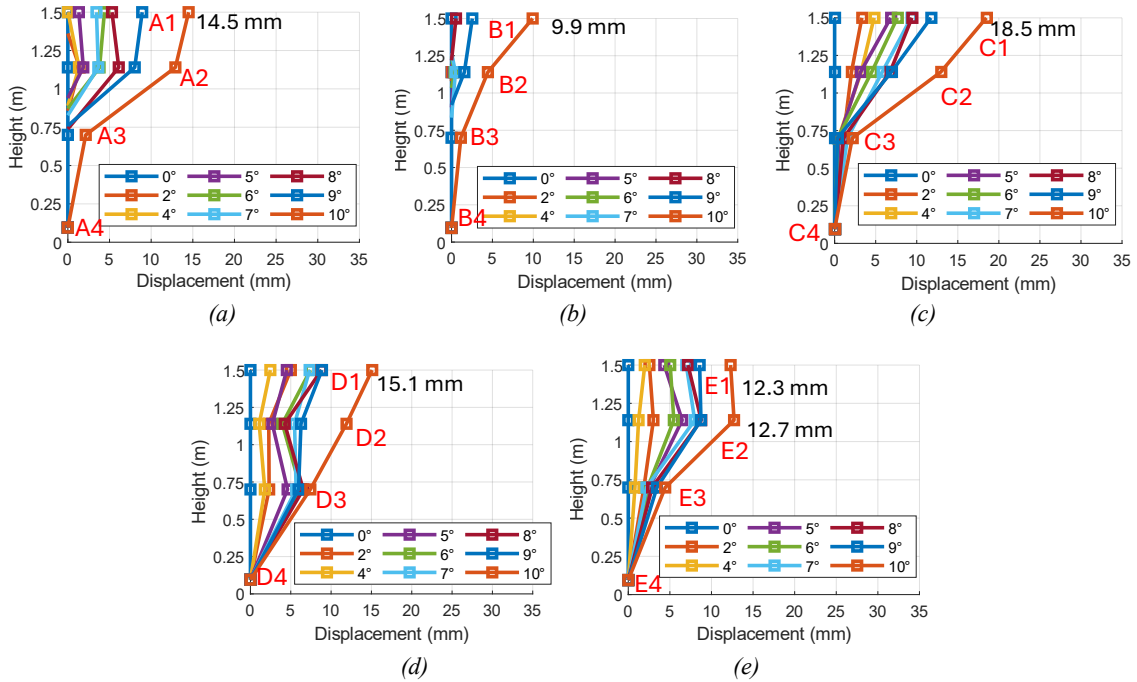


Fig. 20 Relative displacement of Wall B-2:

(a) Column A, (b) Column B, (c) Column C, (d) Column D, and (e) Column E

Table 5 presents the collapse angle, the percentage ratio of the perpendicular force to the weight at collapse (F_h/W), the maximum drift, and the average drift measured 1.5 m above the base at points A1, B1, C1, D1, and E1 for each wall. All walls failed by overturning, though Wall A-1 (Model A) exhibited premature partial collapse due to its vertical running joints. However, its collapse angle did not differ significantly from those of the other walls. Similarly, the average F_h/W showed no significant variation, maintaining a mean of 18.06% across all tests. This value is relatively low compared with other wall typologies, which have achieved up to 37% (Santa-Cruz et al. 2021; Ita et al. 2023; Ita et al. 2024). However, the average drifts align well with values reported by other authors for different wall typologies (Jaimes 2024). Notably, the Coefficient of Variation (COV) of the set of drifts located at 1.5 m height in Wall A-1 was much higher (66%) than for the other walls because the running joints led to varied displacements at the upper

measurement points, a phenomenon absent in Walls B-1 and B-2. Furthermore, Walls B-1 and B-2 exhibited similar overturning collapse behaviour.

