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Behaviour of drystone retaining structures

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Drystone walling is an ancient form of wall construction, used worldwide wherever there is an abundance of raw building materials. However, very little research has been conducted on these structures, making their analysis difficult. As part of an ongoing investigation, four full-scale drystone retaining walls were built and tested to failure in a bespoke outdoor test laboratory. Through the course of the testing, the distinctive bulge patterns that are found in many in situ walls were successfully recreated. This paper describes the set-up of the test laboratory and instrumentation used, in addition to the proceedings of each wall test. Initial findings of the project tests and a discussion regarding the underlying reasons behind bulging in drystone walls are presented.

1. INTRODUCTION
Drystone walling is a type of construction used wherever suitable material is available. Construction style varies depending on regional traditions and on the characteristics of the stone used. Drystone walls are unmortared structures used for boundary walls, retaining walls and some simple building forms, using the interlock between stones and friction to maintain wall integrity and resisting overturning via self-weight. Generally, minimal shaping to the stone is applied, with construction relying on the skill of the mason to select an appropriate block for each location.

The majority of drystone retaining walls in the UK were constructed in the nineteenth and early twentieth centuries, lining in excess of 9000 km of the national road and rail networks (Powrie *et al.*, 2002). Most of these walls remain perfectly stable despite increased loading conditions and continual weathering. However, the walls often exhibit signs of post-construction deformation, such as bulging and leaning, and as such may be regarded as potentially less stable. With little guidance available to assist structural engineers in the assessment of these structures, responsible authorities are often forced to replace the walls at great cost based on visual inspection rather than following structural analysis. Replacement of all retaining walls along the UK's highways has been estimated to cost in excess of £10 billion (O'Reilly and Perry, 2009).

2. OBJECTIVES
Although historically constructed without the aid of codes of practice or modern analysis methods, drystone walls are complex structures that can be affected by several factors; the mechanical properties of the retained fill and the wall, age, build quality, location, foundation strength and loading conditions can all combine in a variety of ways to encourage instability or deformation.

To investigate the interaction and importance of these variables and further current understanding of the stability of drystone structures, an extensive study has been funded by the Engineering and Physical Sciences Research Council (EPSRC). As part of this work, four full-scale drystone retaining walls were constructed and tested to destruction. Analysis of these tests is being conducted at the University of Bath and Southampton University, where numerical models are under development to replicate physical testing. This paper describes the work done at the University of Bath, which includes the test set-up and procedure, details of the tests themselves, the manner in which failures occurred and analysis of the mechanisms that instigate failure.

3. DRYSTONE CONSTRUCTION
Although many differences exist between the various drystone construction styles, several common features are usually exhibited. Typical drystone walls are built in horizontal layers or 'courses' with each course ideally consisting of stones of a uniform thickness, thus presenting a straight and level appearance. The cross-section of the wall usually consists of a tightly packed outer face with a core of smaller blocks and fill packed behind (Figure 1(a)). Some drystone retaining walls follow this core material directly with the retained backfill material, while others have a second inner face, usually less well finished than the outer face. 'Through-stones' span from the outer to the inner face, binding the wall together (Figure 1(b)). Where there is no inner face, through-stones are often used to anchor the outer face further back into the packing fill. Coping stones can act in a similar manner, spanning the entire width of the wall at the crest (Figure 1(c)).

Each block within the wall should ideally be in contact with several other stones, and pressure upon any part of a freshly placed stone should not cause any rocking or lifting at the opposite corner. In practice, it is usually necessary to wedge in small shards of stone (known as pins) to prevent rocking. The unavoidable presence of these pins presents a weakness for all drystone structures, especially as weathering of these smaller

Figure 1. Drystone wall details: (a) face and fill; (b) through-stones; (c) coping stones
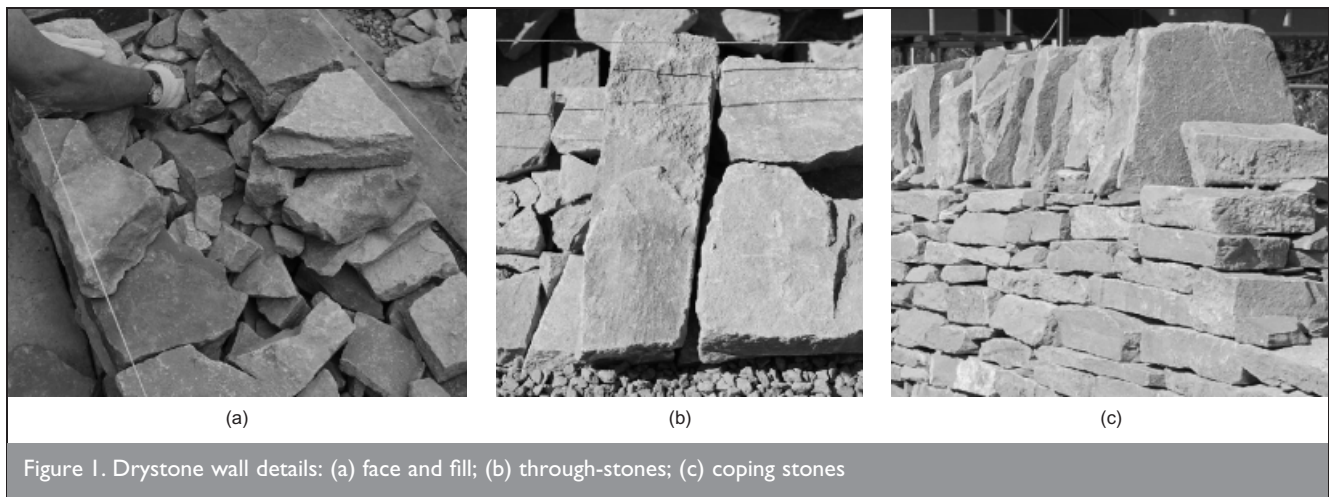
elements will occur more quickly than for larger stones. Pins are often used to allow a more even appearance to the face by tilting stones so that their outer surface is in the plane of the face, or to improve drainage. Thus the face of a structure can often give the misleading impression of a very tight well-ordered construction, while behind the face there are substantial voids held open by a large number of small pins.

