Large scale testing of drystone retaining walls

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ABSTRACT: There are numerous historic drystone retaining walls in the UK, but the analysis of these economically important walls is restricted by the lack of experimental data. As part of an ongoing investigation into the stability of drystone retaining structures, two full-scale walls have been constructed and tested to collapse. Details of each test are described, including set-up, wall construction, instrumentation and data collection. Initial findings are presented and analysed, along with the continuing aims and goals of the project.

1 INTRODUCTION

Some of the oldest built structures found around the world were constructed using drystone technology, ranging from simple field walls to large earth retaining structures. In the UK alone over 9000 km of drystone retaining walls line the road and rail networks [1], most constructed during the 19th and 20th centuries and many still remaining perfectly stable today. Despite their widespread use, these structures present a challenge for maintenance engineers assessing their stability. Substantially deformed walls, typically bulged, may remain stable for generations whilst other walls displaying only minimal movements can suddenly and inexplicably collapse.

Drystone walling is an empirical form of construction, using best practice methods formed over generations, building on existing examples and past experience. Historically, masons have learnt their trade through apprenticeships. Although presently there are several UK bodies, including The Drystone Walling Association and British Trust for Conservation Volunteers, which promote professional standards of work, there are presently no minimum standards for a practicing waller to adhere to. The task of unifying good practice is made more difficult by regional differences that have developed in a response to variations in local stone and ground conditions.

Whilst the general principles for building structurally safe drystone retaining walls are fairly well understood, the mechanisms behind observed failures of these structures are not. There are several reasons for this, with one of the critical factors being the deficit of scientific tests on full size retaining walls. In fact, a series of full scale tests carried out over 170 years ago remains almost the only data available regarding the failure of ‘real life’ walls to date [3].

The bespoke nature of such walls must also be considered when attempting to understand the causes of a wall failure. In addition to occurrences such as wall or backfill settlement, pore water build up and increasing loading conditions, drystone walls are equally affected by build quality and age. For any given wall, any combination of these factors may affect stability to some degree, again defying any simple means of standardisation.

2 AIM OF STUDY

The overall aim of the research project is to provide in-depth data on failure mechanisms, including bulging, and three-dimensional effects that influence this behaviour. Through the use of a bespoke test rig, two full size drystone retaining walls have been tested to failure and a further two are to be subjected to localised surcharging, backfill settlement or wall subsidence. Actions are to be applied simultaneously or independently, in a controlled manner whilst a range of
instrumentation is used to monitor the walls’ response through to their collapse.

The main aim is to have sufficient data to verify existing models and theories, and assist with the generation of new analysis techniques, more accurate modelling tools and assessment guidelines for drystone structures. The study is funded by the Engineering & Physical Sciences Research Council (EPSRC) and is conducted in collaboration with the University of Southampton. The creation of these guidelines, together with appropriate engineering tools to back them up, could save a substantial part of the estimated £1 billion estimated cost of replacing the walls in the UK alone [2].

3 RELATED RESEARCH

As previously mentioned, physical test data regarding drystone retaining wall structures is sparse. The largest reported tests to date were conducted in 1834 by Lieutenant-General Sir John Burgoyne [3], 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. 1). 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.

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 2 m–4.25 m high, up to 1.8 m thick and between 2 m and 3 m 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 (2–3 m 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.

4 TEST SETUP

4.1 Testing rig

For the purposes of this project, a bespoke test rig was required. The rig required the ability to impose various conditions upon the test walls, necessary to undermine their stability. These conditions, including relative wall settlement, backfill compaction and surcharge loading, mimic the real-life situations to which many drystone walls are subjected.

To simulate settlement of the foundations or the backfill, the ability to raise, lower or tilt the wall is required. To achieve these movements, the wall itself is constructed on a 1.4 m × 4 m steel platform, which is in turn mounted on four 20 tonne mechanical screwjacks. This arrangement can produce vertical movements of up to 400 mm, tilt up to 17° forwards or backwards, as well as intermediate combinations of these movements.

A steel frame spans the central portion of the wall and backfill, under which a hydraulic jack is mounted. This jack allows a patch surcharge to be applied to almost any location behind the wall, simulating additional localised loadings of up to 200 kN (Fig. 2). The walls were designed to be sufficiently long to minimise the edge effects from both induced settlements and surcharge loading on the behaviour of the wall which was a serious limitation of the Burgoyne experiments.

