Citation for published version:
McCombie, PF, Mundell, C, Heath, A & Walker, P 2012, 'Drystone retaining walls: ductile engineering structures with tensile strength', Engineering Structures, vol. 45, JEST4017, pp. 238-243.
https://doi.org/10.1016/j.engstruct.2012.06.046

DOI:
10.1016/j.engstruct.2012.06.046

Publication date:
2012

Document Version
Peer reviewed version

NOTICE: this is the author's version of a work that was accepted for publication in Engineering Structures. Changes resulting from the publishing process, such as peer review, editing, corrections, structural formatting, and other quality control mechanisms may not be reflected in this document. Changes may have been made to this work since it was submitted for publication. A definitive version was published in Engineering Structures, vol 45, 2012, DOI 10.1016/j.engstruct.2012.06.046

Engineering Structures

Drystone retaining walls: ductile engineering structures with tensile strength

Authors:

Dr. Paul McCombie[1], BA MSc PhD CEng FICE FRSA

P.F.McCombie@bath.ac.uk Tel (+44) 1225 826629 Fax (+44) 1225 386691

(Corresponding author)

Dr. Chris Mundell[2], MEng(Hons) PhD

Chris.Mundell@atkinsglobal.com

Dr. Andrew Heath[1], BSc(Eng) MS PhD

A.Heath@bath.ac.uk

Professor Peter Walker[1], BSc PhD MIEAust CPEng MICE

P.Walker@bath.ac.uk

Contact details:

[1]BRE Centre for Innovative Construction Materials, Department of Architecture and Civil Engineering, University of Bath, Bath BA2 7AY, UK

[2]Atkins, The Hub, 500 Park Avenue, Aztec West, Bristol, BS32 4RZ , UK

Keywords:

Retaining walls; drystone; ductility; assessment.

Abstract

Drystone retaining walls are sustainable engineering structures constructed with locally obtained natural stone. They were commonly built with very slender profiles compared with modern mass-masonry structures, leading to a common belief among engineers that they have very low margins of safety. These structures remain critical to the transport infrastructure in many parts of the world, and have proven to be very durable, yet very few new drystone retaining walls are built, and walls which do fail are usually replaced with concrete constructions. We show that these walls are ductile even though their components are brittle, and in having tensile strength through the interlocking of their stones, even though they are assembled without any cohesive material such as mortar. These properties are critical to a proper understanding of their behaviour and durability. Full-scale testing of five drystone retaining walls has shown that bulging, most commonly regarded as a sign of incipient failure, begins as a ductile adaptation of the geometry to the loads imposed on it. Localised bulging can be a consequence of small defects in construction or foundation conditions, or concentrated loading, and may be sustained indefinitely in a wall which is in general well-constructed. These insights into the behaviour of walls allow the design of new walls which use materials efficiently, and enable existing walls to be kept in service, and may inspire new ways of achieving ductility in engineering materials.

1. Introduction

Drystone walling is an ancient technique in which locally sourced stones are carefully assembled by hand following traditional practices to construct field walls, buildings, and earth-retaining structures. Until recently the behaviour of these structures was not well understood. Typical retaining walls were built with a much more slender profile than modern mass-masonry structures, and it is a common belief among engineers that they have very low margins of safety [1], so structures with a distorted profile are presumed to be 'distressed' even though it is known that they can remain stable for decades. Drystone retaining walls remain critical to the transport infrastructure in many parts of the world, as well as shaping the ground to form terraces for housing and agriculture. Even though they are sustainable, using locally sourced unmanufactured materials and skilled labour, and are aesthetically pleasing, when a drystone wall reaches the end of its life it is usually replaced with concrete, and very few new drystone retaining walls are built.

Despite the importance of drystone retaining walls to transport infrastructure, very little research has been undertaken. Four walls were built and tested to destruction in Ireland in 1834 [2], while a further four walls retaining water-filled bags were loaded to initial yield in France [3]. Extensive numerical analysis has been done on the Irish walls using the Finite Element Method [4] and the Discrete Element Method [5,6], even though relatively little information on their properties is known and the walls were not instrumented during testing. This previous work demonstrated the impracticality of using such methods other than as experimental tools, owing to the difficulty and complexity of making a realistic model. Nevertheless, it is useful to have such experimental tools, and some of the equilibrium analyses carried out for comparison with the numerical modelling results gave useful insights into wall behaviour. In particular, the significance of rotation of individual blocks has been highlighted, which does not occur in conventional mass retaining walls [6].

