DESIGNING AND RETROFITTING WALLS FOR EXPLOSIVE EFFECTS AND NATURAL HAZARDS

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by
Major Stephen P Ward, CEng, MICE, Mi Mgt. M.I. Exp. E R.E. Ret.

Authors:
C. Brookes[1], J. Rance[2] and S.P.Ward[3]

ABSTRACT

This paper describes how numerical modelling based on a discrete element formulation has been employed to simulate the response of masonry to seismic loading and the effects of explosions. Using dynamic non-linear numerical analysis the performance of walls with and without retrofitted strengthening has been compared. Models representative of ashlar[4] masonry laid with a weak lime-based or regular mortar have been investigated. In general, wall arrangements are typical of those forming the end elevations of conventionally constructed low-rise buildings loaded horizontally. Walls including idealised openings have also been investigated. The benefits of strengthening by the introduction of passively stressed reinforcement are predicted for various arrangements when subjected to dynamic loading. The reinforcement is represented explicitly in the analysis allowing direct assessment of damage and potential failure mechanisms. The paper concludes that, the discrete element technique is ideally suited to dynamic masonry simulation and overcomes many difficulties experienced with traditional finite element analysis. The overall performance of masonry acting compositely with retrofitted reinforcement has been predicted and comparisons made between different reinforcement dispositions.

[1] Associate, Gifford and Partners, Consulting Engineers, Carlton House, Ringwood Road, Woodlands, Southampton, SO40 7HT, United Kingdom
[2] Director, Rockfield Software Ltd, Innovation Centre, University Of Wales, Singleton Park, Swansea, SA2 8PP, United Kingdom
[3] Division Manager, Cintec America Inc, 5540 Connecticut Ave., Suite 220, Washington DC 20015
[4] Ashlar masonry – masonry in which the block size is relatively large compared to the mortar layer holding the blocks together.

NATURAL HAZARDS

1. Introduction

a. The use of continuum based numerical models to simulate discontinuous structures, such as masonry, is fraught with difficulty. The introduction of discontinuities such as cracks during the loading event, or because of loading history, has to be wholly or partly predetermined. The use of gap elements allows cracks to open and maintain normal and shear force connection when closed but the crack locations have to be known in advance. Another approach is to avoid explicit representation of discontinuities but instead smear their effect by using a brittle non-linear material model. However, these models fail to predict mechanisms where, for example, initially isolated parts react dynamically together. Continuum methods can give satisfactory results but generally fail to provide a practical method of analysis for masonry.

b. As an alternative to the traditional finite element continuum approach, a discrete element (DE) formulation has been employed to simulate masonry with and without strengthening. So far, the results of the analyses have been applied to parts of buildings and used to help develop remedial design philosophies by providing simulations under ground excitations and explosive effects. A separate project where the engineering analysis has been based on the DE technique has involved the successful strengthening of over sixty masonry arches in Europe, Australia and USA. Predictive verification of full-scale tests underlies this work and has involved calculated collapse loads of masonry arch bridges as well as supplementary load tests on in-service bridges. Results have been shown to correlate very closely with tests (Brookes, Tilly 1999). As the technique is developed it is hoped that the performance of whole buildings can be checked before strengthening systems have been installed.

2. Analytical Requirements. In order to represent masonry with or without retrofitted reinforcement, particularly in seismic engineering where non-linear structural performance defines how ductility and energy absorption characteristics are exhibited, the following types of fundamental behaviour need to be included in the model.

a. Material and geometric properties of the masonry blocks themselves.

b. Contact-gap-friction effects along joints between the masonry blocks.

c. Depending on block and joint properties, the ability to evolve further joints by fracturing which in turn depends on limiting tensile strength and fracture energy.

d. Full account of stiffness and derived inertia loads which may occur over very short time intervals.

e. The capability to model post-failure behaviour to help verify simulations against the evidence collected after observed seismic damage and collapse.

f. To allow stress and initial damage from previous seismic events to be included.

g. The ability to represent retrofitted reinforcement including materially non-linear behaviour of the steel and the non-linear shear coupling behaviour of the bond with the surrounding masonry.

To date, most numerical simulations of masonry have been based on finite element continuum methods in which sophisticated and often-complex material models in conjunction with arrays of gap elements are used to account for the requirements listed above. A more intrinsically satisfactory approach for masonry is to base the analysis on a series of *discrete elements.* This more natural approach can be used to represent ranges of masonry from completely intact buildings to piles of random rubble.

3. Discrete Element Technique.

a. The technique used to perform all the analysis in this study is the discrete element (DE) method. This is a development of the distinct element method (Cundall, 1971) in which the concept of individual elements being separate and reacting with their neighbours by contact through friction/adhesion was first successfully applied to geotechnical and granular flow problems. Here elements were considered rigid but later developments (Munjiza et al, 1995) included the addition of element deformations and fracturing, with some overlap with traditional finite element theory.

b. In the current investigation the DE formulation available in the explicit dynamic version of ELFEN (Rockfield Software Limited, 1998) has been used. Essentially three different approaches have been used for the non-linear analysis of masonry each requiring different modelling approaches:

- *Macro Blocks.* The category where the joints between blocks have predominantly no strength and models the construction generally found in historic structures.

- *Brittle Material. This is where the masonry blocks and joints have predominantly similar strengths, as is more likely in modern forms of construction or where masonry is weak and random.*

- *Brittle Macro Blocks. Here the macro block approach is used with brittle materials thus permitting a blocky representation to fracture into further parts.*

The first two approaches have been investigated for shear wall applications to investigate the sensitivity of seismic resistivity to mortar properties (Brookes, Mehrkar-Asl, 1998).

(1) **Macro Block.** The macro block approach has been achieved by separately modelling each block or group of blocks in the structure and applying permanent static loads and seismic excitation to the base. Individual blocks of elements have defined elastic and plastic materials and are arranged to the required bond. All joints and therefore potential discontinuities are predefined and have friction parameters assigned. It is assumed that failure at joints always develops before blocks fail. However, the introduction of a von-Mises non-linear material model without hardening has been used to approximately represent block crushing thus giving a compressive stress cap. Material properties have been based on characteristic values determined for the masonry as a whole.

(2) **Brittle Material.** Where masonry includes high strength mortar or where the strength of blocks is low, a brittle non-linear material model has previously been used. Here the continuum becomes discretised due to evolving fractures in the blocks and possibly through joints. This is achieved in the analysis automatically using adaptive mesh algorithms. Using a Rankine material model, including fracture energy, newly generated cracks become contact surfaces requiring friction parameters to be assigned as for the macro block approach. However, for ancient ashlar walls with little or no mortar the macro block approach is preferred.

4. Reinforcement Representation. The finite element technique is used to model the reinforcement independently of the masonry using a partially constrained spar formulation (Roberts, 1999). Connection between the reinforcement and masonry models is achieved through non-linear bond elements. Modelling of reinforcement arrangements is completely automated without the need for topologically consistent element meshes thus accelerating the modelling process and permitting rapid comparison of designs. Currently, the capacity for reinforcement elements crossing masonry joints to generate transverse shearing strength or dowel effects is ignored. This is believed to be conservative.

