LARGE SCALE TESTING OF DRYSTONE RETAINING STRUCTURES

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ABSTRACT
A series of full scale drystone walls have been constructed and tested to destruction as part of an EPSRC funded project at the University of Bath, UK. The properties of each wall have been varied to allow the investigation of various behavioural aspects associated with drystone structures. In particular, the phenomenon of bulging has been successfully recreated. The development and implementation of the instrumentation used to record the tests is described, as well the testing procedure itself.

KEYWORDS: retaining walls, masonry & brickwork, geotechnical engineering.

INTRODUCTION
Drystone technology is an ancient form of construction which relies for its integrity on careful construction giving an appropriate degree of overlap between stones, which are held in position through interlock and friction. Used extensively around the world wherever suitable building material is to be found, the technique is most commonly used for boundary walls, but is also used for earth-retaining structures. The stone is generally used as it comes, either as it is broken from quarrying, or simply picked up from fields, though some minimal shaping may be applied to make a piece fit in a particular position. The aim of the masons is to select a stone and place it in an appropriate position straight away, with as little trial and error as possible. Together with the need to ensure appropriate overlaps, and the challenge of maintaining stability if the stone does not tend to have parallel faces, this requires considerable skill on the part of the masons.

In the UK alone there are estimated to be 9000 km of drystone retaining structures lining the road and rail network [1], mostly dating to the 19th and 20th centuries. Though poorly constructed walls presumably collapsed shortly after their construction, the majority of walls have remained perfectly stable over decades of usually steadily increasing loading and weathering of the constituent stone. However, many walls have deformed or bulged and are regarded as potentially unstable. Because little guidance is currently available to assist engineers in the assessment of these structures [2], they are often replaced, at great cost. They are very rarely rebuilt in drystone, as the dimensions required by current design practice make this substantially more expensive than a concrete replacement. It has been estimated that the total replacement cost for the drystone walls lining the UK's highways would be over £10 billion [3]. Indeed, internationally accepted design practice would deem most existing structures to be inadequate.

There is therefore a clear requirement to have means of assessing existing structures that is realistic. There are substantial difficulties in obtaining information about individual walls, especially their effective thickness and backfill properties, but there is also considerable uncertainty regarding the appropriateness of current design methods for such structures, and research has been carried out at the Universities of Bath and Southampton to address this. The main focus has been on model and full scale testing linked to advanced numerical modeling. However valuable such computational techniques are for research, they are not suitable for routine work by local engineers around the world, who simply do not have the appropriate expertise and resources. A part of the work at Bath has therefore been to develop a simple computer program, which can be distributed freely and used easily to explore the stability of drystone retaining structures.

DRYSTONE CONSTRUCTION

If the nature of the stone allows it, drystone walls are typically built in horizontal layers or 'courses', with each course ideally consisting of stones of a 'uniform' thickness, retaining a straight and level appearance. Walls usually consist of a tightly-packed outer face and a core of smaller random material packed behind, sometimes followed by a more roughly built inner face (fig. 1a). 'Tie-stones' span from the outer to the inner face or into the backfill, binding the wall together (fig 1b). Coping stones can act in a similar manner, spanning the entire width of the wall at the crest (fig. 1c), whilst their greater concentrated mass stabilises the stones in the upper levels of the wall.

Figure 1: Typical wall details: a) face and fill; b) through stones; c) coping stones

It is usually necessary to wedge small pieces of rock known as pins underneath many stones to prevent them rocking, though these will eventually result in deformation of the structure as their small size allows them to weather away more quickly than the larger main stones. Pins are often used extensively to tilt stones to make the face appear more even and assist in drainage. A tightly constructed and planar appearance can therefore conceal a construction which will weather and deform relatively quickly. Though the timescale may still be in years or decades, depending on the quality of rock used for the pins, this is quick relative to the normal lifespan of drystone structures.