2. Full-scale testing

2.1 Test configuration

To address a pressing need for detailed observations of the behaviour of real walls, five test walls were built as summarised in Table 1, each 2.5m high and over 12m long, with a central test section built on a steel platform which could be raised, lowered and tilted under fine control using motorised screw jacks. Hinged transition pieces supported the walls between the platform and the ground to either side, from where wing walls tapered to ground level. The general arrangement of the site is shown in schematic section and photograph in Figure 1. The walls were backfilled with gravel to a height of 2.2m, and the platform raised so that the friction of the backfill on the back of the wall produced a downdrag, as would normally occur during settlement of the backfill and foundation soil. This force helps the structure to resist the overturning action from the earth pressure on the back of the wall. The first four walls were built of limestone, whilst the test section of the fifth was built of a weak slate. Each structure was instrumented, with load cells on the platform itself, and displacement monitoring using draw-wire transducers, photogrammetry, and reflectorless surveying techniques. Because of the large stones resulting in stress concentrations in the wall and backfill behind the wall, information from the pressure cells which were installed was of limited use. Deformations observed during backfilling were small in relation to the height of the walls.

2.2 Testing to destruction

Following the raising of the platform, a load was applied to the surface of the gravel behind the top of each wall via a 600mm square plate, as seen in Figure 1. The walls began to yield when loads of between 6 and 11 tonne (60 and 110 kN) were applied to this plate, with final collapse occurring only after considerable deformation, as indicated in Table 1. Prior to the final collapse, the load could be removed and all movement of the structures would cease. The walls are shown at their maximum deformations prior to collapse in Figure 2. The targets attached to the wall face were horizontal prior to the application of load to the top of the backfill, and hence indicate the rotation of stones at the face. The horizontal sliding of the slate in Wall 5 is conspicuous, as is the extreme inclination of Wall 1, which was supported by tension along the face of the wall, which resulted in a catenary in plan extending from the lightly loaded section to one side which carried only pressure due to the self-weight of the backfill, through the central test section, to the lightly loaded section on the other side. This is discussed further below. Careful observations were made during the loading of the walls, in addition to the measurements, by multiple still cameras, a continuously recording high definition video camera, and the team of investigators. The observed deformations were seen to arise primarily from the accumulated effects of small rotations of individual stones, but as displacements increased sliding of stone on stone became important. The section of wall carrying the applied load bulged out relative to the adjacent sections, stretching as it did so.

2.3 Observations

The first test wall was constructed following best drystone walling practice, with good bonding – that is, the stones on successive courses overlapped each other throughout, so that if a stone was pushed forwards, it pulled the stones above and below it through a frictional interaction; these stones in turn pulled the stones on either side of the first stone in the same way. Thus a tensile connection was established between the first stone and the stones on either side purely as a consequence of the frictional connection with the stones above and below. This tensile strength is similar to that which develops in a natural fibre rope, which is made up of short strands which transfer load via friction, which is maintained by the lateral compression generated by the twist of the strands. Laboratory tests confirmed that the coefficient of friction for the limestone used is high, which with the weight of wall above any given course produces a strong frictional resistance. Normally in engineering construction a tensile connection is a weak link: if the material is brittle then once the tensile strength is exceeded a failure is inevitable. Ductile behaviour is much preferred, so that rather than simply breaking once a yield load is achieved, the material stretches significantly. This can allow load to be transferred to adjacent elements with spare capacity, or at least give warning that a failure is approaching. The tensile strength arising from frictional interaction is ductile in nature, in that until displacements are so great that stones lose contact with each other completely, the frictional interaction will persist with approximately the same strength.

This tensile strength along the line of the wall provides a mechanism for transferring load from one section of wall to another, so that local overloading does not result in immediate failure. It is not, however, the only means by which the structures behave in a ductile manner and maintain their stability.

Walls 1 and 2 were both built with the stones very tightly packed together (the voidage, or percentage of volume not occupied by stone, is given in table 1). Wall 2 was built to a higher overall density because of its reduced width, making it very slender, and was deliberately constructed with vertical running joints. That is, some of the gaps between the stones were aligned up the height of the wall, disrupting the tensile strength along the length of the wall and allowing the test section to move more freely. The tightly packed stones left little room for internal deformation as the load increased, and the failure mode was principally a forwards rotation of the entire wall over its toe (that is, the front of its base), reflecting conventional assumptions about the behaviour of gravity walls. The wall was unusually slender for a gravity wall, with a base width only 25% of the retained height. This was only possible because of the stabilising effect of the downdrag force on the back of the wall, resisting the overturning moment from the horizontal component of earth pressure. Whilst this force had been induced in a controlled manner by the raising of the wall against the backfill, simulating backfill settlement, it would also have been generated as the wall began to rotate forwards, so lifting up at the back.