Table 5 Summary of experimental results for the three wall specimens

	Collapse angle (degrees)	F_h/W (%)	Drift max (%)	Average drift at 1.50 m height (%)
Wall A-1	9.8°	17.02	1.92	1.10 (COV: 66%)
Wall B-1	10.86°	18.84	2.07	1.70 (COV: 15.5%)
Wall B-2	10.55°	18.31	1.23	0.94 (COV: 22.9%)
Average	10.40°	18.06	1.74	1.25
COV (%)	5.24	5.18	25.75	

Fig. 21 shows the curves of drift (averaged at 1.5 m above the wall) versus the F_h/W ratio (in percentage) for the three walls tested. Walls B-1 and B-2 exhibit similar structural behaviour, though Wall B-2 is initially more rigid than Wall B-1. Based on these results, it can be inferred that these structures withstood up to 1.25% drift on average.

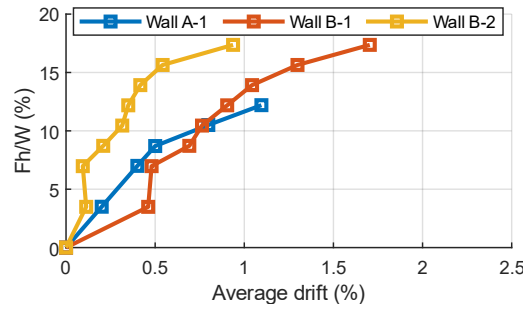


Fig. 21 Capacity curves for the three wall specimens, based on the average drift at a height of 1.5m (%)

5. Numerical Analysis

Numerical modelling of the tested walls was conducted using Abaqus© to assess the feasibility of reproducing the collapse behaviour of these heritage structures. Numerical models of the tested walls were developed using finite-element analysis and rigid-body mechanics. Explicit analyses were employed to investigate the nonlinear dynamic behaviour. Contact laws, defining normal and tangential behaviour between blocks, were implemented. The numerical models of the walls were subjected to overturning simulations, and the outputs included the collapse mode, collapse angle, and relative displacements at the specified control points.

The Computational Mechanics Group at the Polytechnic University of Madrid provided computational resources for all analyses. This included a computer equipped with an INTEL© Xeon© (R) CPU E5-2643 v4 @ 3.40 GHz (6 physical, 12 logical processors) and 32 GB of RAM, running Windows 11.

5.1. Mechanical behaviour of rigid bodies

Given the high stiffness and strength of the stone blocks, they were modelled as individual rigid bodies (Claxton et al. 2005; Gong et al 2022) in the numerical analysis. This approach enabled the development of a rigorous nonlinear simulation that accounted for large block displacements and contact and friction between them, while reducing computational time. Each block is represented by a rigid body controlled by a single reference node (Dassaults Systems Simulia 2023a), which has six degrees of freedom: three translational, $\mathbf{u}$ (u_1, u_2, u_3), and three rotational ϕ (ϕ_1, ϕ_2, ϕ_3). These rotational components constitute the Euler rotation vector, which defines the orientation of the rigid body in space. Additionally, the contact interface between any two blocks is achieved by user-defined discretisation of the surfaces and edges. The procedure employed allows full consideration of uneven distributions of normal and tangential contact forces along lateral surfaces, edges, and corners, thereby enabling rigorous modelling of large

5.2. Mechanical behaviour of contact joints

Given the inherent complexity of the mechanical behaviour of contact joints, several models have been developed to characterise rough-surface interactions, including the Hertz solution, statistical models, and various contact-stiffness criteria. These approaches typically account for elastic-plastic deformation under combined normal and tangential loads. In this study, the Abaqus penalty contact method was utilised. This constraint enforcement algorithm detects node-into-face and edge-into-edge penetrations at each time increment. A primary advantage of the penalty method is its ability to minimise over-constraint issues and accelerate convergence by reducing the number of iterations. Furthermore, increased mesh discretisation leads to higher numerical accuracy. This model incorporates a normal and tangential contact stiffness (K_n and K_t respectively) to govern the corresponding contact forces, as described by Equations 1 and 2. This criterion was chosen because the current contact area between the stones is known (Pirzada et al. 2020).

$$F_n = K_n \cdot \Delta u_n \cdot A_c \quad (1)$$

$$F_t = K_t \cdot \Delta u_t \cdot A_c \quad (2)$$

Where F_n is the normal contact force, F_t is the tangential contact force, Δu_n is the relative displacement between two bodies in contact (penetration between blocks), Δu_t is the relative displacement between two bodies in tangential contact, and A_c is the contact area.