4. PREVIOUS WORK

Despite widespread historic use and a growing resurgence of interest in drystone construction, very little research involving physical testing has been carried out on these structures. In 1834, Lieut-General Burgoyne carried out the first and almost only tests conducted to date regarding drystone retaining walls in an attempt to determine the most efficient geometry. Burgoyne constructed four 6·1 m high, 6·1 m long granite walls. The same volume of material was used for all the walls, but each was built with a different profile (Figure 2). Backfilling occurred after construction in small lifts, with records kept of the failure modes, movements and general observations. It should be noted that this work was only reported posthumously from Burgoyne's records (Burgoyne, 1853).

The work was conducted as scientifically accurately as possible for the time and, although dated, is still used for verification of several current drystone analysis techniques. Burgoyne proved that geometry has a very important impact on stability (Table 1), although it may be questionable how representative his

walls were of traditional drystone constructions; the walls were constructed of well-cut and tightly packed granite blocks, displaying a generally monolithic behaviour unrepresentative of most existing walls. In addition, and perhaps more importantly, each wall was built between bay walls, introducing the problem of end effects at these junctions. These end effects may have caused both the earth pressures and the walls themselves to behave differently; however, in terms of Burgoyne's aims of validating geometrical proportions, this would have little effect.

No further research involving purpose-built drystone test walls was carried out until the work conducted by Villemus et al. in 2004 (Villemus et al., 2007). This research focused on the need to quantify a safety factor for drystone walls, examining the effects of geometry, irregular block patterns and the internal failures that may occur within the wall. Testing was carried out on five full-scale walls, between 2 and 4·25 m high, which were loaded using hydrostatic pressure via a large PVC-lined bag.

Villemus et al. used short sections of wall – between 2 and 3 m long – in order to be able to view the cross-section of the structure from either end. The use of water to load the wall ensured that purely horizontal forces were applied during the test and, as a result of controlling the flow of water, the magnitude and position of the loading was at all times known. Through careful monitoring of the end faces during loading, the vectors of the internal blocks were determined, giving
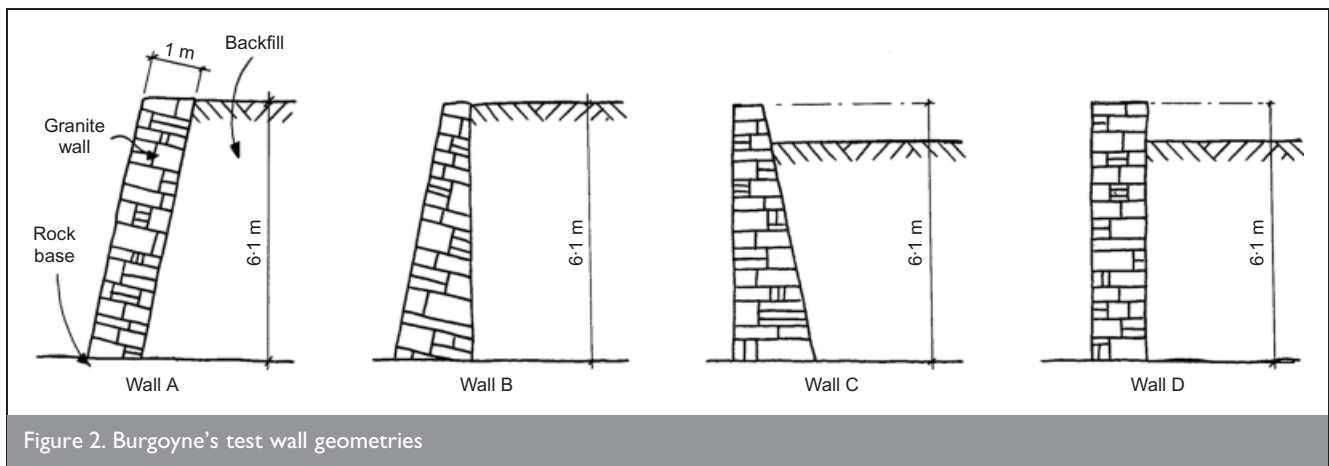

Figure 2. Burgoyne's test wall geometries

	Fill height attained	Failure mode	Observations
Wall A	Full height	N/A	No signs of distress
Wall B	Full height	N/A	Slight fissuring
Wall C	5·2 m	Bursting at 1·7 m	Bulging at 1·5 m, significant fissuring
Wall D	5·2 m	Toppling from base	0·45 m overhang prior to failure

Table 1. Burgoyne's test results

sufficient information to assess the static equilibrium of the structure. Hence, the stability against sliding or overturning failures could be calculated and compared with the physical test results.