Wall 3 was deliberately built much more quickly with a lower density, and a quality of construction which better reflected common historic practice in the field. Modern professional drystone wallers work to very high standards, producing densely built walls with very few running joints and sufficient through-stones, which span from the front to the back of the wall and hold it together. The packing of the stone in Walls 1 and 3 is compared in Figure 3. Wall 3 included running joints, as in wall 2. The lower density permitted noticeable rotation of individual stones as load was applied. For the most part, such rotations were through relatively small angles from a position which was stable during construction to a position which was stable under the applied loading. The consequent small local movement would result in a small local reduction in the applied load, which might result in a transfer of load to adjacent sections of the wall through friction in the fill, or may result in an overall reduction in the load applied to the wall if the upper part of the wall rotates back slightly against the fill (Figure 4). The accumulation of such small changes in geometry resulted in a configuration which increased the favourable loads while reducing the unfavourable loads, and could better resist those loads.

3. Analysis and discussion

This has been explored using the concept of the 'Line of Thrust', defined as the line joining the points of action of the resultant forces between each section of masonry, which has been most commonly used in the assessment of masonry arches[7]. A computer program was written to enable rapid exploration of the effect of changes in geometry, loading and material properties[8,9]. The analysis is simple, taking a cross-section through the wall to be representative of the whole wall, and so all deformations to be in plane strain. The load applied to the plate was assumed to distribute both along the length of the wall and at right angles to the wall, over an area generated by a load spread of 1 horizontal: 2 vertical. The analysis is thus a 2D approximation of a 3D situation. The geometry of each of the test walls was entered into the program based upon the surveyed profiles, and the total forces and moments acting at each level within the wall were calculated using simple Mohr-Coulomb earth pressure theory as used for conventional gravity retaining walls. The results of analysing all five test walls, both on completion of backfilling and just prior to failure, are shown in Figure 5. The cross-sections of the walls are taken from the screen display of the computer program, and show lines of thrust (heavy lines), and the position and assumed spread of the load applied to the surface of the backfill (light lines). The horizontal lines on the cross-sections correspond to the levels at which observations were made during the tests, which were generally at the tops and bottoms of individual stones. Surveying of the topmost section of the wall was obscured by the loading frame, so less detail can be shown. As the load applied to the fill behind the wall was increased, the line of thrust through the base of the wall moved towards the toe. Failure is expected when the line passes outside the base of the wall, emerging from the wall above the toe. In practice the foundation will probably yield significantly before this point is reached, but as the test walls were constructed on a strong steel platform the correspondence between testing and analysis was excellent.

These results show that the deformed shape which was observed to be stable could be demonstrated to be stable by analysis. The exception is the result for Wall 1, which received additional support via the induced tensile strength along its length, allowing it to remain standing with a line of thrust emerging from the face of the wall. Wall 3, and the corresponding analysis, demonstrated that dry-stone walls can have considerable ductility, and that their characteristic bulging deformation may take them to a position of stable equilibrium. The large deformations prior to collapse give warning of walls in distress, a most important consequence of ductility, while modest bulging need not give concern except in poorly built walls. Detailed observations indicated that deformations are only likely to lead to collapse after individual stones have moved so far that they are losing the support of the stones below them, and the degree of movement involved is related to the size and shape of the stones and the density of construction. The more dense the construction, the less the space between the stones which allows them to rotate, whilst stones which are longer and flatter can rotate less than stones which are more spherical or cubic. On the other hand, as rotation becomes more difficult, sliding becomes more likely, so that ductility arises from translational movements resulting in redistribution of load, while frictional shear resistance between stones is more or less maintained. These are important insights for the assessment of deformed structures. Figure 6a shows where stones fell out of the face of Wall 3 just below the bulge; even this significant loss of stone did not lead to immediate collapse, because the stones only fell out because all load was already passing through the adjacent stones, which remained in place.