5. Shear Wall Investigation

a. As part of the continuing development of retrofitted reinforced masonry support system applications and the expansion of joint venture historic structure remedial projects, Gifford and Partners with Cintec International are undertaking studies to investigate how the seismic resistivity of low-rise masonry buildings might be improved (Cintec International Ltd, 2000).

b. Retrofitted reinforced masonry support system anchors are comprised of stainless steel bar(s), a grouting sock and an engineered grout. Installation is by precisely drilled holes using wet or dry diamond coring technology. The sock consists of a specially woven polyester fabric shaped into a tubular sleeve to fit the required hole diameter. The use of the sock controls the volume of grout and ensures good contact is achieved with the surrounding masonry. The engineered grout has similar characteristics to Portland Cement based products, contains graded aggregates and other constituents which, when mixed with water, produce a pumpable mixture that exhibits good strength with no shrinkage. The size of the steel anchor, strength of grout and diameter of hole can be varied to provide the required design parameters and good stiffness compatibility with the masonry. Design parameters such as the bond strength between the grout and the masonry, which is often critical, are normally derived from static pullout tests. Figure 1 shows a diagrammatic retrofitted reinforced masonry support system anchor manufactured by Cintec.

c. Recognising that the out of plane behaviour of masonry panels has been the subject of much previous work (Key, 1998), effort has now concentrated on the detailed analysis of masonry shear walls, the primary structural element in masonry buildings. These shear walls are described below. The main objectives of the analyses were to provide some comparison between the performance of the walls with and without strengthening and to continue to explore the potential of the DE technique including modelled reinforcement applied to masonry under dynamic loading. An on-going programme of strengthening projects using the DE method to predict the ultimate strength of masonry arches, including comparisons with full scale tests, has shown the technique to be very accurate and better than alternative analyses for the case of static loading. Further work involving the out of plane prediction of masonry wall behaviour under high-speed dynamic loads arising from blast is also encouraging.

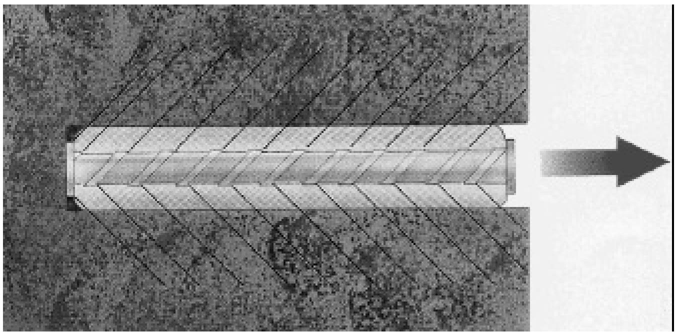


Figure 1. Typical installed Cintec Anchor

6. **Description.** The masonry shear wall under investigation is shown in Figure 2. The wall is 6.15m (20ft 2in) wide and 6.76m (22ft 2in) high, has shallow foundations over rock and has been considered with and without a large opening in each storey. It forms the shorter side of the rectangular building shown in Figure 3 and supports two floors capable of behaving as diaphragms. The longer side walls (not explicitly modelled) partly support the floors, and have little out-of-plane shear resistance. Vertical body forces and imposed loads are supported uniformly by all of the external walls. Loads developed by horizontal seismic ground accelerations in the transverse direction of the building are resisted by in-plane forces in the side walls. One of these walls is the subject of the current investigation.

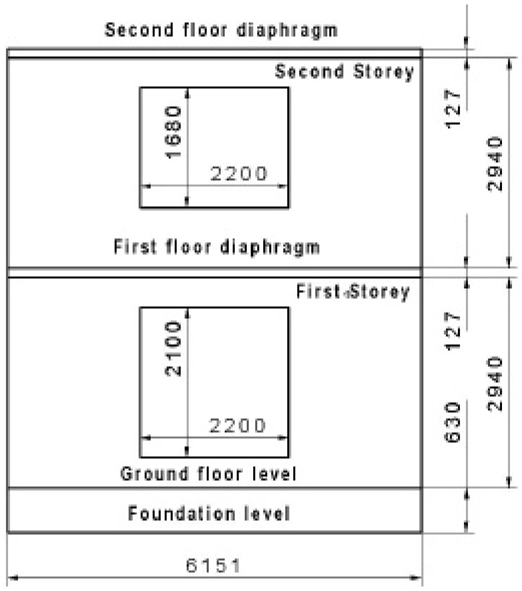


Figure 2. Stone masonry shear wall details (all dimensions in mm)

7.   Discrete Element Model.   Several plane stress DE models of a single side wall were developed incorporating the masonry blocks and slabs. The vertical loads and masses attributed to the slabs were modified to reflect mass and load transfer from the rest of the building. The wall is constructed from ashlar blocks, bedded on narrow and relatively weak lime-mortar. Without full scale testing of part of the wall, uncertainties exist for most material parameters and indeed the modelling. Previously both macro block and brittle material masonry models were used to help characterise wall behaviour (Brookes, Mehrkar-Asl, 1998). The focus of the work was the sensitivity of structural behaviour to equivalent friction of the mortar joints rather than the relative performance of different strengthening arrangements. In the current investigation, masonry material parameters have been fixed and based on those most representative of the form of construction. Similarly, macro block models have been used throughout so that the focus of the investigation is on the effectiveness of strengthening.

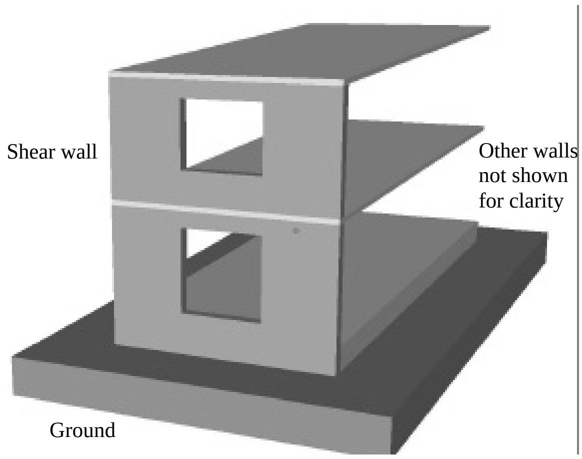


Figure 3. General arrangement of idealised building

Although it is feasible to include all of the blocks in the macro block representation, previous work on masonry arches had shown that there was little advantage in terms of accuracy and computational efficiency. Hence, each block in reality may include several squared stones. The floor slabs and foundation were defined as separate continuums with similar perimeter frictional properties to the blocks. It has been assumed that the floors and foundation are constructed such that their global behaviour is linearly elastic e.g. reinforced concrete slabs. All models were developed within a solid modelling environment using DISPLAY3 (EMRC, 1999), translated using in-house software and imported from within the ELFEN pre-processor.

8. **Material Properties.** Masonry material properties were based on those typical of well-built ashlar construction and using a weak calcareous sandstone laid with a lime based mortar. The strength and stiffness of the modelled blocks have been based on average composite values for the masonry treated as a whole. The contact and frictional behaviour of the mortar is modelled explicitly at the joints. Hence, it is not necessary to individually include the stiffness and strength of the joints or the stiffness and strength of the stone blocks. Table 1 lists the material parameters used.

Table 1. Masonry Material Parameters.