As with the work conducted by Burgoyne, the unavoidable use of short test sections by Villemus *et al.* may have caused issues with the behaviour of the walls. The work described in this paper thus used test walls with sufficiently large width/height ratios to ensure that full three-dimensional behaviour may occur and be examined. In addition, while the use of water allows a full understanding of the applied forces, the importance of the vertical forces generated by friction at the wall–backfill interface was seen as sufficient reason for adopting the use of a more representative material for retention.

5. TEST SET-UP
Each of the tests described in this paper was carried out consecutively in a unique outdoor test laboratory. As mentioned earlier, to avoid the issue of end effects, each wall was required to have a significant length/height ratio; 12 m wall lengths were chosen, with a height of 2·5 m through the central test area (this includes coping stones that constituted

the top 300 mm). The central 4 m of each wall rests on an articulated platform, supported by four screwjacks, with the ability to move vertically as well as tilt forwards or backwards. This allows both foundation and backfill settlement to be imitated, with movements being directed from a remote control station at a rate of up to 10 mm/min. In addition, a steel frame was erected over the central portion of each wall, from which a 200 kN capacity hydraulic jack was suspended, allowing a localised surcharge to be applied through a loading plate onto the backfill (Figure 3).

Due to backfill pressures and surcharging, each test involved significant lateral forces that, if transferred to the jacks, could cause considerable damage. To avoid this, the screwjacks were pinned at each end to allow only axial forces, with steel bars anchoring the platform to a large concrete block to resist lateral loads. An advantage of this system is that it allows the use of simple tension/compression load cells on both the screwjacks and the anchor bars to monitor the overall horizontal and vertical forces being applied to the platform, and hence the wall resting upon it.

The material used to construct the walls (approximately 30 t for each test) was an undressed Cotswold limestone provided by Natural Stone Market Ltd. Limestone quarried from this region generally comes in two varieties that can be identified by their colour – either grey or a lighter, creamier colour. Grey limestone is generally considered to be much more durable and was used throughout this project. Constructed by a team of
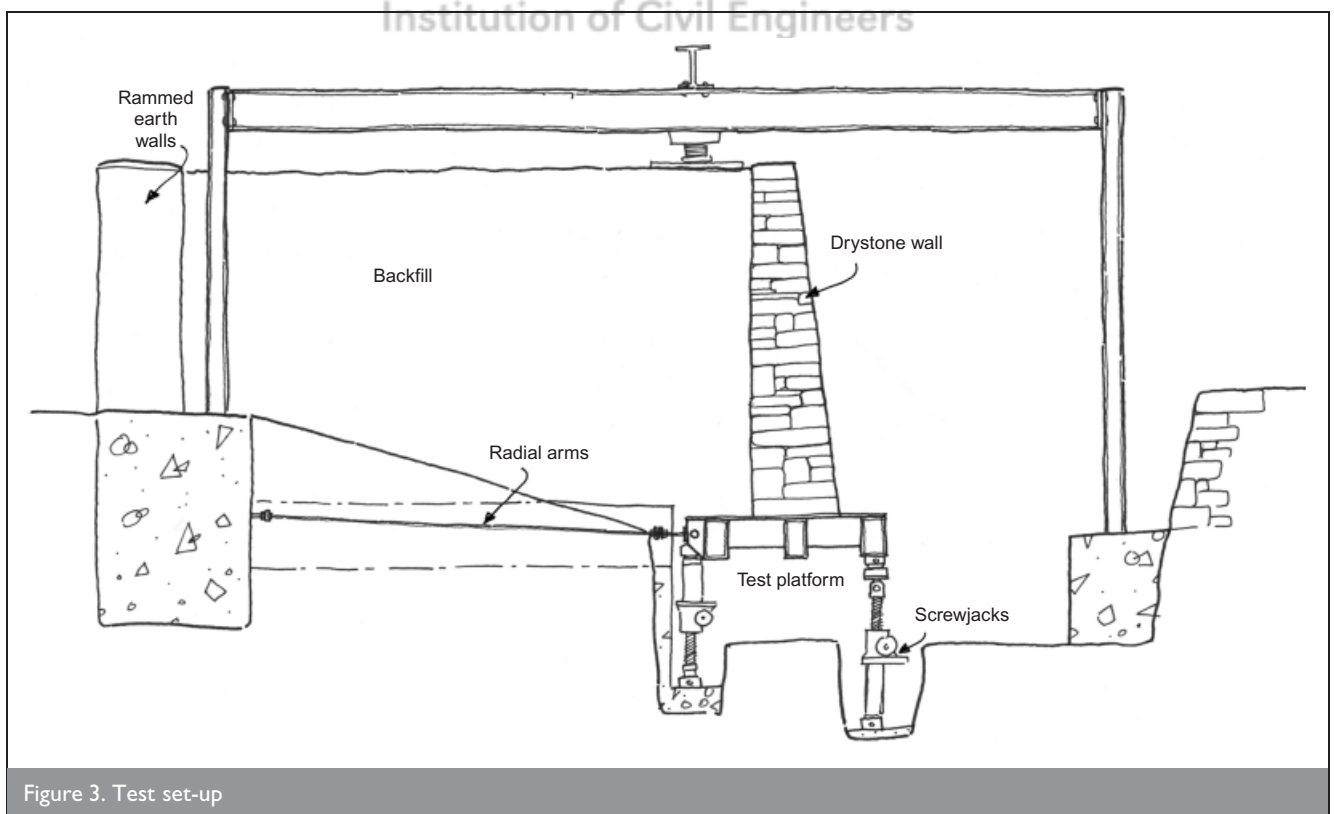

Figure 3. Test set-up

three professional masons and led by Richard Tufnell (member of the Drystone Wallers' Association), fabrication of the walls took between 3 and 5 days. The project employed professional masons to ensure that construction techniques were comparable with existing walls, as well as giving consistency between the tests.