The maximum shear force is obtained from the Coulomb equation, where slip occurs when the maximum possible shear force is reached ($F_{t,crit}$):

$$F_{t,crit} = F_n \cdot \tan \varphi = F_n \cdot \mu \quad (3)$$

Where φ is the contact friction angle, and μ is the coefficient of friction. As an alternative, viscous contact damping can be implemented to regulate relative surface motion. By incorporating damping at the contact interface, kinetic energy is effectively dissipated, facilitating numerical equilibrium and significantly reducing computational time. This mechanism is defined by a damping coefficient expressed in units of pressure per unit velocity. In Abaqus, this results in the generation of a force acting in opposition to the relative motion, thereby stabilising the contact interaction as follows (Dassaults Systems Simulia 2023b):

$$F_{damping} = \mu_0 \cdot A_c \cdot Vel_{rel} \quad (4)$$

Where μ_0 is the damping coefficient and Vel_{rel} is the rate of relative motion between two surfaces. A small amount of viscous contact damping (0.03) is used to reduce solution noise.

5.3. Numerical evaluation of the constructed walls

Model A (Wall A-1) and Model B (Walls B-1 and B-2) were constructed in Abaqus/Explicit. The blocks were assigned a density of 2364 kg/m^3 (23190.8 N/m^3) and a friction coefficient of 0.7 ($\mu = \tan 34.67^\circ$), values derived from the experimental campaign (Chapter 3). *Fig. 22a* and *Fig. 22b* show the Abaqus representations of Model A and Model B, respectively. The models were meshed with cubic C3D8R hexahedral elements, each with a side length of 0.05 m. Meshes with 0.01 m elements were also evaluated; however, they did not significantly alter the results and substantially increased computational time. Therefore, 0.05 m elements were determined to be the optimal choice for the analysis.

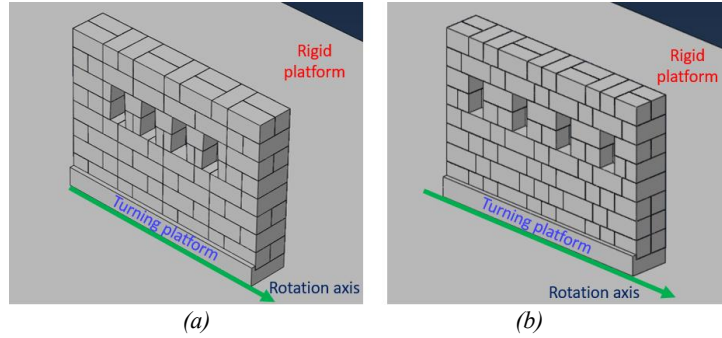


Fig. 22 Models in Abaqus/Explicit: (a) Model A (Wall A-1), and (b) Model B (Wall B-1 and B-2)

As depicted in Fig. 22, the numerical models incorporated two rigid platforms: a fixed base and a turning platform. The rigid platform was assigned embedded boundary conditions that prevented all translations and rotations. The turning platform was constrained to rotate only about the rotation axis shown in Fig. 22, simulating the laboratory tilting table. Furthermore, the model included a $0.05 \text{ m} \times 0.05 \text{ m}$ upward projection on the rotation platform, analogous to the fixed bar used in the experiments (Fig. 4 and Fig. 9a). This projection prevents the first course from sliding, mirroring the conditions of the physical tests.

A time increment of 0.0001 s was chosen for the explicit simulation. While a finer increment was possible, it significantly increased computational cost without yielding substantially different results. The analysis proceeded through the following steps:

- Step 1- Gravity (0 - 2 s): Gravity was applied linearly over the first second and then held constant for the subsequent second.
- Step 2 – High-speed rotation (2 - 10 s): The turning platform was rotated at $1^\circ/\text{s}$ (up to 8°). This elevated rotational speed, compared with the experimental $0.2^\circ/\text{s}$, was adopted to mitigate the high computational demand of the explicit dynamic solver.
- Step 3 - Standard rotation (10 - 40 s): The turning platform was rotated at $0.2^\circ/\text{s}$ (from 8° to 14°), simulating the experimental rate.