Description	Parameter	Value
Sandstone/lime masonry	Density	2200 kg/m^3
	Young's modulus	3.5 kN/mm^2
	Strength	5 N/mm^2
Mortar joints	Coefficient of friction (μ)	0.6
	Cohesion	0.15 N/mm^2
	Adhesion	0

9. **Loading.** Hypothetical horizontal seismic loading based on a circular frequency of approximately 0.6 Hz and containing six shocks was derived and applied to all of the models as displacement functions at foundation level (Figure 4). Two magnitudes of this simplified motion have been used with peak accelerations of 0.15g and 0.3g. Vertical accelerations were not considered due to the inherent inconsistencies in the distribution of mass that were required to simplify the problem to one of two dimensions. Horizontal motion results in a greater proportion of the effective mass being distributed to the two shear walls than that corresponding to vertical motion. All load-bearing walls resist vertical motion. However, whilst concurrent vertical motion has an influence on the overall behaviour of masonry shear walls, it is generally accepted that horizontal motion is more critical.

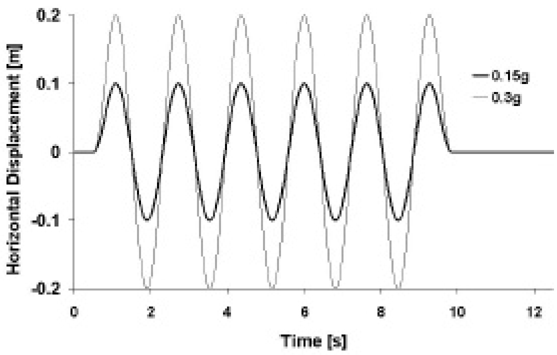


Figure 4. Time history of input motions

10. **Strengthening**. Both the plain wall and wall with openings were modified to include various arrangements of strengthening. Proposed dispositions of reinforcement included horizontal through individual block courses, vertical at the ends of the wall and end diagonal. Combinations of these patterns have also been considered. The reinforcement is fully bonded along its length. The reinforcement is introduced into the wall using the retrofitted reinforced masonry support system. All dispositions of reinforcement investigated used single 20mm diameter (#6 bar) ribbed bars installed in 50mm (2in) diameter holes. The anchors are designed not to be deliberately stressed but attract load during a seismic event. The modelled anchors permit recovery of bond stresses, axial stresses and slippage along the length of the anchor at any time during loading. Table 2 shows the various wall and reinforcement arrangements that were the subject of the investigation. Figure 6 shows a typical model including DE boundaries (bold), finite element subdivisions (fine) and modelled reinforcement (bold).

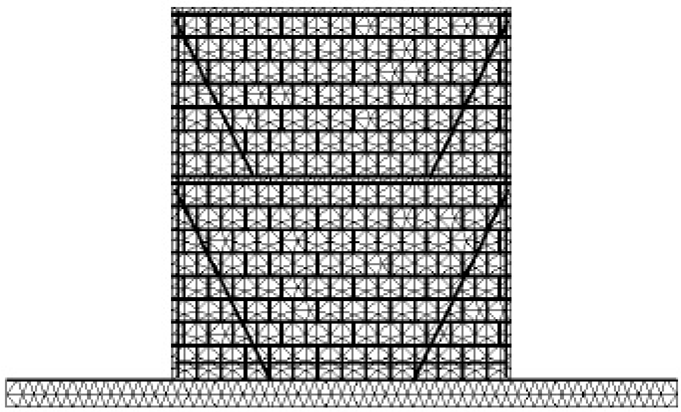
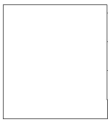
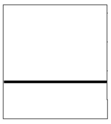
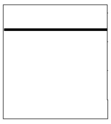
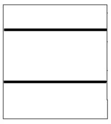
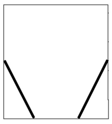
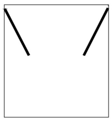
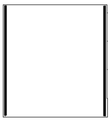
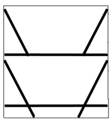
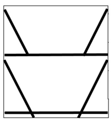
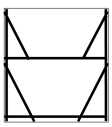
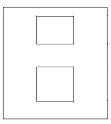
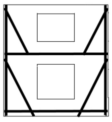


Figure 6. Typical wall DE and reinforcement model

Table 2. Wall and Strengthening Arrangements.

Arrangement	No.	Description (friction with cohesion and 0.15g loading used unless otherwise indicated)
	2,12,14	Plain wall. Also plain friction and 0.3g loading investigated.
	3,11	Horizontal reinforcement at the centre of the first storey. Also plain friction investigated.
	4	Horizontal reinforcement at the centre of the second storey
	5	Horizontal reinforcement at the centre of the first storey and the centre of the second storey
	7	Diagonal reinforcement at the ends of the first storey
	8	Diagonal reinforcement at the ends of the second storey
	6	Vertical reinforcement at the ends of the wall
	13	Combination of diagonal and horizontal reinforcement. Bottom reinforcement in second layer of blocks
	19	Combination of diagonal and horizontal reinforcement. Bottom reinforcement in first layer of blocks
	20,21	Combination of diagonal, vertical and horizontal reinforcement. Bottom reinforcement in first layer of blocks. Also 0.3g loading investigated
	22	Wall with openings in each storey

	23,24	Combination of diagonal, vertical and horizontal reinforcement. Bottom reinforcement in first layer of blocks. Also 0.3g loading investigated

11. **Discussion of Results**. Chiefly, DE simulations were carried out to show how the strength and ductility of the walls varied with reinforcement arrangement. However, for both the plain and reinforced wall No.3 (considered most probable), mortar with and without cohesion has also been considered. This limited investigation into the influence of mortar properties was carried out to allow comparisons to be made with earlier, simpler work. Where walls have exhibited a high degree of seismic resistivity, an additional ground motion with peak accelerations of 0.3g has been applied. The results have been illustrated by arrays of contour diagrams in which, the friction behaviour in the joints, magnitude of ground motion and the position of reinforcement have been varied.

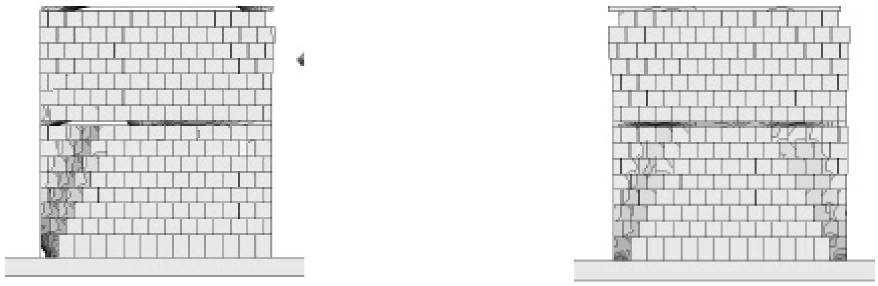
a. **Unstrengthened Simulations**.

(1) The aim of the current investigation is to develop a pattern of reinforcement that works equally well for plain walls as well as those with openings. In the model, the idealised opening has been created using a smeared[5] cell approach where approximately 36% of the wall is removed. The predicted ductility of the walls is highly sensitive to the properties of the joints and the duration of the event (Brookes, Mehrkar-Asl, 1998). In addition, the inherited damage history from preceding shocks increases the seismic vulnerability of the wall.