The retained material for each test was a 14 mm single-sized aggregate; 100 t was required to completely backfill the walls and ensure that any failure planes that might develop would not be impeded by the test area's boundary walls. This particular backfill was selected to ensure that the retained material is completely free draining, allowing no build-up of pore water pressures that would cause complications when attempting to analyse wall behaviour because, to some extent, the actual pore pressure distribution would inevitably be unknown. Furthermore, capillary tension in finer-grained soils would undoubtedly reduce earth pressures, but to an extent that would be difficult to determine accurately. Elevated water pressures are certainly a factor in deformation and failure of drystone walls, but this phenomenon is better addressed by the aforementioned numerical work conducted at Southampton University than by practical testing.

6. INSTRUMENTATION

As previously mentioned, load cells attached to the platform monitor the overall forces being applied to the wall. The first two tests augmented this information by using small load cells within the backfill. These load cells were sandwiched between 100 mm × 100 mm steel plates and placed at critical locations within the backfill, orientated to record either horizontal or vertical pressures (Figure 4). The aim was to use these data to help determine the distribution of stress within the gravel arising from the surcharge loading. This form of monitoring was discontinued after the second test wall as the results were often inconsistent and erratic, mainly due to the small scale of the steel plates in relation to the size of the gravel. Larger plates were considered, which would give more reliable readings, but these would have a greater impact on the test itself and possibly affect wall behaviour.

Also of interest regarding the backfill is the manner in which it

moves during testing, and in particular where failure planes develop. Two methods were used to assess this, beginning with the placement of ball bearings within the gravel. Each ball bearing was numbered and its position determined using a total station. Upon destruction of each wall, the gravel was carefully unearthed and the ball bearings located using a metal detector. Using the total station to determine their final positions, the overall movements of the ball bearings were calculated and from these data the location of the gravel's failure plane found.

For the third and fourth walls, the ball bearings were replaced with long, very flexible plastic tubes placed vertically into the gravel using a mandrel. Throughout the tests, long marker poles were lowered down the flexible tubes until either the end or an obstruction (such as kinks caused by developing shear planes) occurred (Figure 5). With similar data obtained from several locations, this method identifies the failure plane quickly and easily during the test and so was adopted in favour of the time-consuming ball bearing approach.

To monitor the walls themselves, a combination of transducer, surveying and photographic techniques were used. A total station mounted on a fixed concrete column recorded the positions of marked points along the wall face, covering around 180 points per wall. Prior to failure, the behaviour of the wall is relatively static; movement ceases once loading is halted, allowing the necessary time to complete the surveying, which is accurate to ±1 mm.

To capture a visual record of the tests, four digital single-lens reflex (SLR) cameras were used extensively, along with full video recordings on a high-definition camcorder. Two of the cameras were attached to mounting points placed 500 mm apart equidistant from the wall, so that the images could be used as stereo pairs. The third camera was used as a roving camera, taking detailed images including bulges, cracks and movements. The fourth camera was used to monitor targets mounted on the wall face. The images were then analysed at

Figure 4. Load cells prior to burying within backfill

Figure 5. Gravel shear plane indicators

Southampton University using particle image velocimetry (PIV) techniques to accurately determine the monitored block movements and rotations (±0·1 mm).

The use of transducers was particularly important in capturing the final moments of each test, as surveying and photographic techniques cannot be relied on to capture critical moments. A series of draw-wire transducers were used with sacrificial lengths of wire between the instrumentation and the wall, with the instrumentation removed from the collapse zone, thus avoiding damage. Up to 25 transducers were used in each test (with the exception of wall 1 where no transducers were used), focusing on the central wall zones where the majority of movements occurred.

7. TEST PROCEEDINGS

7.1. Test wall 1
The first test wall was constructed in June 2007. It was well finished and tightly packed with a double-faced construction ranging in thickness from 600 mm at the base to 300 mm at the coping level, with the front face battered back 6·8° from vertical. As is common practice, layers of through-stones were incorporated at several levels, tying the two faces together.

Testing of this wall (Figure 6(a)) was carried out a month after construction in July 2007. The wall was regularly monitored using the surveying equipment to identify any settlement or deformation in this period. Backfilling took place in tandem with wall construction, ensuring that the fill height was approximately 0·5–1·0 m below the wall height, allowing a comfortable working position for the masons. The fill was introduced in layers of 300 mm and compacted with a 1 kN vibrating plate compactor until the full height of 2·2 m was achieved, leaving the top 300 mm of coping stones uncovered. Plate loading tests on the gravel indicated an initial angle of friction of 50·1°. As this is likely to be significantly higher than generally found behind walls of this nature, this issue was addressed for subsequent walls. The voidage of the wall was approximately 28%.