The overall density achieved in drystone wall construction varies considerably with the skill of the builders and the speed of construction. Experiments at the University of Bath have demonstrated a range of voidage from 20% to 40% to be possible with the same ideal stone, and larger variation is probable given the range of stone types that can be used. As well as reducing the weight of the wall, to which its ability to resist earth pressures is directly related, increased voidage considerably increases the deformability of the wall. Whilst ductility is in general good, as it allows differential settlement to be accommodated, concentrated loadings to be distributed, and weak areas of construction to be unloaded, excessive deformation can result in a geometry that is no longer stable. Assessing the consequences of deformation has therefore been an important goal in the development of the software reported here.

PREVIOUS WORK

Relevant data regarding drystone retaining wall structures is sparse. The largest reported tests to date were conducted in 1834 by Lieutenant-General Sir John Burgoyne, who constructed four full size test walls in Dun Laoghaire, Ireland. Each wall was built using the same overall volume of square cut granite blocks, but arranged in different sections (Fig 2). Testing consisted of backfilling each wall until the full retention height (6 metres) was achieved, unless premature collapse occurred.

From this work Burgoyne proved that wall geometry has a substantial impact on overall stability, although the use of highly worked granite blocks perhaps caused the walls to act more monolithically than would otherwise be observed in more traditional drystone structures. Regardless, his findings and observations remain the basis for the validation of almost all of the numerical studies carried out to date on drystone retaining walls, despite consisting solely of dimensional measurements and visual observations reported 19 years after the tests [4].

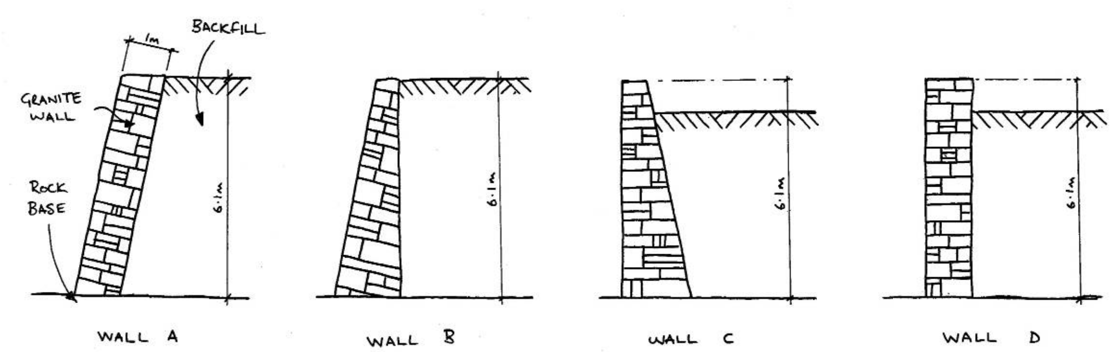

Figure 2: Burgoyne's test wall geometries

Work of such a physical nature was not conducted again until 2005, when a French engineering team led by Jean-Claude Morel and Boris Villemus built and tested five large scale test walls at ENTPE, Lyon [5]. The walls were of various sizes ranging from 2m – 4.25m high, up to 1.8m thick and between 2m and 3m long. Each wall was subjected to hydrostatic forces via a PVC-lined water filled bag, in order to load the wall using purely horizontal pressures that could be precisely known at all times.

The main aim of Morel and Villemus' study was to identify the internal failure angle within the walls at yield. As each test wall was only a short section (2-3m long), the wall ends were left exposed and were considered to reveal the internal actions throughout the entire length. The tests could not proceed to collapse because of the way pressure was applied to them, but monitoring of the end faces allowed relative movements within the wall to be measured as yielding took place. The angles of the resulting shearing surfaces to the horizontal were related to the pattern of construction of the walls.

TEST SETUP

Each of the tests described in this paper were carried out consecutively in a unique outdoor test laboratory situated on the University of Bath campus. To avoid the issue of end effects each wall was required to have a significant width/height ratio. Wall spans of 12 metres were chosen, having a height of 2.5 metres through the central test area. This includes coping stones which constitute the top 300mm.The central 4 metres of each wall rests upon a mechanically jacked platform, which has 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 10mm/min. In addition, a steel frame is erected over the central portion of each wall, from which a 200 kN capacity hydraulic jack is suspended, allowing a localised surcharge to be applied through a loading plate onto the backfill (fig. 3).