Furthermore, the normal stiffness (K_n) and tangential stiffness (K_t) are inherently dependent on the material properties and the area of contact between the blocks. Table 6 compiles K_n and K_t values reported by various researchers in numerical models of different stone types.

Table 6 Contact properties of various stone materials

Stone type	K_n (N/m^3)	K_t (N/m^3)
Semi-carved limestone (Walker et al. 2007)	1×10^9	5×10^8
Semi-carved granite (Walker et al. 2007)	1×10^8	2.5×10^9
Semi-carved shale (Santa-Cruz et al. 2021)	5×10^7	2.5×10^7
Semi-carved cobblestone (Santa-Cruz et al. 2021)	1.75×10^8	0.86×10^8
Irregular andesite (Ita et al. 2023)	5×10^7	2.5×10^7
Semi-carved shale and limestone (Lipa et al. 2024)	2×10^8	1×10^8

Table 7 presents the collapse angles from the experimental tests and the numerical models with different K_n values. Following the trend in Table 6, tangential stiffness (K_t) was set to half the normal stiffness (K_n) for this case study. As shown in Table 7, a K_n of $4 \times 10^8 \text{ N/m}^3$ yielded collapse angles similar to those observed experimentally. The computational time for Model A was 2 hours, and for Model B, it was 2 hours and 11 minutes.

Table 7 Comparison of collapse angles of experimental and numerical tests

Model	Case	Model A	Model B
Experimental tests	Wall A-1	9.80°	-
	Wall B-1	-	10.65°
	Wall B-2	-	10.55°
Numerical model	$K_n = 2 \times 10^8 \text{ N/m}^3$	8.00°	9.12°
	$K_n = 4 \times 10^8 \text{ N/m}^3$	9.64°	10.36°
	$K_n = 6 \times 10^8 \text{ N/m}^3$	10.44°	11.36°

Table 8 and Table 9 present a comparison of the average drifts measured at 1.5 m above ground level between the experimental and numerical tests. As shown in Table 8 (Wall A-1), the drifts at 5° and 7° are very similar in the numerical and experimental models, despite Wall A-1 being only a pilot specimen. In Table 9, referring to Walls B-1 and B-2, the average numerical drifts are, in most cases, very close to the average experimental values. However, a higher numerical drift is observed at 9° and 10°, which are close to collapse. This variation is attributed to the contact law used (Equations 1 and 2).

Table 8 Wall A-1: Experimental and numerical drifts

Angle	Fh/W (%)	Wall A-1	
		Experimental	Numerical
		Drift average (%)	
5	8.7	0.560	0.525
7	12.2	0.954	0.988

Table 9 Wall B-1 and B-2: Experimental and numerical drifts

Angle	Fh/W (%)	Wall B-1	Wall B-2	Wall B-1 and B-2
		Experimental		Numerical
		Drift average (%)		
0	0.0	0	0	0
2	3.5	0.461	0.113	0.397
4	7.0	0.480	0.094	0.491
5	8.7	0.692	0.209	0.528
6	10.5	0.763	0.315	0.606
7	12.2	0.904	0.349	0.988
8	13.9	1.044	0.417	1.038
9	15.6	1.298	0.542	1.644
10	17.4	1.704	0.937	2.925

5.4. Collapse mechanisms

This section compares the collapse mechanisms observed in the numerical models with those from the experimental tests, focusing on the results obtained with a K_n of $4 \times 10^8 \text{ N/m}^3$. Fig. 23 illustrates the progressive failure of Model A (Wall A-1) at various inclination angles. As seen in Fig. 23c and Fig. 23d, the failure plane along the vertical continuity closely resembles that of the Wall A-1 experiment (Fig. 12a and Fig. 12b). The collapse of the niches in Fig. 23e is also very similar to that of Wall A-1 (Fig. 12b). However, the numerical model does not capture the uplift from the third course, observed in the experiment.

Furthermore, Fig. 23f shows a remaining tower on the right side, analogous to a tower observed in Wall A-1 at approximately 10.9° (Fig. 13b), although located at the opposite end. Finally, Fig. 23g depicts the final state of the structure, which is, overall, similar to that of Wall A-1 (Fig. 13c).