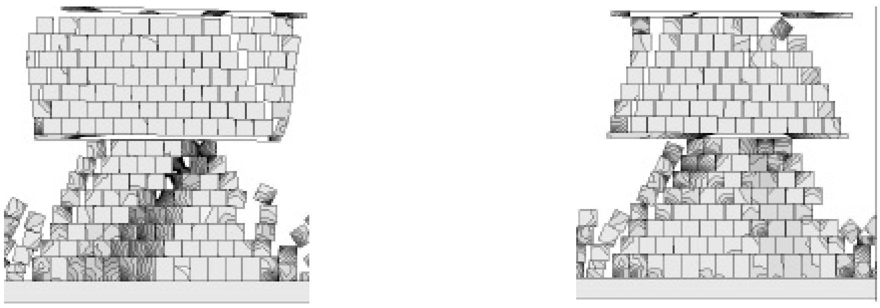
(2) In Figure 7, compressive stresses and deformed geometry half way through the seismic event and after ground motion has ceased (5.2s and 12.5s respectively) are shown. The shaded contours range between 2.2 N/mm^2 (dark) and -0.2 N/mm^2 (330psi and –30psi respectively). The plain wall is also loaded with a 0.3g ground motion. After the 0.15g events, the plain wall remains relatively undamaged with cracking in both storeys. The cracks in the second storey result in significant dilation across the wall whereas the cracks in the first storey are less dilated and are associated with locked in stresses. Doubling the acceleration results in massive damage. With much less masonry capable of resisting shear, the wall with openings is severely damaged after three shocks and collapses before the end of the event.

[5] Smeared cells in which discrete openings are represented by a single opening which characterises the behaviour of several openings.

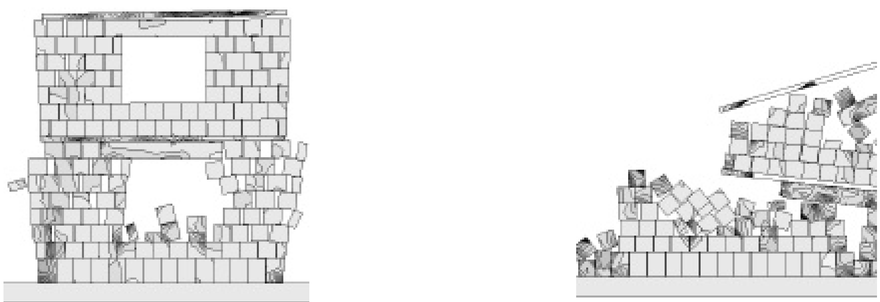
Halfway through event After event



Plain wall (No.2) – Maximum acceleration 0.15g



Plain wall (No.14) – Maximum acceleration 0.3g



Wall with openings (No.22) – Maximum acceleration 0.15g

Figure 7. Unstrengthened walls (contours of principal compressive stresses)

(3) The results show the general behaviour as well as the initiation of cracking. Cracking is marked by sudden stress discontinuities as well as the relative movement of blocks. This movement develops rapidly into local failure mechanisms when subjected to continued shocks in the hypothetical seismic event. The predicted failure and local collapse, reflecting modelled ductility and energy absorption, is similar to damage frequently sustained in seismic regions. Hence, these three models have been used as the benchmarks to compare the performance of various retrofitted reinforcement schemes.

b. **Strengthened Simulations.** The patterns of reinforcement included horizontal, vertical, diagonal and combined . Figure 8 shows stress and reinforcement slippage results obtained for horizontal reinforcement with and without cohesion effects included in the mortar (No.3 and No. 11 respectively). In the more realistic case with cohesion, the introduction of reinforcement has lead to more damage. Without cohesion but with μ=0.6, less damage occurs and good correlation is obtained with earlier simulations (Brookes, Mehrkar-Asl, 1998). The following reasons are likely for this behaviour:

(1) With cohesion, less energy is dissipated across the joints leading to less structural damping.

(2) Increased shear capacity across joints provided by cohesion helps a rocking mechanism develop in the wall. This mechanism causes sudden vertical oscillations of the wall above the reinforcement leading to additional damage.

(3) Axial slippage of reinforcement in excess of 5mm indicates that bond with the masonry has failed. The graphs show progressive bond failure at both ends of the reinforcement. As with the masonry, less damage occurs to the reinforcement when no cohesion is considered.

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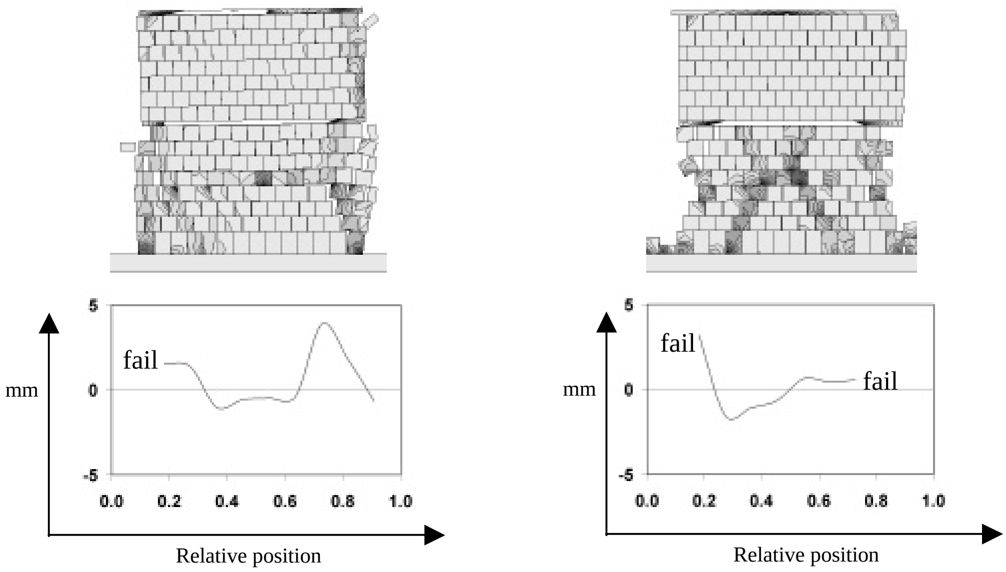
Halfway through event After event

Plain wall (No.2) – Friction with cohesion



Plain wall (No.11) – Simple friction

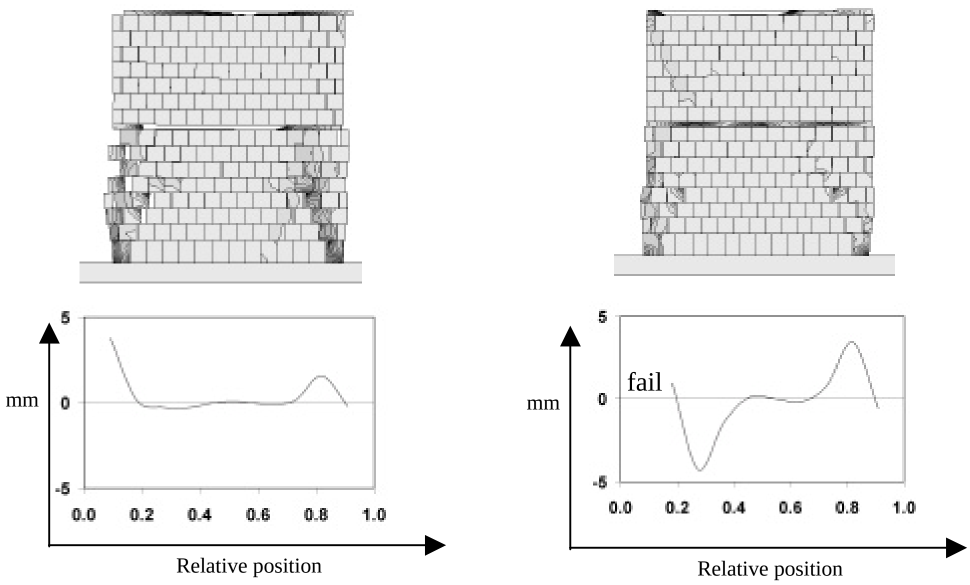


Figure 8. Horizontal reinforcement (contours of principal compressive stresses)