4.2 Instrumentation

To ensure as much data as possible are gathered from each test, a combination of techniques were implemented. Load cells monitor the forces acting on the platform, and the applied patch surcharge. Simple
button-type compression cells are used for the screw-jacks, with S-type tension/compression cells located on the radial arms. Data-logging records readings at appropriate intervals throughout the different stages of construction and testing.

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 purposes of these tests, several layers of steel ball bearings have been laid in grids within the backfill, with each ball carefully placed and surveyed into position using a reflectorless Total Station. The polished surfaces of the balls ensure that the measuring beam is reflected back from a point on the surface in line between the instrument and the centre of the ball, so that a correction for the radius is all that is needed to determine the accurate positions of the centres. After failure of the wall, the ball bearings are carefully recovered using metal detection equipment, and again their positions recorded, thus revealing any movements and aiding identification of failure planes within the backfill.

Backfill pressures are measured using pressure cells buried within the fill. These consist of parallel 100 mm × 100 mm steel plates separated by 2500 N or 5000 N load cells. These are placed within the fill either vertically or horizontally, allowing continuous monitoring of lateral or vertical pressures. These sensors are particularly useful to indicate how the surcharging force is distributed through the backfill. The pressure cells are spread out through the backfill, with the majority concentrated along the centre-line of the wall where the loads are expected to be highest.

To monitor the wall face itself, a combination of techniques have been used. Around 350 small targets have been scored into the wall face in five vertical lines. The points are relocated throughout testing using a reflectorless Total Station. 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.

In addition to the Total Station readings, draw wire transducers were attached to the centre line of the wall. These allow continuous high resolution monitoring of wall face movements during testing, facilitating control of test procedures. The transducers are anchored to a stanchion attached to the loading platform, so that measurements represent deformation of the wall directly, without the need to correct for movement of the platform.

Given the scale of variations within the construction of drystone walls, it is not possible to guarantee that the transducers and load cells will be located on parts of the structure where key events unfold. To ensure detailed recording of mechanisms, this project employs a range of visual tools to supplement the instruments. In front of the test wall face, on a fixed pedestal, are a pair of Nikon D40x digital SLRs, providing stereo photographs of the wall face throughout testing. An additional camera is mounted behind the backfill to record back face and backfill conditions in response to the surcharging.

Southampton University used Particle Image Velocimetry (PIV) to record movements of the centreline of the face to sub-millimetre accuracy. High resolution digital photographs were taken sighting along the line of the wall to targets mounted on brackets attached to a number of individual stones. A computer program was then used to determine changes from one image to the next.

The whole test process has also been captured on a Canon HV20 HD camcorder. This final piece of equipment ensures that the final moments of the test are recorded in a high-definition video format. Additionally, it allows any sudden movements or noises to be recorded, information that might otherwise be missed.

5 EXPERIMENTAL TESTING

5.1 1st test wall

The first test wall was constructed in June 2007, requiring approximately 35 tonnes of limestone. The wall was 2.5 m high, tapering from 600 mm at the base to 400 mm at the coping, with a vertical rear face and a battered front face. This initial wall was constructed with two fair faces, so that the wall could be constructed without the need to be building and backfilling simultaneously. The wall contained three layers of through-stones, at 0.5 m, 1 m, and 1.5 m high. As is standard practice amongst most UK masons, these
through-stones protruded clearly from the face of the wall, fig. 3.

The aggregate backfill was placed in layers of 250 mm–300 mm thick, compacted using a 1 kN vibrating wacker plate. Through laboratory shear box and triaxial testing, also confirmed by an in-situ plate loading test, the angle of friction for this compacted material was found to be 51°. This is very high when compared to standard fills generally found behind drystone walls but this aggregate type was necessary to ensure adequate drainage. The required height of 2.2 m (the height to the bottom of the coping stones) was achieved with no deformations greater than 2 mm recorded during construction.

This wall was tested over the course of three days in July 2007, with a day between each test day to allow any relaxation or further settlement to occur. On the first day the platform was raised up by 20 mm, to ensure full mobilisation of friction against the back of the wall, as would occur during settlement under self weight of a normal fill.