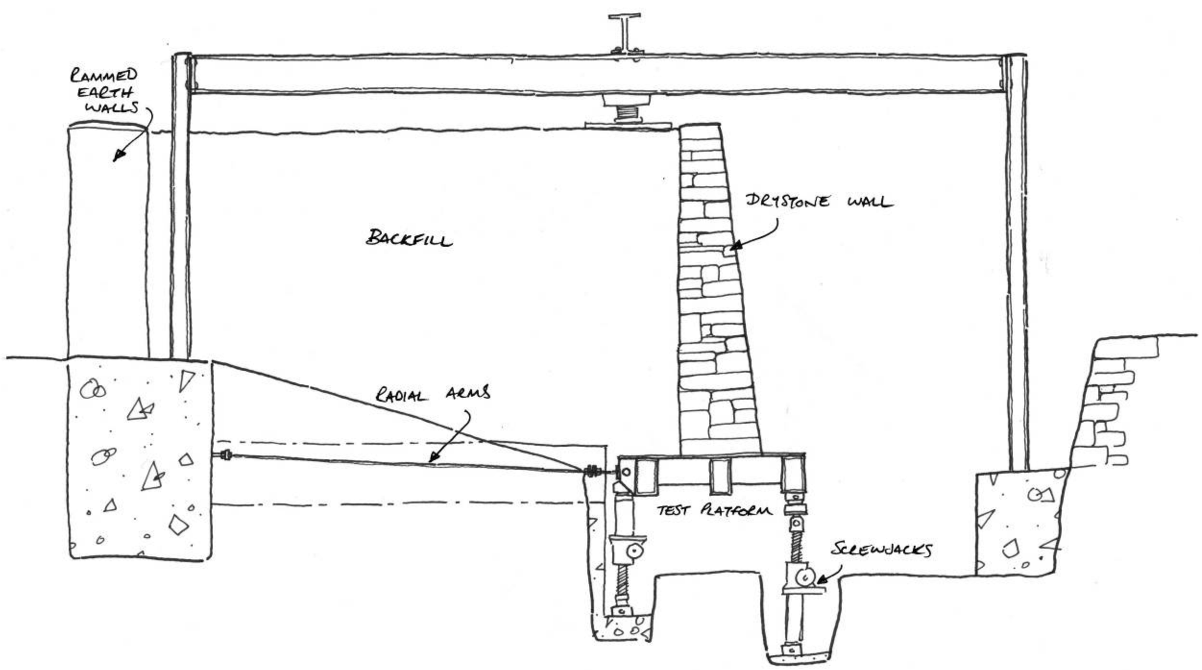

Figure 3: Diagram of test setup

Due to the backfill pressures and surcharging, each test involves significant lateral forces which if transferred to the jacks could cause considerable damage. To avoid this, the screwjacks are pinned at each end to allow only axial forces, with steel bars anchoring the platform to a large concrete block to resist the lateral loads. An advantage to this system is that this 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 was an undressed Cotswold limestone provided by the Natural Stone Market Ltd., requiring approximately 30 Tonnes for each test. Limestone quarried from this region generally comes in two varieties, which can be identified by their colour - either grey or a lighter, creamier colour. The grey limestone is generally considered to be much more durable and so has been used throughout this project.

The retained material for each test is a 14mm single sized aggregate, requiring 100 Tonnes to completely backfill the wall and ensure that any failure planes which develop are not impeded by the test area's boundary walls. This particular backfill has been chosen to ensure that the retained material is completely free draining, allowing no build up of pore water pressures which would cause complications when attempting to analyse wall behaviour, because to some extent the actual pore pressure distribution would inevitably be unknown. Also, capillary tension in finer grained soils would undoubtedly reduce earth pressures, but to an extent which 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 numerical work conducted at Southampton University than by practical testing.

INSTRUMENTATION

To ensure as much data as possible are gathered from each test, a combination of techniques were implemented. As previously mentioned, load cells monitor the forces acting on the platform. Data-logging records readings at appropriate intervals throughout the different stages of construction and testing.