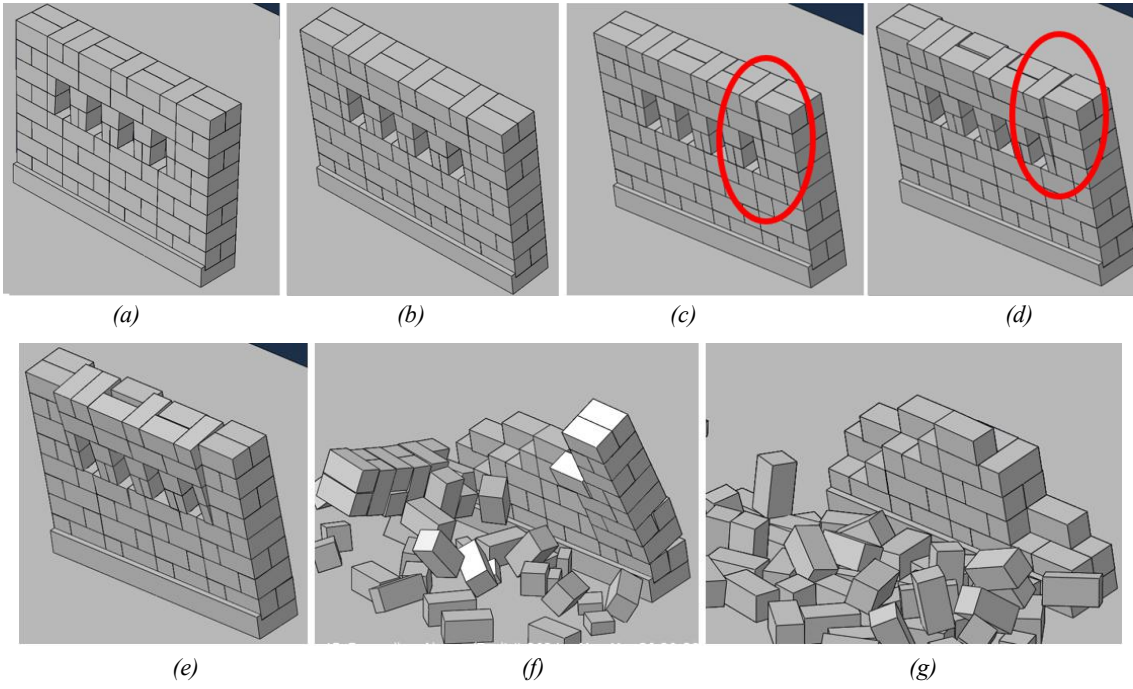


Fig. 23 Model A (Wall A-1) at different times:
 (a) 0° , (b) 8° , (c) 9.4° (d) 9.7° , (e) 9.8° , (f) 10.2° , and (g) 10.4°

Fig. 24 compares the rear view of the collapse mechanism of experimental Wall A-1 with that of the numerical model. As shown, the numerical model (Fig. 24b) predicts splitting planes caused by running joints, consistent with the experimental model (Fig. 24a). Furthermore, the movement of some upper blocks is similar to that of the experimental model.