The fourth test wall was constructed to confirm that the behaviour seen in could be reproduced. Analysis of the first four walls showed the internal friction to be more than sufficient to prevent the wall from sliding forwards on a course of masonry, so the fifth wall was constructed to investigate this type of failure by using slate, with a much lower coefficient of friction between stones. The resulting failure mechanism involved very little rotation, but substantial sliding within the lower courses of the wall (fig 6b), until the wall toppled forwards. Because this failure was triggered by excessive sliding of an individual stone until it tipped over, the final marginally stable geometry was not captured in the data shown in figure 5e, in which the resultant still lies within the base of the wall.

4. Conclusions

The work reported here has demonstrated that drystone retaining walls may be analysed and designed using limit equilibrium methods, and behave in a predictable manner under static loading. This work does not address the dynamic behaviour of drystone structures, which is important in many parts of the world; both numerical and experimental work is needed to address this issue. They are therefore safe to use when designed to have normal margins of safety – these vary from country to country, but may be represented by a factor of 1.5 against sliding, and maintaining the resultant within the central third or two-thirds of the base. For economical design, it is important to take account of the friction mobilised between the wall and the backfill. It has been demonstrated that even poorly constructed walls have considerable ductility and some induced tensile strength, allowing them to adapt to localised loading and weaknesses in the ground. It has been shown that these structures are unusual in that they attain tensile strength despite having no direct tensile connection between elements, and ductility arising from the ability of the geometry to adapt to the loading both locally and globally, rather than through the elasticity of its components.

Acknowledgements

The authors wish to acknowledge the support of EPSRC, and of Bradford Metropolitan District Council; Cornwall County Council; County Surveyors Society; Gloucestershire County Council; Network Rail; and, Wiltshire County Council. The work was carried out in parallel with numerical modelling work at Southampton University by Dr. John Harkness and Prof. William Powrie.

References

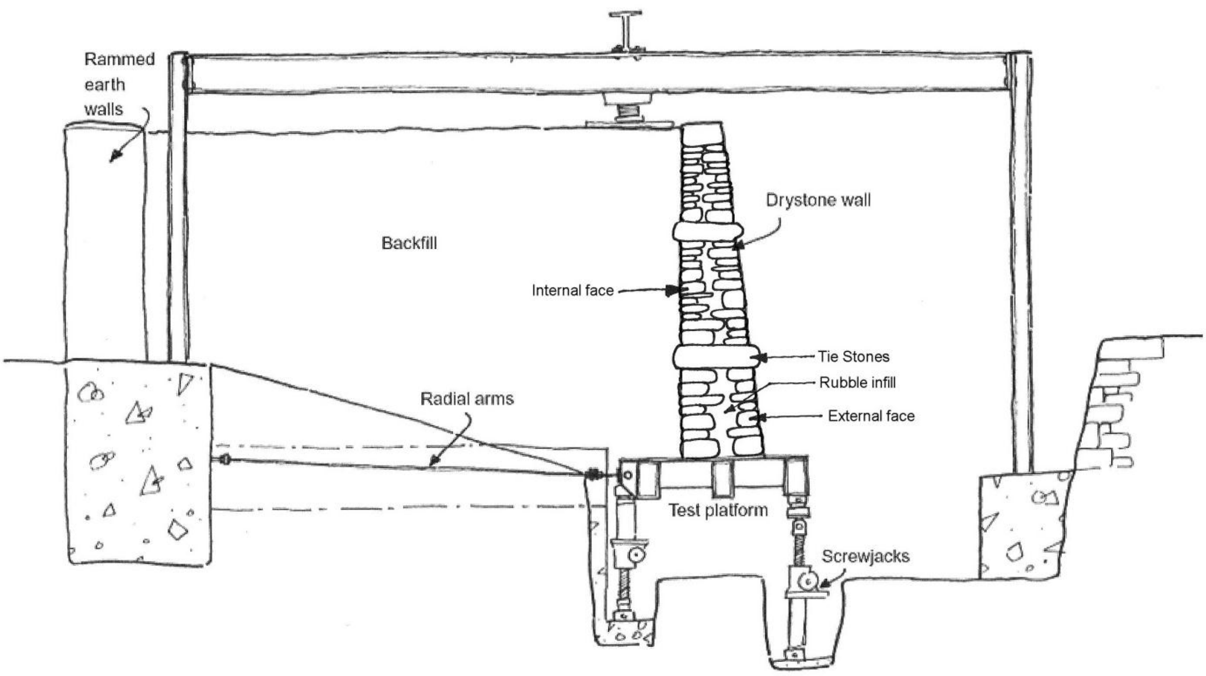
1. O'Reilly, M. P. & Perry, J.. *Drystone retaining walls and their modifications – condition appraisal and remedial treatment*. RP723 CIRIA, London (2008)
2. Burgoyne, J. Revetments or retaining walls. *Corps of Royal Engineers* **3**, 154–159 (1853)
3. Villemus, B., Morel, J. & Boutin, C. Experimental assessment of drystone retaining wall stability on a rigid foundation. *Engineering Structures* **29**, 2124–2132 (2007)
4. Zhang, X., Koutsabeloulis, N.C. , Hope, S. & Pearce A. A finite element analysis for the stability of dry stone masonry retaining walls. *Géotechnique* **54,** 57-60 (2004)
5. Harkness, R.M., Powrie, W., Zhang, X. Brady, K.C. & O'Reilly , M.P. Numerical modeling of full-scale tests on drystone masonry retaining walls. *Géotechnique* **50** 165-179 (2000)
6. Claxton, M. , Hart, R.A. ,McCombie, P.F. & Walker , P.J. Rigid block distinct element modeling of dry stone retaining walls in plane strain. *.J Geotechnical and Geoenvironmental Engineering,* 381-389 (2005)
7. Heyman, J. The stone skeleton. *Int. J. Solids and Structures* **2**, 249-279 (1966)
8. Mundell, C., McCombie, P., Bailey,C., Heath, A. & Walker, P.. Limit-equilibrium assessment of drystone retaining structures. *Proc. Inst. Civil Engineers: Geotechnical Engineering* **162 GE4**, 203-212.