On the second test day, the surcharging rig was used to load the backfill, using a hydraulic jack situated 0.5 m from the back face of the wall along the centre-line. The patch load was applied over a stiffened plate 400 mm × 600 mm. A force of 5 tonnes was reached before loading ceased. It was found that due to the high stiffness of the aggregate, this surcharge force was too close to the back face of the wall, resulting in deformations mainly towards the top of the wall. The jack was then reset to 1 m from the back face, and a bigger, 600 × 400 mm, surcharging plate installed.

To model realistic wall movements, the wall was rotated forwards, imitating ground settlement beneath the toe of the wall. Rotation of 2.5° was induced, causing the front face of the wall to appear almost vertical. At this point, several readings were taken using the Total Station and further experimentation was delayed.

The final day of testing involved surcharge loading, 1 m behind the initial wall position. The plate was gradually loaded up to 11 tonnes, at which point the load levelled off as the failure wedge within the backfill began to shear. Substantial movements were recorded, with the wall coping overhanging the toe by 500 mm prior to failure (fig. 4).

As wall movements became more pronounced, the load that could be sustained from the surcharging frame steadily dropped, until the final failure mechanism was produced. The wall failed through toppling, having undergone a significant three-dimensional distortion.

5.2 2nd test wall

The second wall was constructed in September 2007, re-using the same material from the first wall. In an attempt to create more pronounced deformations, the wall was reduced in section to 500 mm at the base, tapering to 300 mm at the coping level.

This wall was of a much looser construction, especially on the rear face of the wall. Due to the comparatively unstable nature of this wall, backfill was introduced as the wall was built, ensuring stability during construction.

In an attempt to model real conditions more precisely, the backfill was not compacted for this second test. Plate loading tests indicated the angle of friction to be about 41°, which is much closer to the material properties found behind most existing retaining walls. In a similar manner to the first test, the second
Figure 5. Test wall 2 (prior to failure).

The wall retained 2.2 m of backfill, with the top 300 mm of coping left uncovered.

Testing was conducted in October 2007, over three consecutive days. Test procedures similar to those of the first tests were used, in order that a basis for comparison of the data generated would exist.

To this end, the platform was initially raised 40 mm in 5 mm increments. Loads on the platform were carefully monitored until a stable plateau was reached, confirming full mobilisation of backfill friction on the back of the wall.

The wall was then slowly surcharged over the course of the next three days. The same plate was employed in the same location (600 mm × 400 mm at a distance of 1 m from the back of the wall). Peak loads of 7.5 tonnes were reached, accompanied by wall deformation, block sliding and clearly audible movements within the wall. In response to backfill mobilisation and initial surcharging a distinct bulge developed in the lower section of the wall prior to collapse.

Failure finally occurred after the peak load had dropped from 7.5 tonnes to 4 tonnes. The final failure mode was again overturning, however more pronounced sliding and rotation was noted in the lower courses. Overall deformations prior to collapse were much reduced from the first test wall; however the area of wall that failed was significantly larger.

5.3 Materials testing

Following tests an intact area of wall was measured, weighed and removed to ascertain the void percentages and densities. A void percentage of 28% was measured, giving an average wall density of 17.9 kN/m³.

Two further test sections, each of dimensions 0.5 m × 0.5 m × 0.7 m, were constructed by the masons to investigate this further. One was built to the highest standard that could be achieved with this stone, but using fill pieces of a normal size. The second was built to a functional standard, but to represent a structure built quickly without taking care to achieve an optimal fit between stones. Upon disassembly, the void percentages for these two volumes were 21% and 37%, with overall densities of 19.7 kNm⁻³ and 15.6 kNm⁻³ respectively.

Other material tests are scheduled to be carried out within the time frame of this project, including laboratory investigations into the interface between the wall and the backfill, and sliding between wall beds.

6 ANALYSIS

As the backfill was raised to the full height, and the aggregate fully mobilised against the back of the wall, bulging occurred; indeed the second wall began bulging even before the platform was raised to mobilise wall friction. Bulging consisted of movements less than 5 mm, undetectable to the naked eye on a structure of this scale, but easily detected by the instrumentation.