(4) In Figure 9, the results of the event are given for different reinforcement arrangements. All schemes except for No.19 and No.8 (not shown) cause either more damage to the masonry or produce higher locked in stresses compared with the similar unstrengthened wall. Arrangement No.19 has marginally improved resistance and prevented any blocks from falling away from the wall. In the case of No.13, an oscillatory mechanism develops above the first storey horizontal reinforcement causing increased damage by vertical movement.

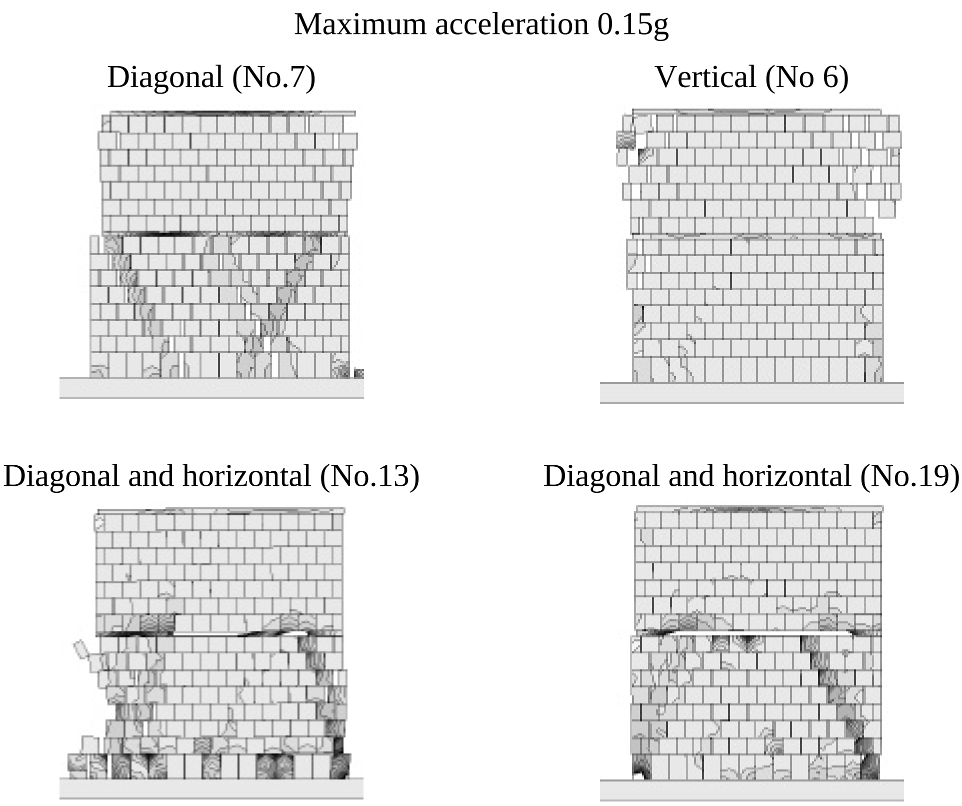
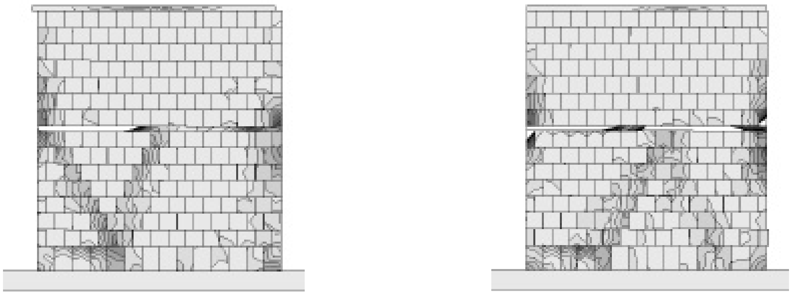


Figure 9. Diagonal, vertical, horizontal reinforcement
(contours of principal compressive stresses)

(5) Results obtained from schemes with the most developed patterns of reinforcement combining diagonal, vertical and horizontal bars are shown in Figure 10. Here, similar schemes have been applied to both the plain wall and the wall with openings. It is similar to No.19 except that the bottom horizontal reinforcement lies in the bottom course of blocks. This helps eliminate vertical motion caused by rocking of the blocks below the reinforcement level. The plain wall remains intact throughout the seismic event with the reinforcement controlling the development of cracking. Even under 0.3g loading, although higher locked in stresses are predicted little damage is evident. Without reinforcement, blocks left unsupported rapidly propagate failure mechanisms leading to collapse.

Maximum acceleration 0.15g Maximum acceleration 0.3g

Diagonal, horizontal and vertical (No.20 and 21)



Diagonal, horizontal and vertical (No.23 and 24)

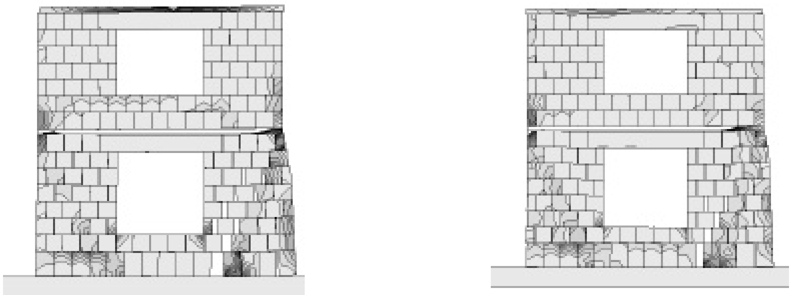


Figure 10. Combined reinforcement arrangements
(contours of principal compressive stresses)

(6) The wall with openings is similarly improved except that considerable cracking occurs to the first storey with appreciable dilation on one side. Compared to the unstrengthened wall, the improvement is significant. Debonding of some reinforcement has marked the beginning of a failure mechanism and continued shocks would be very damaging. Apart from directly supporting the ends of both walls, the reinforcement has encouraged shearing at the foundation level creating an energy absorbing process as well as offering some degree of base isolation.