Testing was conducted over five days, consisting of a day of movement/loading, followed by a full day of observations to identify any further movements, with final collapse occurring on the fifth day. On the first day, the only action was a uniform raise of the platform by 20 mm. This was done to ensure that the backfill friction was fully mobilised against the back of the wall, and was repeated with each wall test. Readings from the platform load cells were monitored; after the initial spike in readings the loads were seen to plateau, indicating the maximum friction angle had indeed been achieved (Figure 7). In normal practice, this friction angle would be attained due to gradual settlement of the backfill or of the underlying soil, so raising the wall provides the same relative motion.

The third and fifth days of testing consisted of a combination of lowering the front jacks beneath the platform by a total of 75 mm, simulating localised foundation settlement (tilting the platform 3·75°), and surcharging of up to 110 kN. Initially, surcharging was via a 400 mm² plate located 500 mm from the rear face of the wall. However, it was found that before loads

(a)
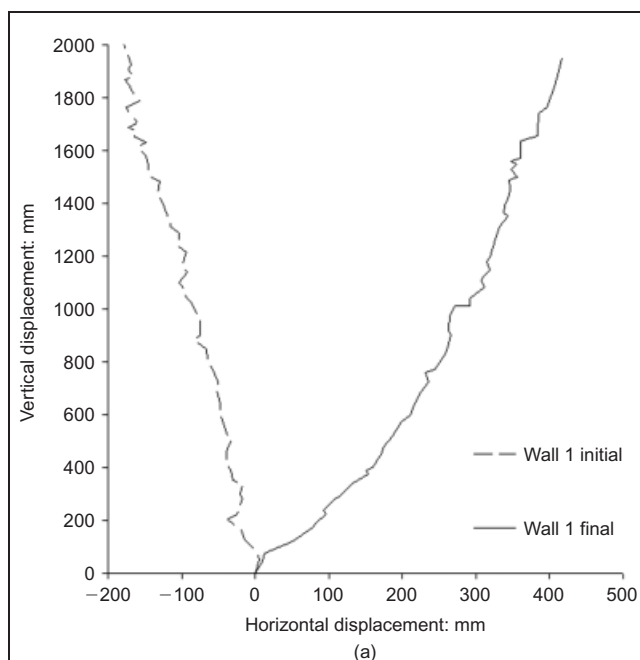
(b)

Figure 6. Test wall 1: (a) survey readings; (b) prior to failure

became sufficiently high to produce a failure wedge within the backfill, the plate punched through the surface of the gravel, causing the hydraulic jack to run out of stroke. In addition, the load was found to be slightly too close to the wall, so inducing movement primarily at the top of the wall, encouraging a toppling failure. The plate was therefore enlarged to 500 mm × 600 mm, and moved to 1 m away from the back of the wall.

Prior to failure, the wall had substantially deformed, having moved forwards over 650 mm at the crest, overhanging the wall toe by some 500 mm (Figure 6(b)). Although there was evidence of bulging within the structure, the primary failure mechanism was toppling, encouraged by both the rotation and the initial surcharge location.

7.2. Test wall 2
The second wall was of generally poorer construction quality, with an unfinished rear face, to increase flexibility and deformation. Although care was taken for each stone placement, less time was spent shaping the stones, giving a much rougher appearance, and 'running joints' (vertical lines whereby the joints between adjacent blocks correspond to those of the joint above or below) were intentionally
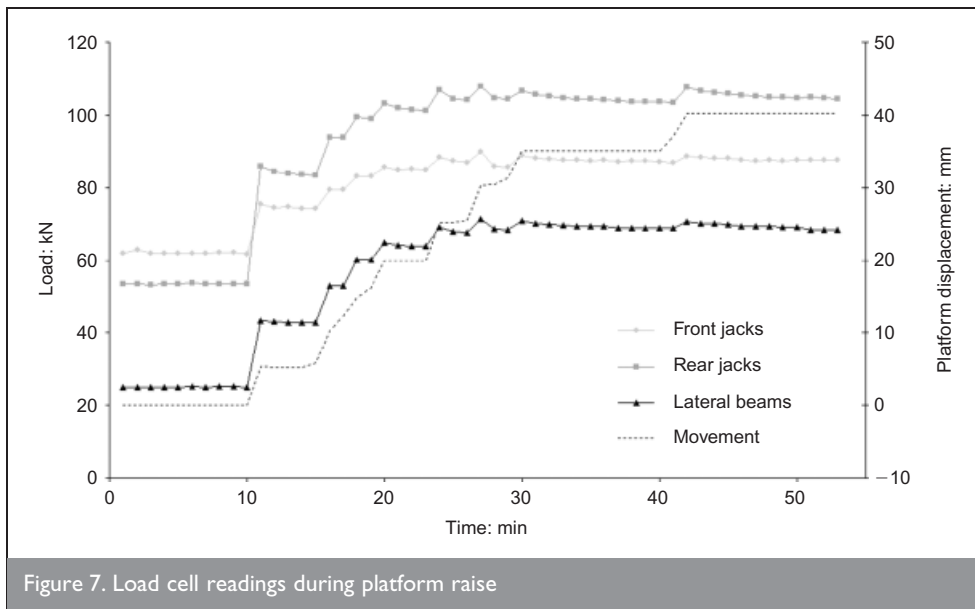
Figure 7. Load cell readings during platform raise

eventually allowing one or more to freely rotate and fail locally. These key blocks created a cascade effect, causing the area directly above to become destabilised, in turn initiating rotational failure of the central portion of the wall and ending the testing process.