These movements hint at the mechanisms within the wall at these locations, and are probably due to the wall adjusting to support the initial stresses through internal block sliding and small rotations.

An obvious difference between walls 1 and 2 was the amount of deformation prior to failure. The first test wall moved almost 350 mm further at the coping (see fig. 6). Although the thickness of the wall is understood to be one of the critical reasons for this difference, several other unforeseen reasons present themselves.

After failure of the first wall, the three-dimensional nature of the test became apparent. Due to friction between courses, a certain amount of tension had been generated along the front face of the wall, allowing the wing walls to help retain the failing central section.

Conversely, the second wall was of a much looser construction, and during testing several vertical joints appeared throughout the wall. These joints limited any of the stabilising effects from the wing walls, and hence smaller deformations were visible before the failure conditions were reached.

The compaction of the backfill was also a critical factor in determining the failure mode. Certainly the high stiffness of the aggregate in the first test caused the surcharge load to be distributed over a wider area.
than in the second test, leading to much higher loads before slip planes were formed. This also had the effect of transmitting lateral stress from the surcharge much higher up the wall than originally desired, encouraging toppling rather than bulging.

After dismantling the first wall, the ball bearings within the fill were unearthed, located with the Total Station, and then analysed for changes in position (fig. 7). The stiffness of the fill allowed very little movement away from the wall face; indeed the only movements which were recorded coincided with a failure wedge originating at the rear edge of the surcharging plate.

The other significant impact that the angle of backfill friction has upon the failure is the magnitude of active pressure exerted on the wall. The first test with the higher friction angle has the added benefit of a much greater vertical component upon the wall, whereas the lower friction angle has a relatively greater horizontal component.

It is difficult to be certain exactly how much of the backfill friction has upon the failure is the magnitude of active pressure exerted on the wall. The first test with the higher friction angle has the added benefit of a much greater vertical component upon the wall, whereas the lower friction angle has a relatively greater horizontal component.

It is difficult to be certain exactly how much of the backfill friction is mobilised against the wall. However by initially raising the platform until the platform loads peak, and given the roughness of the back of the walls, it is reasonable to assume that it is almost fully activated.

Both walls eventually failed via toppling. This was to be expected for the first wall, as the combined effects of the platform rotation and early loading near the top of the wall succeeded in pushing the centre of gravity very far forwards.

Although the second wall also toppled, the manner in which it initiated this failure was different. Block sliding through the lower courses occurred to the point where a significant portion of some blocks were overhanging the course below. This in turn allowed an overhanging block to begin to rotate. In the final phases of test two, specific blocks near the base of the wall could be identified in both the photography and the video which had clearly slid forwards over the blocks beneath them.

As the wall is pushed further forwards, the resultant thrust line will similarly move forwards until it is passing through the ends of these overhanging blocks. The overturning of the wall was then triggered by the rotation of one of these overhanging blocks. At this point the thrust line will fall outside of the wall at this level and toppling would occur. The movement of this rotation along the course of stones could be seen clearly in the video and in the sequence of stereo photographs.

7 CONCLUSIONS

Although only two of the four programmed tests have as yet been carried out, substantial results have already been generated. Each wall has been constructed to emulate real walls as closely as possible, with carefully limited changes between successive wall tests to facilitate meaningful comparison of the results.

An initial aim was to create the distinctive ‘belly bulges’ found in many walls today, and both test walls showed some bulging deformation. The reasons why these particular failure mechanisms formed are now better understood, highlighting issues that were not previously considered.
Though these first two wall tests do not provide data applicable to a wide range of wall failures, the mechanisms observed most probably occur, and every dry stone retaining wall that is still standing must necessarily be resisting failure by these mechanisms. The tests also provide detailed observations of the behaviour of real walls which can be used to verify the results of numerical modelling, including that currently being carried out at Southampton University. This numerical modelling, once verified, will be used to investigate other aspects that cannot be safely or effectively investigated by full scale testing, such as the effect of major changes in pore water pressures. The tests therefore represent a clear step towards a greater understanding of these structures, and also towards the eventual goal of formulating guidelines and accurate analysis techniques. At the very least these tests prove beyond all doubt that even walls such as these, designed to deform and fail, are incredibly resilient and able to adapt to significant loads with little affect on overall long term stability.

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