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Racking shear resistance of prefabricated straw-bale panels

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The present study investigated the resistance to racking shear of 3 m × 3 m low-carbon, high-thermal-performance, prefabricated cladding panels that use straw bales. Although developed as a non-load-bearing panel for use with structurally framed buildings, the panels have the potential for use in low-rise, load-bearing structural walling. Test results confirmed that the panels exceeded current design requirements for racking shear and identified the mechanism whereby this occurred. The panels are considered suitable for future applications in low-rise structural walling solutions. A UK patent application (no. 0803489.4) has been filed on the panel.

1. INTRODUCTION

Meeting the challenge of climate change is a primary driver to find more sustainable methods of building. The *Code for Sustainable Homes*[1] aims to deliver zero-carbon homes for all new housing by 2016, requiring greatly enhanced performance in comparison with current practice. To minimise environmental impact, materials and buildings must minimise energy and carbon dioxide emissions in use and embodied within the fabric, as well as reduce water consumption and other environmental impacts of materials (e.g. eutrophication, resource depletion). Mitigation of environmental impacts of new developments has to be considered throughout all stages of the life cycle, including during design, material procurement, construction, service life and on end use. The construction industry is attempting to address the challenge of climate change in a range of creative and imaginative ways. Initiatives include the more efficient use of water, superior insulation, efficient energy and power systems and appliances, passive design and the use of low-impact materials.

The cladding panels have been developed as a low-carbon, high-thermal-insulation solution that uses straw bales, a widely available renewable resource. Using large quantities of plant-based materials offers the opportunity to store carbon within the fabric of the building, enabling better than zero-carbon methods of wall construction. Straw bales have been used in construction in the USA since the 1800s.[2] The durability of straw-bale construction is often raised as an issue by clients. Fire resistance has been shown to be more than satisfactory,[3,4] and techniques have been evolved to protect straw from decay.[5] To date, applications of the panels have included York Eco-Depot building and more recently the Knowle West Media Centre in Bristol (Figure 1). In these applications the panels have been used as non-structural cladding, but they also have the potential for use in low-rise structural walling, including housing. Using panels in this way would further contribute to the sustainability of the system through the elimination of steel and concrete elements, reducing the carbon footprint still further.

This paper reports the results of testing of full-size panels and elements of those panels. The performance criteria that need to be assessed are detailed and the methods used to assess these criteria are described.

2. THE PANEL

The panel (Figure 2) consists of a laminated timber frame, in-filled with straw bales and rendered in a proprietary formulated lime render. The timber frame is formed from 81 mm thick, three-ply, cross-laminated, untreated softwood board. Prefabricated straw-bale panels can be formed in a variety of sizes, but most conveniently accommodate the modular size of the standard-size straw bale (nominally 1 m × 0·45 m × 0·35 m) to minimise bale cutting. The frame used in the following tests had internal dimensions 2·92 m wide, 3·18 m high and 0·48 m thick. Typically construction takes place in a temporary 'flying factory' close to the building site in order to minimise the carbon footprint produced by transportation, and eliminate wet and 'non-traditional' trades from the site. Straw bales are locally sourced for the same reason. These panels are designed to contain nine layers of straw bales with each layer consisting of three straw bales. The straw bales are stacked in a running bond and are fixed together with timber stakes. In addition to this the bales are tied to the frame by timber stakes passed through holes in the surrounding timber frame. The height of the panels is also designed to require compression of the straw bales in order to mitigate against natural settlement. Corners of the panel are reinforced with stainless steel bars to maintain close dimensional tolerances during construction, and two vertical steel bars are inserted on each side of the panel to resist temporary short-term distortion caused by the compression of the straw bales. The faces of the straw bales are trimmed to produce an even surface which is coated with a 'scratch coat' of lime render to provide fire and weather resistance prior to assembly on the building. Once in situ, panels are then treated with a further two coats of lime render up to a minimum thickness of 30 mm.

Figure 1. Knowle West Media Centre, Bristol

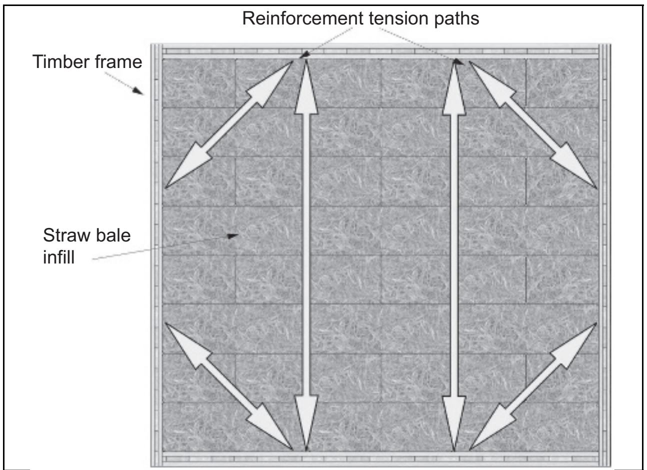

Figure 2. Panel components (patent pending)