7.3. Test wall 3

To encourage bulging while attempting to restrict the degree of toppling as the test progressed, the third wall (built in June 2008) was built with a wide profile similar to test wall 1 (600 mm at the base, tapering to 400 mm at the coping level), but with a much rougher build quality and utilising comparatively smaller stones. For test walls 1 and 2, the blocks used were generally large slab-like stones, usually 200–300 mm per side and roughly 50–100 mm thick. To encourage block rotation – judged to be a key factor in bulging development – much smaller stones were used in wall 3 in an attempt to create a wall using blocks with a close height/depth ratio. The voidage of test wall 3 was approximately 46%.

introduced into the structure. With sufficiently long running joints, the wall is less able to shed load into the more stable, unloaded 'wing wall' sections; this effectively allows the central section to behave in a more two-dimensional manner. The inclusion of an unfinished rear face also allowed the wall thickness to be reduced, as facing stones were no longer required, leaving the wall 500 mm wide at the base tapering to 300 mm at the coping. In addition, the backfill was placed uncompacted, giving a lower friction angle of 40·3° as determined by plate loading tests. The voidage of the wall was approximately 23%.

Testing, in October 2007, again took place over the course of 5 days, but consisted solely of raising of the platform to ensure full frictional interface between the wall and the backfill, and then loading via the 500 mm × 600 mm steel plate until bulging and subsequent failure mechanisms occurred. With the uncompacted, less dense backfill, the surcharging load peaked at 75 kN, at which point the failure plane developed within the retained material. However, as with all the tests, deformation and subsequent failure were displacement controlled and not dependent on any specific loading applied. In this way, the tests were continued past peak loads, allowing further deformation and eventual failure in a controlled and safe manner.

Final deformations were less severe than in test wall 1; the coping moved 300 mm in total, with some 250 mm overhang over the toe (Figure 8). The manner of movement was also much closer to the mechanisms found in many existing retaining walls. Instead of monolithic toppling of the first wall, the second wall displacements were non-linear with respect to vertical height. For example, an hour prior to failure an overhang of 100 mm was measured at a third-height of the wall, followed above by a gradual reduction in overhang, ending with the coping stones being 50 mm in front of the toe line.

Although test wall 2 also eventually failed by toppling, the manner in which this toppling was achieved was different to that of the first test wall. In this instance, it was found that the lower blocks of the wall were sliding over one another,

The backfill was again introduced uncompacted and the subsequent test followed a procedure similar to that of the second wall – that is, initially raising the platform until the loads stabilised then surcharging through to failure. Platform movement of 50 mm was initially implemented before the load readings were sufficiently consistent. This was followed by a surcharging that reached loads of 80 kN though the backfill as the failure wedge was generated.

After the initial platform movements, the applied loads to the structure caused definite bulging to occur within the wall. Monitoring of the flexible tubes within the backfill indicated that the failure wedge was developing at a relatively shallow angle, beginning directly behind the surcharging plate and terminating at the face of the wall some 500 mm above the toe. As a consequence, the bulge's centre was slightly higher than that produced in test wall 2, occurring roughly 1 m above the base and overhanging the toe by over 350 mm before failure (Figure 9). The extremely pronounced nature of this bulge, combined with the large internal void spaces due to the roughness of construction, caused significant cracks to open along the face of the wall. In addition, large amounts of material were able to drop both through internal voids within the wall and out of the wall face itself (Figure 10(a)).

Eventual failure was again because of bulging, driven by the continuous displacement of the backfill. However, there was also a visible bursting at various sections during the failure, as the areas below the main bulges were unable to resist the horizontal forces of the sections above (Figure 10(b)). As a result, a large amount of material slipped forwards rather than toppling from the toe as failure progressed.
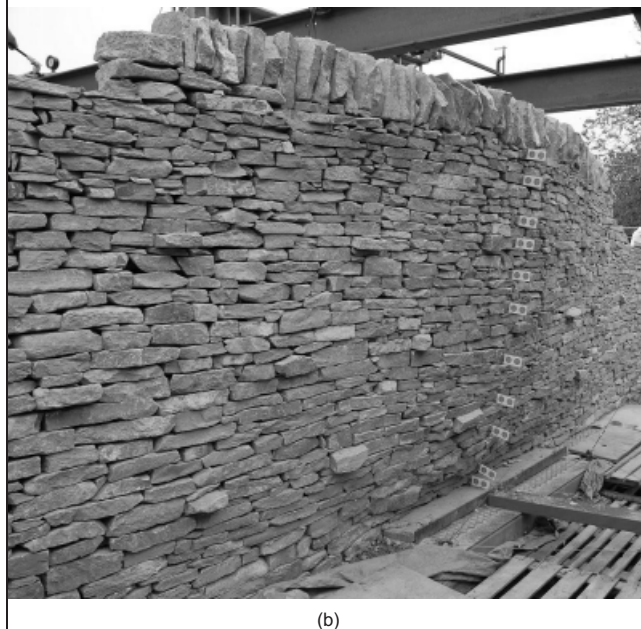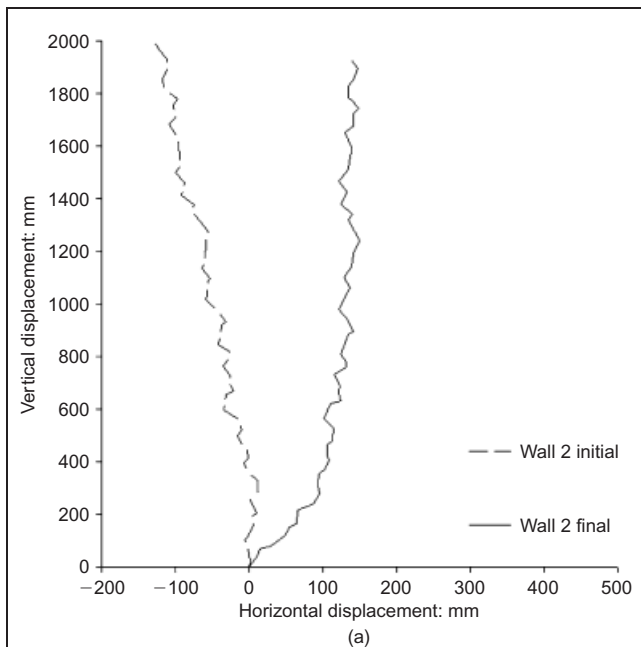
Figure 8. Test wall 2: (a) survey readings; (b) prior to failure