EXPLOSIVE EFFECTS ON WALLS

12. **Previous Tests and Method**

a. In parallel with the seismic modeling presented above, a similar programme of work has utilised the combined finite/discrete element technology to investigate the strengthening of masonry walls subjected to blast loading. Both unstrengthened and strengthened walls were simulated and compared against field test results performed at COTEC[6] and Spadeadam[7].

b. In the field tests, various types of masonry walls consisting of regular brick and concrete block, with and without window apertures have been subjected to variety of blast loads. The aim of the tests was to investigate the behaviour of the retrofitted reinforced masonry support system under realistic conditions and develop the best techniques for design and installation.

c. The representation of the interaction of the blast wave with the masonry structure was accomplished using a semi-coupled approach. A CFD[8] analysis was used to predict the propagation of the blast wave and a combined finite/discrete element analysis to predict the response of the masonry wall.

d. Analysis of the blast wave propagation was undertaken using Air3d[9] assuming that the masonry structure was stationary. Since the blast wave passes the wall within 4ms this assumption is valid. The pressure history predicted on the face of the wall was subsequently applied as a loading to the finite/discrete element model.

e. The finite/discrete element (f/de) model comprised the masonry wall, a steel reaction frame, concrete support blocks and reinforcing anchors as depicted in Figure 11. A detailed representation of the masonry wall was provided. Each brick was modelled as a discrete elastic solid, with a mortar interface model employed between the brick surfaces to account for the behaviour of the grout. The steel frame utilised an elasto-plastic material model to enable plastic deformation to be captured.

f. In the case of the reinforced wall, non-linear anchor elements were used to represent the Cintec anchor reinforcement. These anchor elements account for elasto-plastic behaviour of the steel bars and the stiffness of the anchor grout in the direction along the bars.

g. Loading applied to the structure was gravity and a time-history pressure (obtained from the CFD results) defined for each brick face. Gravity was applied as a first stage to obtained initial conditions for the subsequent pressure loading.

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[6] COTEC – Cranfield University Ordnance Testing Evaluation Centre, West Lavington Down, Wiltshire, UK. Tests undertaken in 1999-2000

[7] Advantica (formerly British Gas), Test Facility at RAF Spadeadam, Cumbria, UK. Test undertaken in June 2001

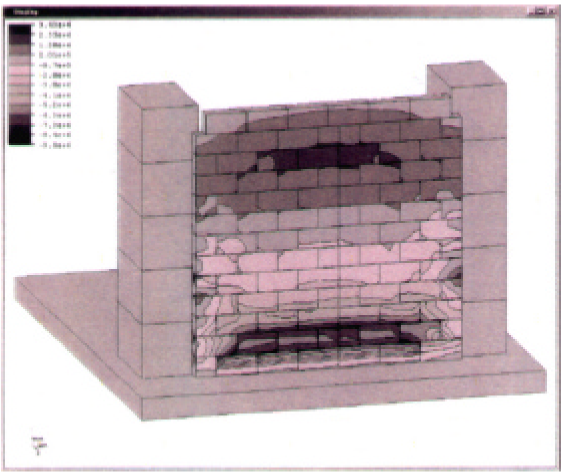
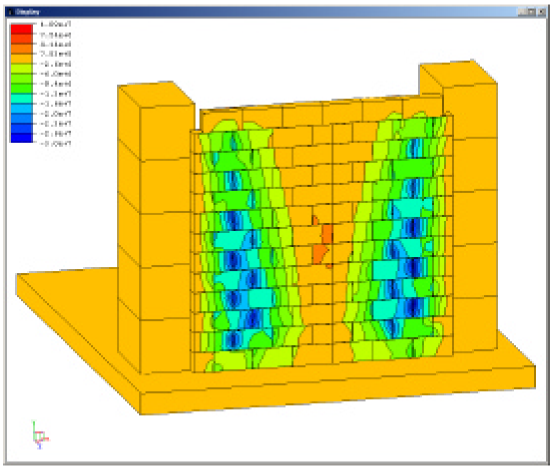
[8] Computational Fluid Dynamics

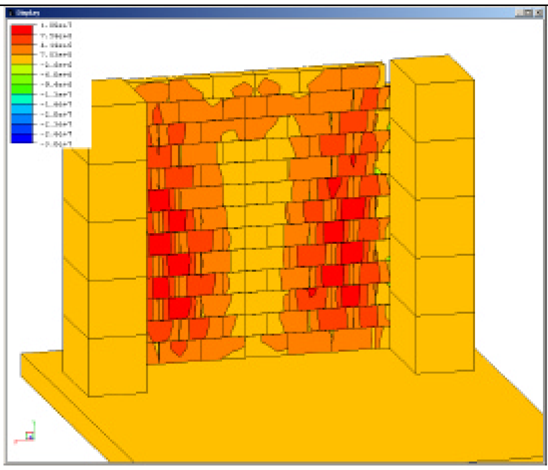
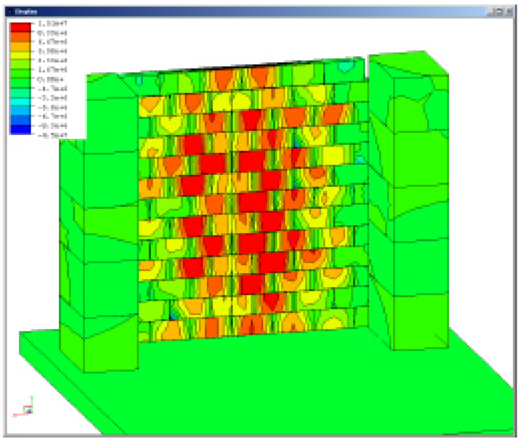
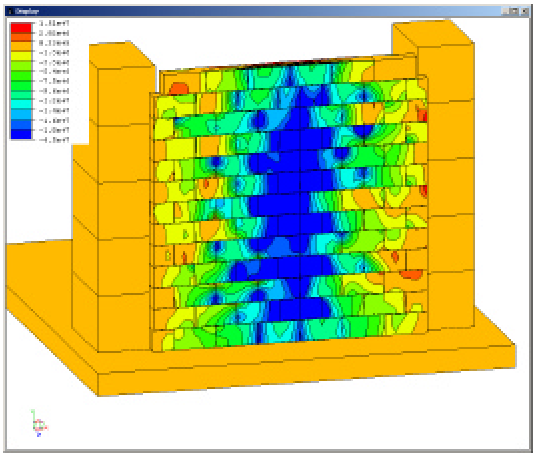
[9] Air3d developed by Dr Tim Rose at the Royal Military College of Science, Cranfield University, Shrivenham, Wiltshire, UK.

13. **Discussion of Results.**

a. The plots of stress distribution on the reinforced wall taken at varying time intervals are shown in Table 3 below. Deflection contours for the same wall are shown at the Appendix.

Table 3. Stress Distributions

	Plot of vertical stress contours at 1.0ms. The stress levels in the compressive zone at the base of wall are high enough to crush the bricks. This zone corresponds to an area on the test wall that exhibited front face brick failure.
	After approximately 1.5ms, the wall starts to bend at the edges and compressive horizontal stresses form at the vertical brick joints on the front face. This compressive force is also large enough to crush of the brick edges as seen in the tests.
	Also at 1.5ms, a horizontal tensile stress zone develops on the back face, commensurate with the bending action. The maximum tensile stress occurs at the centre of each brick, reducing horizontally towards the mortar joints. The joints have already failed since stresses are excess of the mortar tensile strength.