The first two tests also utilised small load cells within the backfill. These load cells were sandwiched between 100 x 100mm steel plates and placed at critical locations within the backfill, orientated to record either horizontal or vertical pressures (fig. 4a). The aim was to use this 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; 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 the wall behaviour.

It is necessary to monitor both local movements of the gravel and the distribution of earth pressures within the backfill. Monitoring of movement is difficult to accomplish during testing, as there are few non-destructive/non-invasive means to achieve accurate results. For the first two tests, several layers of steel ball bearings were laid in grids within the backfill, with each ball carefully placed and surveyed into position using a reflectorless Total Station. After failure of the wall, the ball bearings were carefully recovered using metal detection equipment, and again their positions recorded, thus revealing any movements and aiding identification of failure planes within the backfill.

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 was reached or an obstruction was found, such as kinks caused by developing shear planes. With similar data 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 (fig. 4b).

Figure 4: Instrumentation: a) backfill load cells; b) inclinometer tubes

To monitor the wall face itself, a combination of techniques have been used. Around 350 small targets are scored into each wall face in vertical lines. The points are relocated throughout testing using a reflectorless Total Station to an accuracy of better than +/- 2mm. Although a slow process, with each round of readings taking 15-20 minutes, the nature of the tests allows sufficient time for the Total Station to be used at any point up to the final moments of collapse. Although the final failure of the wall cannot be captured using this technique, it is still very useful for examining bulge formation and the movements prior to failure.

The use of transducers is particularly important in capturing the final moments of each test, as surveying and photographic techniques cannot be relied on to capture the critical moments. A series of draw-wire transducers were used with sacrificial lengths of wire between the instrumentation and the wall, removing the instrumentation from the collapse zone and avoiding damage. Up to 25 transducers have been 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 the movements occurred.

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 permanent mounting points placed 500mm 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 of bulges, cracks, movements etc. A fourth camera was used to monitor targets mounted onto the wall face. The images were then analysed at Southampton University using particle image velocimetry (PIV) techniques to accurately determine the monitored block movements and rotations (+/- 0.1mm).

TEST PROCEEDINGS

The first wall was built in June 2007, constructed over 5 days to a high standard, having tightly packed faces with a well finished appearance (fig. 5a). The overall thickness ranged from

600mm at the base, with a battered front face tapering the wall to 400mm at the coping. Through-stones were incorporated at several levels following standard walling practice.

Figure 5: Test wall 1: a) post construction; b) prior to failure

The backfill was placed in layers of 300mm and compacted using a 1kN vibration-plate compactor. Through plate loading tests, the angle of friction of the fill was found to be 50.1^o, which is much higher than most in-situ retained fills. The fill was placed until a height of 2.2m was attained – 300mm below the crest of the wall and at the level of the base of the coping stones.

Over a period of a month, the wall was monitored to determine if any settlement occurred post construction. After it was determined that no movements were forthcoming, the wall was tested over the course of five days. The first day consisted solely of a 40mm raise of the mechanical platform. The angle of friction mobilised in vertical shearing between the backfill and the back face of the wall has a large influence on the stability of the wall, but is very difficult to measure in practice, as it is a function of the relative movement as well as the intrinsic properties of the materials. Behind in-situ walls, the gradual settlement of the backfill may cause this friction to reach its maximum possible value, equal to the full angle of friction of the fill. The procedure of raising the wall relative to the backfill reproduces this relative motion so that a value can be assumed in the analysis with some confidence.

Following this initial movement, a combination of surcharging and forward rotation of the platform was carried out. A rotation of 3.75^o was achieved by lowering the front jacks a total of 75mm, imitating the effects of local bearing settlement under the toe. The surcharging was initially at a distance of 500mm from the back face of the wall, applied via a plate 400mm x 400mm in size. This was later moved to 1m from the back face as it was found that the initial position was too close to the wall, so affecting only its upper portion. To compensate for the additional distance, a larger plate of 500mm x 600mm was used, allowing larger loads to be applied. The loading reached 110kN, to mimic heavily loaded vehicles passing over the backfill.