Figure 9. Test wall 3: (a) survey readings; (b) prior to failure

7.4. Test wall 4

After the success of the third test and the development of a significant stable bulge, a fourth test was carried out to establish whether this result could be reproduced given the inevitable random variations between structures. Identical construction styles to test wall 3 were adopted (minimal use of pinnings, no shaping of walling material, use of blocks with similar height/depth ratio, etc.). However, due to the use of different formworks to guide the masons during construction, test wall 4 was slightly wider (650 mm wide at the base) and with a slightly greater batter (8·0° rather than the 6·3° of wall 3). The voidage of the wall was approximately 44%. All other factors were kept consistent, including the test itself, which took place in August 2008.

Similarly to the third test, definite bulging began to occur with the introduction of the surcharge load on the backfill. The failure wedge began to develop once the surcharge reached
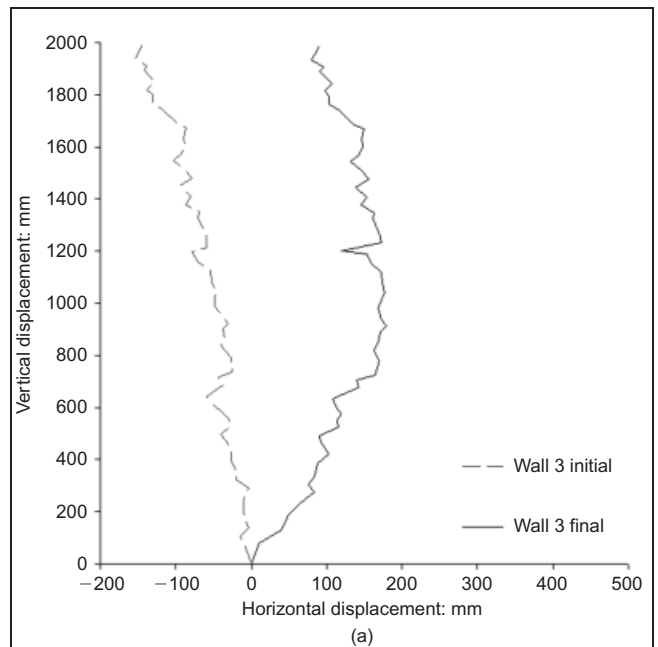
75 kN (maximum surcharge achieved throughout testing was 84 kN), again originating directly behind the loading plate and terminating at the wall face approximately 350 mm above the toe.

Maximum displacements, while remaining stable, were slightly lower than those in test wall 3, bulging outwards some 200 mm

at a vertical height of 1·1 m from the toe (Figure 11). However, the crest of the wall moved a comparatively smaller amount, giving a more visually obvious bulge. From the fixed mount camera images, it is also apparent that this wall contained a higher number of running joints with large vertical cracks opening approximately every 500 mm along the face of the wall. Collapse initially began over the central 4 m of wall as a

(a)

(b)

Figure 10. Test wall 3: (a) local failure; (b) bursting failure
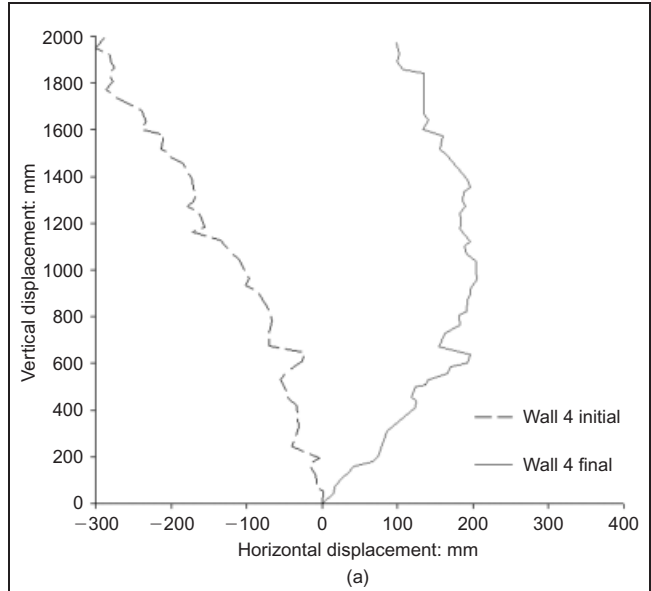
(a)

(b)

Figure 11. Test wall 4: (a) survey readings; (b) prior to failure

topple. However, once this failure was under way a large portion of the adjacent wing walls was destabilised and similarly collapsed.