	
	After 2ms, the bending zones move inwards and meet at the centre of the wall. Significant horizontal stresses on the back face exceed the tensile capacity of the bricks and initiate vertical cracking.
	Also after 2ms, the compressive stress zone meets in the centre of the wall causing significant crushing of the bricks particularly along their vertical edges.

b. In order to compare the output from the model, before and after photographs of the Spadeadam Wall Test are reproduced in Figure 12. The aim of the test was to investigate the behaviour of a hollow concrete blocks (CMU) wall subjected to a severe blast load having been reinforced with retrofitted masonry support anchors. The wall measured 3m x 3m (10ft x 10ft) and was constructed within a steel reaction frame some eight weeks before the test. Once the wall had cured, it was then diamond drilled at 225mm (9in) vertical centres, corresponding with the masonry gauge. Retrofitted reinforced masonry support system anchors were then installed, inflated and allowed to cure for 28 days before the test.

Before After

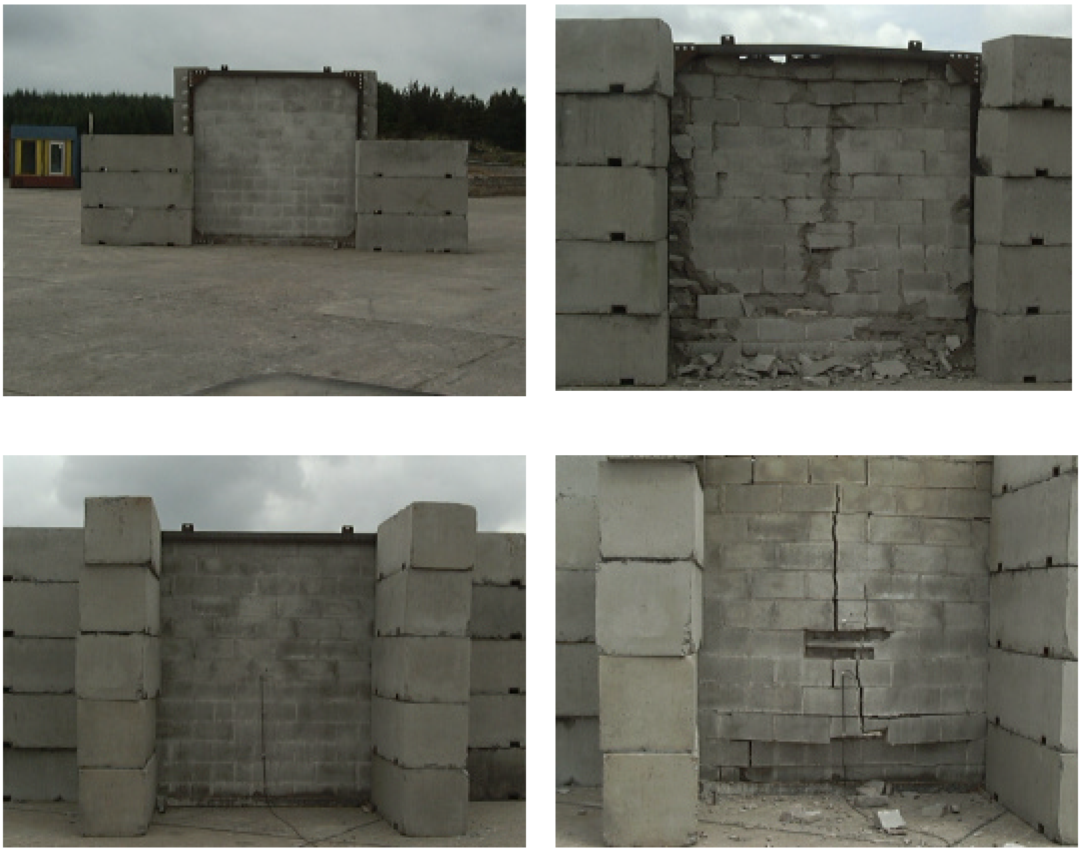
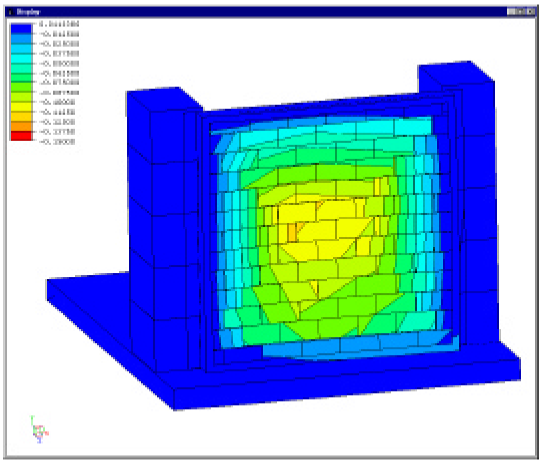
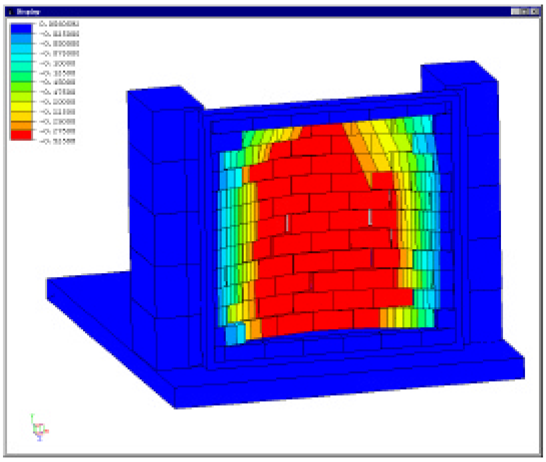
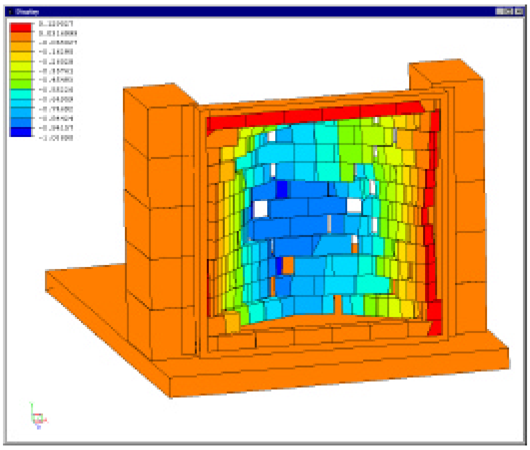
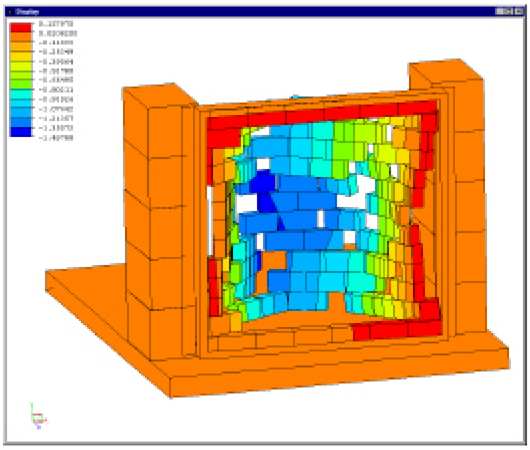


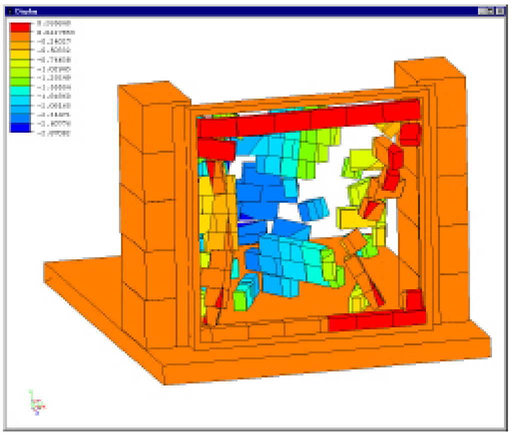
Figure 12. Before and after views of the Spadeadam Wall Test in June 2001. The wall was subjected to a blast load from 200kg TNT NEQ @ 12.5m; 534kPa, 1274kPa-ms (440lbs TNT NEQ @ 41ft; 77psi, 185psi-ms). The front view is at the top.

c. The displacement plots for the unreinforced wall taken at varying time intervals are shown in Table 4 below.