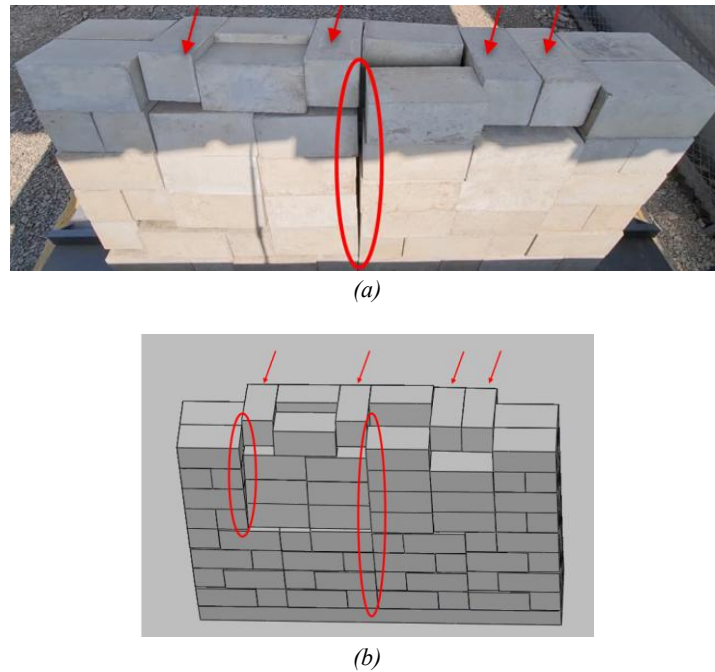


Fig. 24 Collapse mechanism of model I (wall A-1): (a) Experimental test and (b) Numerical model

Fig. 25 illustrates the progressive failure of Model B (Wall B-1 and Wall B-2) at various inclination angles. As shown in Fig. 25c, the collapse initiates similarly to the experimental tests on Wall B-1 and Wall B-2 (Fig. 15a and Fig. 17a, respectively). Furthermore, Fig. 25d demonstrates a global overturning mechanism

that mirrors the experimental behaviour (*Fig. 15b* and *Fig. 17b*). Finally, *Fig. 25e* depicts the remaining blocks on the platform post-collapse, which is also very similar to the experimental outcomes for Wall B-1 and Wall B-2 (*Fig. 15c* and *Fig. 17c*, respectively).

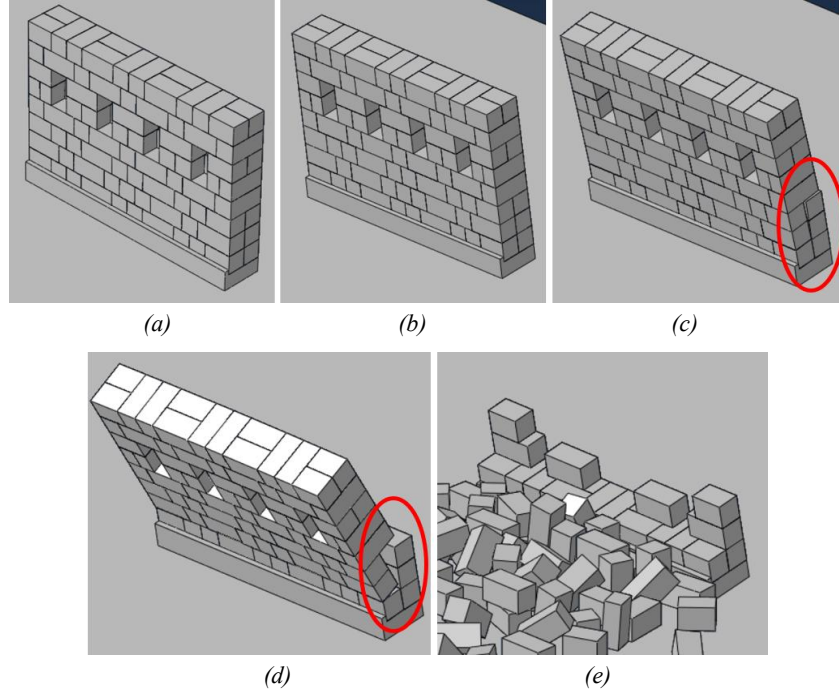


Fig. 25 Model B (wall B-1 and B-2) at different times: (a) 0° , (b) 8° , (c) 10.4° , (d) 10.5° , and (e) 10.8°

Fig. 26 compares the rear collapse mechanism of the experimental Wall B-2 with that of the numerical model. The near-identical failure mechanisms depicted in these images indicate a high degree of fidelity in the numerical model's reproduction of the structural collapse. Videos of both the experimental and numerical trials are available for download from the dataset (Lipa et al. 2026).