9. McCombie P. & Mundell, C. Drystone retaining wall analysis. *Computer program* http://www.bath.ac.uk/ace/drystone-retaining-wall-analysis/ (2011)

Table 1 Summary of test walls

Wall	1	2	3	4	5
Base thickness (m)	0.6	0.5	0.6	0.7	0.67
Stone	Limestone	Limestone	Limestone	Limestone	Slate
Stone joint friction	37.4^{o}	37.4^{o}	37.4^{o}	37.4^{o}	17.5^{o}
Wall voidage (volume of voids as a percentage of total volume)	28%	23%	46%	44%	30%
Peak applied load (kN)	110	75	80	85	60
Load at collapse (kN)	40	47	45	61	24
Initial backfill height (m)	2.2	2.2	2.2	2.2	2.2
Height at failure (m)	1.9	2.0	1.9	2.0	1.8
Approximate lateral deformation at top of wall during backfilling	<10mm	10mm	15mm	25mm	20mm
Deformation mode during test	Rotation	Rotation	Bulging, rotation	Bulging, rotation	Sliding
Level of maximum deformation	Coping	Coping	1.2m	1.1m	Coping
Displacement relative to toe	425mm	150mm	350mm	250mm	175mm
Failure mode	Topple	Topple	Burst / topple	Burst / topple	Topple

Figures



a) Schematic cross-section through a test wall, and (b) overview of Wall 5 during testing.

Figure 1 Test arrangement

a) Wall 1	b) Wall 2	c) Wall 3	d) Wall 4	e) Wall 5

Images of test walls at their last stable position.