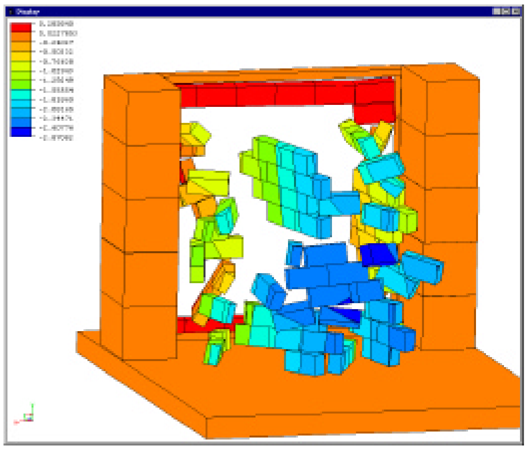
Table 4. Displacement Contours for the Unreinforced Wall

10ms 	40ms 
90ms 	140ms 

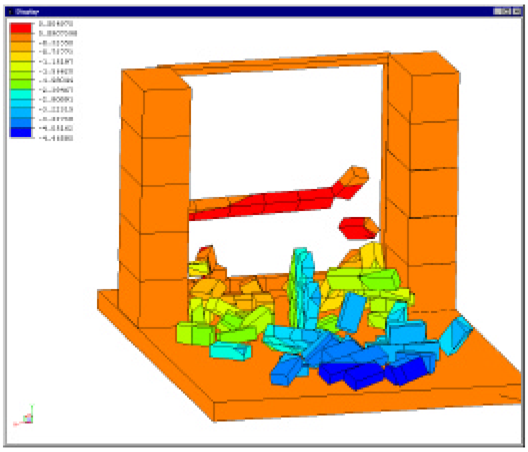
250ms



350ms



1000ms



14. **Conclusions**. By combining the discrete element technique with a finite element formulation for reinforcement, numerical models have been developed that have allowed rapid evaluation of the relative performance of reinforcement-based retrofitted strengthening. This process has been illustrated in two ways. Firstly, a plane shear wall was subjected to simplified hypothetical ground movements and secondly, a masonry wall was loaded by a blast wave. The results of the latter were also verified by a full-scale field test. The following conclusions have been drawn.

a. The overall performance of masonry acting compositely with retrofitted reinforcement can be predicted.

b. The sensitivity of seismic resistivity to the pattern of reinforcement has been determined thereby allowing the comparison of practically viable schemes.

c. A pattern of reinforcement has been investigated that improves seismic resistance and that works equally well for walls with and without openings.

d. The results have also shown that retrofitted reinforcement unless carefully placed may actually reduce seismic resistivity.

e. The development of strengthening schemes would have been extremely difficult and costly using conventional analysis or testing. However, to be completely confident that the results obtained by these numerical simulations are correct, further work is required to verify the simulations against observed behaviour of masonry structures subjected to seismic loading.

f. The field test proved conclusively that the retrofitted reinforced masonry support system is capable of providing the necessary strength to existing masonry walls to resist the effects of large blast loads. Furthermore, using the finite/discrete element model it is now possible to predict accurately the effects of a blast load on a strengthened wall and design the reinforcement pattern accordingly.

Appendix:

Deflection Contours for the Reinforced Wall

References:

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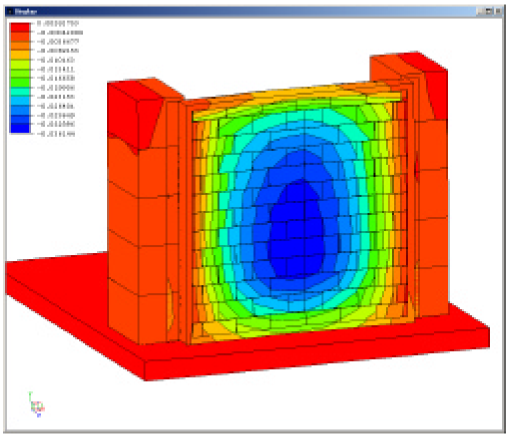
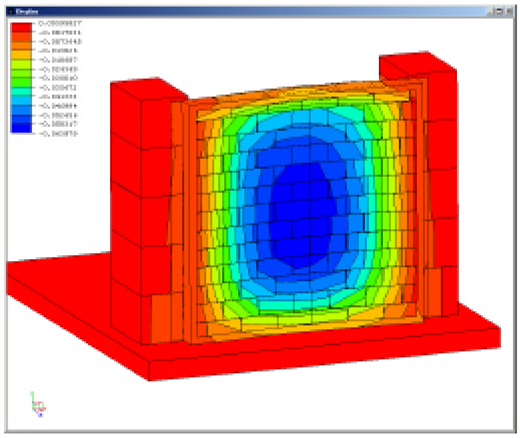
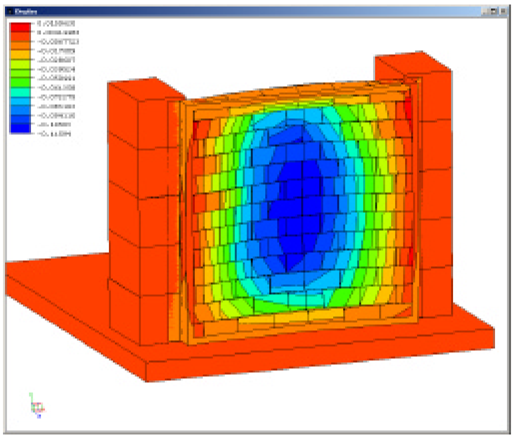
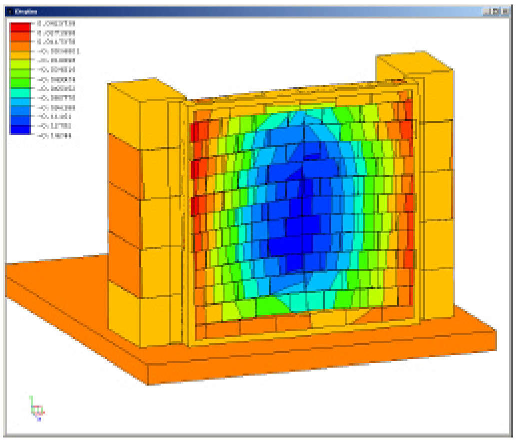
Appendix - Deflections Contours for the Reinforced Wall

4ms	6ms
	
10ms	16ms
	

Note the deflection of the steel reaction frame