The first wall eventually failed via toppling. Although there was deformation in addition to a linear rotation, the wall generally behaved monolithically (fig. 6a). The toppling failure was encouraged by both the initial surcharging near to the coping, and also by the rotation of the platform. The monolithic behaviour was due in large to the tight-knit construction, as the meticulous construction process ensured that each block was placed securely, with no rocking or movement possible.

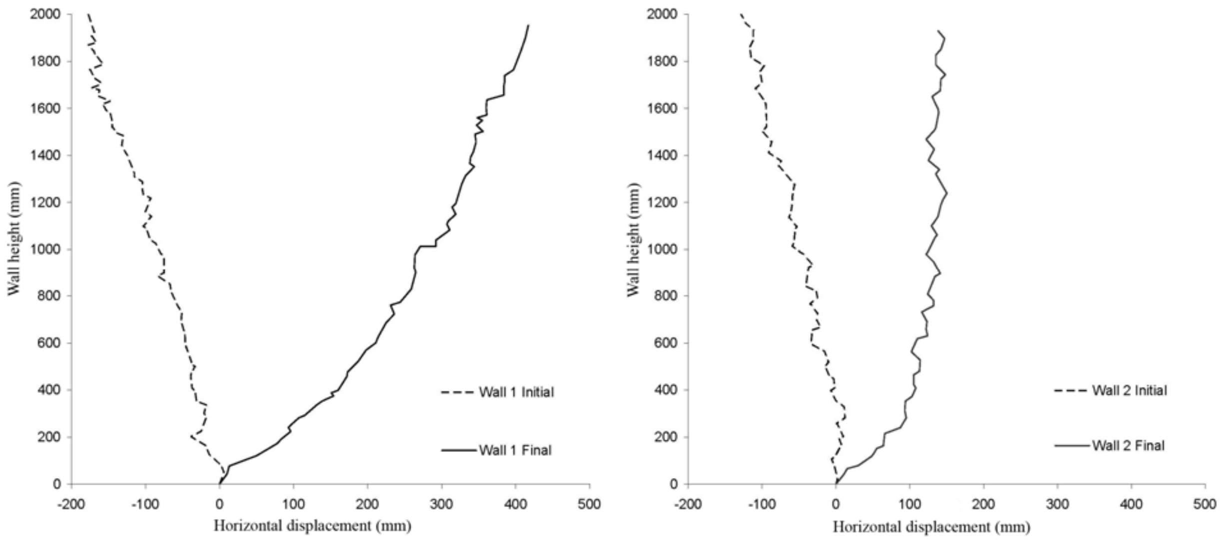

Figure 6: Surveyed movements: a) test wall 1; b) test wall 2

In an attempt to instil flexibility and encourage bulging during testing, the second wall was built with a sectional profile 100mm thinner throughout. In addition, the backfill was placed uncompacted, giving an angle of friction of 41^{o}, which is much closer to that of typical retained fills. The build time was shorter, reflecting a slightly less precise build, incorporating less interlock and allowing more individual block movement.

To maintain continuity between each wall, the second test was in general identical to the first test procedure, but the platform rotation was not applied as it was seen to be mainly promoting a toppling failure. The surcharge load was applied after the initial platform raise, however only the larger plate was applied, at a distance of 1m from the back face of the wall.

Failure was again via toppling, but prior to collapse the wall profile was far from linear. The lower half of the wall had bulged out, with the upper half retaining its integrity and form above this. From figure 6, the two walls can be compared with the differences being apparent. Although the main failure mechanism was toppling, it should be noted that for this wall there was another factor which instigated failure. As the loading progressed, and the deformations occurred, the bulging in the lower courses caused several blocks to move much further than those directly below. Once these blocks had a sufficient overhang, the forces acting through the wall were enough to cause individual stones to further rotate and fall out of place. It is at this point that the local failures of these key stones instigated a total wall collapse.