8. ANALYSIS

There are currently no definitive assessment methods for analysing drystone walls. BS 8002: 1994 (BSI, 1994) recommends that gravity retaining structures be checked for both overturning and sliding, requiring respective factors of safety of 2·0 and 1·5. The safety factor of each of the test walls was thus initially checked using a simple limit equilibrium approach (Table 2). The values assume that the walls were initially fully backfilled but with no surcharge applied. In the case of wall 1, it is assumed that, due to the large deformations that took place, the fill dilated and hence a friction angle of 39° was used in the final stability analysis.

The four test walls did not meet the specified safety margins and hence would normally be classified as unsafe. Wall 1 was built to a standard whereby it would marginally meet the required standards, whereas wall 2 was purposefully made to be especially slender. Walls 3 and 4 had a low build quality with a high percentage of internal voids that consequently lowered the overall wall density.

Table 2 shows the theoretical peak surcharge loads that could be applied before failure. The walls were assumed to behave monolithically and Coulomb's theory was used to calculate the active pressures. In addition, Table 2 gives the peak surcharge loads observed during testing. Considering the simplicity of the theoretical model, the agreement is reasonable. However, this comparison highlights the need for a model that incorporates the deformations associated with drystone walls, as these are critical to stability. During the tests, failure did not occur at the instant peak loading was reached; instead it was possible to maintain the applied load while further deformations occurred. This indicates that, while it is possible to maintain the peak load, the observed deformations are not causing instability but instead represent a rearrangement of wall geometry to adapt to the applied load.

What should also be noted from Table 2 is that the general material properties cannot be used to determine wall behaviour. Walls 1, 2 and 3 all displayed substantially different

profiles prior to collapse despite being tested using largely the same procedures. To understand these mechanisms, the internal configurations of the wall need to be known and their effects understood.

Through examining the build process of each wall and observing the subsequent behaviour during testing, it is possible to ascertain some of the key points that can determine the way a wall will respond to applied loads. For example, build quality, age, weathering and block geometry are not factors that would generally be considered when calculating structural stability, but these tests have proved that they are critical to wall behaviour.

For a wall of poor build quality, it is likely that individual blocks will be less well supported either through careful seating or using strong and carefully placed pins, and these stones will then have more freedom to move and rotate. Over time, weathering and erosion of a well-built wall can have a similar result. If the geometry of individual blocks is such that they are relatively small and more rounded (as in walls 3 and 4), then this rotational freedom is further encouraged. As resistance to sliding is generally very high for most walling materials, it is rotation rather than translation that causes the development of bulges and subsequent bursting failures.

The main aim of the numerical modelling being conducted at Southampton University is to quantify the effects on wall behaviour of variations in stone shape, material properties and construction quality in terms of degrees of interlocking and wall voidage. In order to provide a greater insight into the failure mechanisms of drystone walls, three-dimensional discrete element models are being developed using the potential particle modelling technique (Harkness, 2009). The modelled stones are created in situ, automatically generating an interlocked structure in which adjacent stones have mating surfaces. The particulate model is coupled to a continuum model of the backfill. A surcharge will be applied, as in the real tests, via a displacement-controlled plate on the top of the backfill material.

9. CONCLUSIONS

Testing methods have been refined over the course of this project, tailoring both the instrumentation and the test procedure for greatest effect. This has culminated in the induction of stable bulging within the test series, replicating the behaviour found in many in situ walls. The phenomenon has been linked both to overall build quality, voidage and geometry, and to the shape of the individual stones and their ability to rotate.

One of the goals of this research is to provide enough data to allow more accurate assessments of existing drystone structures.

	Wall 1	Wall 2	Wall 3	Wall 4
Base thickness: m	0·60	0·50	0·60	0·65
Wall height: m	2·5	2·5	2·5	2·5
External batter: deg	6·8	4·6	6·8	8·0
Internal batter: deg	0	0	0	0
Wall material density: kN/m^3	24·5	24·5	24·5	24·5
Backfill density: kN/m^3	18	18	18	18
Wall friction angle: deg	45	45	45	45
Wall voidage: %	28	23	46	44
Backfill friction angle: deg	39	39	39	39
Backfill height: m	2·2	2·2	2·2	2·2
Initial sliding safety factor	2·09	1·93	1·57	1·72
Initial overturning safety factor	1·95	1·55	1·63	1·87
Predicted surcharge for failure: kN	130	53	87	132
Observed peak surcharge: kN	110	75	80	84

Table 2. Safety factors for test walls

To do this, the theories and analysis techniques that are used must account for the fact that walls may deform without necessarily becoming unsafe. This goal can only be reached given a more thorough understanding of the internal wall mechanisms of load transfer and deformation, many of which have been uncovered by these full-scale tests.

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