Figure 2 Test walls immediately before collapse

a) Wall 1 Construction

b) Wall 3 Construction

c) Wall 1 Elevation

d) Wall 3 Elevation

Figure 3 Variations in test wall construction

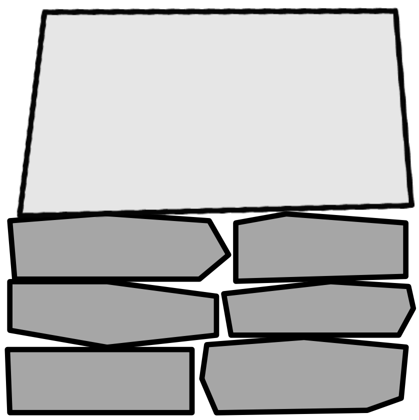
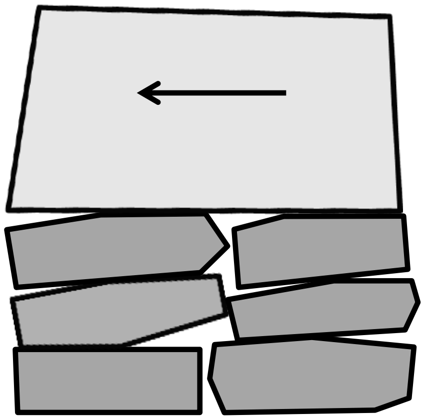
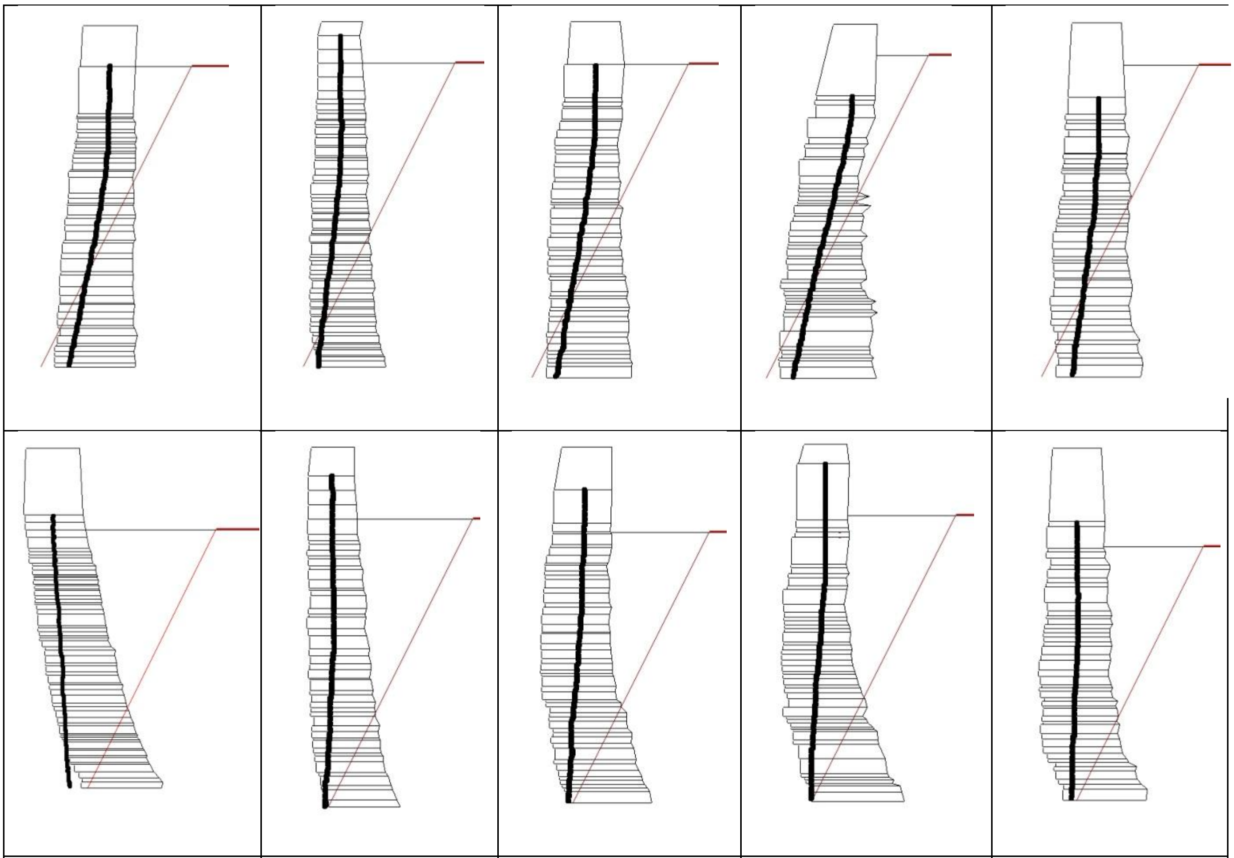
	
a) Initial configuration of stones	b) Small rotations allow some forwards movement and local relief of pressure on the back of the wall.

Figure 4 Effects of stone rotations



a) Wall 1 initial/final	b) Wall 2 initial/final	c) Wall 3 initial/final	d) Wall 4 initial/final	e) Wall 5 initial/final

The cross-sections of the walls are taken from the screen display of the computer program, and show lines of thrust (heavy lines), and the position and assumed spread of the load applied to the surface of the backfill (light lines). The horizontal lines on the cross-sections correspond to the levels at which observations were made during the tests, which were generally at the tops and bottoms of individual stones. Surveying of the topmost section of the wall was obscured by the loading frame, so less detail can be shown.

Figure 5 Analysed cross-sections of the walls

	
a) Wall 3: arching within the wall below the bulge	b) Wall 5: sliding of the wall over the lowest courses

Figure 6 Deformation features within the test walls