It was determined that the third wall should examine further the role of individual block rotation. To investigate this process, the wall was built identical in profile to the first wall (600mm wide at the base, tapering to 400mm). This wide cross-section was intended to provide as much stability as possible against overturning failures, allowing other mechanisms to develop. The internal make-up of the wall was much different to the first wall, consisting of a much rougher construction, utilising much smaller unfaced blocks ensuring that as few large, slab-like stones were used as possible. The standard practice of pinning the stones to eliminate movement was largely disregarded, giving each block the ability to rock slightly in place. In short, the wall more closely resembled a much older construction, subjected to several years of weathering and erosion. Indeed, once testing began, a small section of the wall failed, leaving a large hole whilst the rest of the wall remained stable (fig. 7a)

Figure 7: Test wall 3: a) local failure; b) prior to total failure

The testing procedure remained identical, with an initial platform raise followed by a surcharge at a distance of 1m until the collapse mechanism occurred. A stable bulge quickly developed, forming the distinct 'belly bulge' which is commonly found in many in-situ walls (fig. 7b, fig 8a). This bulge continued to develop as the loading progressed, until collapse occurred mainly as a bursting failure.

As the third wall displayed the bulging and eventual failure mechanisms described within the project goals, the fourth wall was used as a control, repeating the third test wall. The wall construction technique was the same, utilising similarly sized blocks in the same manner as before. This wall was also tested in an identical manner to the third wall, and as figure 8 shows, the repeatability of this test was demonstrated.

CONCLUSIONS

The main goals of the physical tests described were to recreate and understand the mechanisms which occur in drystone structures in the field, in particular the phenomenon of bulging. Through the four wall tests described in this paper, various aspects of drystone behaviour have been investigated, culminating with the repeatable recreation of a stable bulge. This has been linked to block rotation, build quality and overall wall geometry.

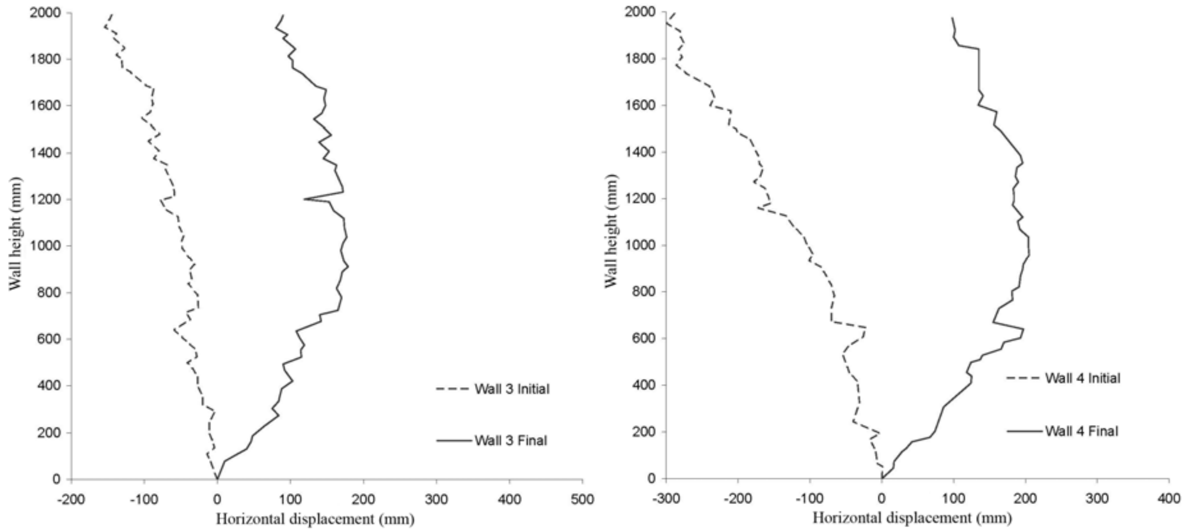

Figure 8: Surveyed movements: a) test wall 3; b) test wall 4

In addition, over the course of the four walls, refinements have been made to both the instrumentation and the test procedures themselves. These improvements have ensured that each test generates the data required to analyse the mechanisms which instigate the actions which occur in each test.

The eventual goal of the project discussed in this paper is to develop guidelines and codes of practice to use in the field. Although still requiring further testing, particularly in the area of stabilising distressed walls, this work represents a large step forwards, giving an understanding of the forces at work and the important behavioural aspects of drystone walls.

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