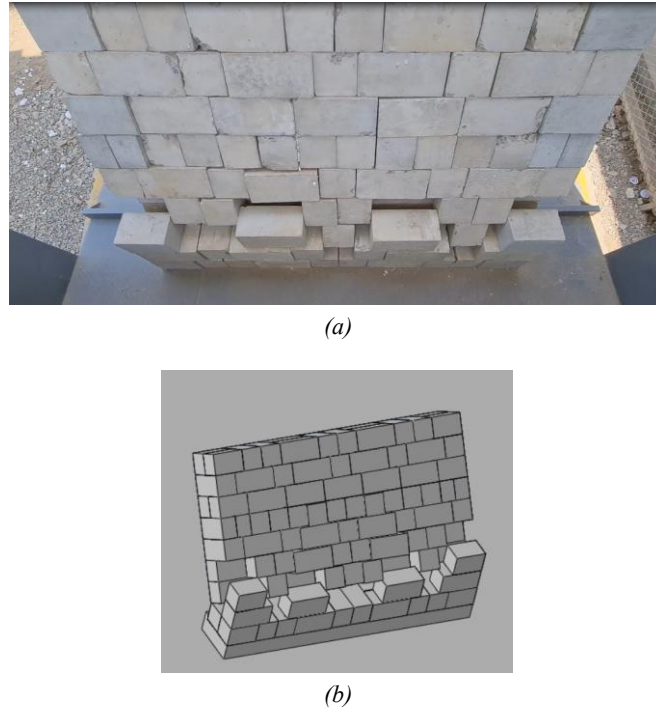


Fig. 26 Collapse mechanism of model B (Wall B-2): (a) Experimental test and (b) Numerical model

6. Conclusions

Numerical modelling of dry-joint stone historical walls, validated by experimental tests on concrete block replicas, shows promising results for assessing historical heritage. This is evidenced by strong agreement between numerical and experimental results for the collapse angle, collapse behaviour, and drifts obtained at various inclination angles. This reliability stems from the models' accurate representation of block geometry and the use of experimentally determined physical and mechanical properties. Specifically, applying a linear elastic-plastic contact model, with linear normal and tangential stiffnesses and constant damping at the contact points, is sufficient to reproduce the mechanical behaviour of these constructions. This explains the potential of finite-element-based numerical models with rigid blocks and contact laws for simulating the behaviour of dry-joint block walls, highlighting the critical importance of accounting for the physical model in their numerical evaluation.

While the model captures the general behaviour, it deviates from experimental drift measurements near the point of collapse. This stems from the specific contact law used; specifically, the concentration of force at the rotation line leads to excessive penetration. Future research should address this by implementing a finer mesh at the pivot points or employing alternative contact formulations.

Additionally, this study confirms that using concrete block replicas is a practical method for simulating carved stone walls, particularly when complex stone carving is too challenging. However, great care must be taken during the manufacturing process. Significant dimensional variations in the blocks may lead to inaccurate measurements of relative displacement. To improve precision and avoid this issue, it is recommended that new concrete blocks be cast consecutively, with the hardened adjacent blocks used as part of the formwork. This method would ensure more precise contact between blocks, yielding more accurate results.

Finally, the out-of-plane collapse of a dry-jointed stone wall was caused by base overturning. This suggests that friction between the blocks, arising from their near-perfect contact areas, yields overturning as the primary failure mode. Although the experiment did not achieve a perfect fit across all concrete blocks due to minor dimensional variations, the contact remained extensive, and the failure mechanism remained overturning. Additionally, if blocks are to be reused to repeat other experimental tests, it is advisable to manufacture additional blocks. These will replace those in the course that support the rotation of the upper blocks, as various tests have shown that some blocks in this course sustained edge crushing during rotation.

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8. Statements and declarations

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8.2. Competing interests

The authors declare that they have no relevant financial or non-financial interests to disclose.

8.3. Author contributions

Leonel Lipa conceived and designed the study, performed the experiments and simulations, analysed the data, and prepared the original draft of the manuscript. Luca Pelà and Nicola Tarque supervised the project, contributed to the methodology, and actively participated in the manuscript's writing, critical revision, and editing. Leonard Quispe and Edson Pillaca contributed exclusively to experimental work and data collection. José María Goicolea contributed to the implementation of the numerical modelling methodology and to the revision of the manuscript. Sandra Santa-Cruz supported the experimental testing methodology and contributed to the writing process by providing valuable comments and feedback on specific sections of the study. All authors have read and approved the published version of the manuscript.

8.4. Data availability

The dataset generated during the current study is available in the Zenodo repository: <https://doi.org/10.5281/zenodo.19582161>