3. PREVIOUS WORK

Although no previous research has been conducted on self-contained straw-bale panels of the type utilised in this investigation, there have been a number of studies on rendered straw-bale walls. Ash *et al.*[6] examined the racking strength of straw-bale walls measuring 2·44 m high × 2·44 m long with different renders and wire mesh reinforcement. Cyclical loading designed to simulate an earthquake was applied horizontally to a beam fixed to the top of the wall. Peak capacities at failure ranged from 5·82 kN/m for an un-reinforced earth plaster wall, 8·57 kN/m for a plastic-mesh-reinforced earth plaster wall, 10·94 kN/m for an earth plaster wall with heavy-gauge wire mesh reinforcement, 11·64 kN/m for a chicken-wire-reinforced cement stucco wall, 34·59 kN/m for a cement stucco render with 14 gauge (2 mm) 50 mm × 50 mm mesh, and 33·20 kN/m for a similar cement stucco render with additional spikes and cross-ties.

Carrick and Glassford[7] tested walls which were 2·7 m high × 3·6 m long reinforced with 1 mm diameter chicken wire and rendered with a 4:1 ratio, 30 mm thick sand and cement mortar. A 10 kN racking load (3·7 kN/m) produced a 2·4 mm deflection. Faine and Zhang[8] tested rendered straw-bale walls under compression. They made the comment that the bond between straw and render was observed to be very strong.

All of the previous tests have used mesh reinforcement of some type or other since this is standard practice in the USA and Australia, where straw-bale construction is more common.[2] Steen *et al.*[9] stated that mesh provides a mechanical connection for the plaster. Magwood *et al.*[10] proposed wire or plastic mesh for all render types. The panels under investigation currently use a lime-based render without mesh reinforcement. The tests described in this paper partly set out to assess the need to reinforce render.

4. TESTING PROGRAMME

Previous work[11,12] has shown that rendered straw-bale walls have adequate compressive strength for single-storey construction. The use of independent prefabricated panels offers the potential for use as structural building units with the majority of the compressive loading going through the timber frames. These require performance evaluation outside of the scope of earlier tests. The testing programme was designed to evaluate the ability of the panels to resist lateral and out-of-plane loads. Appropriate resistance to lateral loads would allow prefabricated straw-bale panels to be used as structural elements in a two- or three-storey domestic context without the need for a structural framework, such as is currently used in some buildings (e.g. York Eco-depot and Knowle West Media Centre). Adequate resistance to out-of-plane loads is necessary to meet wind-load resistance criteria. The results of the out-of-plane tests will be discussed in a later paper. Factors contributing to resistance to lateral loads were examined by testing the component materials, timber frame joints, three 2 m × 2 m experimental panels and four full-size 32 m × 3 m prototype panels.

The compressive and flexural strengths of the render at the time of the panel test (21 days from manufacture) were measured in accordance with BS EN 1015-11.[13] This test was conducted to establish the contribution that the rendered skin made to the overall stiffness of the panels. Two forms of the render were tested. A standard rendering mix for application by hand, which had a flow of 125% according to BS 4551-1,[14] and a wetter mix as required for spraying, which was the preferred method of application. It is known that the water/binder ratio affects the compressive strength of hydraulic lime mortars.[15]

Timber joints were tested for resistance to opening, closing and vertical pull-out, with and without reinforcement. This test was conducted to establish the contribution that the corner joints made to the overall stiffness of the panel. Half-metre lengths of the timber used for the panel frame were jointed in the same manner as on the panels using a single finger joint held in place by four 100 mm no. 8 wood screws on each side. An opening or closing force was applied 450 mm from the joint across the full width of the frame to both reinforced and unreinforced joints. The deflection from vertical was measured using four displacement transducers. These were placed in pairs close to the edges of the panel at two levels as shown in Figure 3.

The displacement measured by the upper pair of transducers was converted to an equivalent displacement at 3 m from the joint using trigonometry. The horizontal deflection (d) at an angular deflection of $\theta°$ for a panel with a height (h) is given by the equation

$$d = \sqrt{(2h^2 - 4h\cos\theta) - \left\{\left[\sqrt{(2h^2 - 4h\cos\theta)}\right]\sin\theta\right\}^2} \tag{1}$$

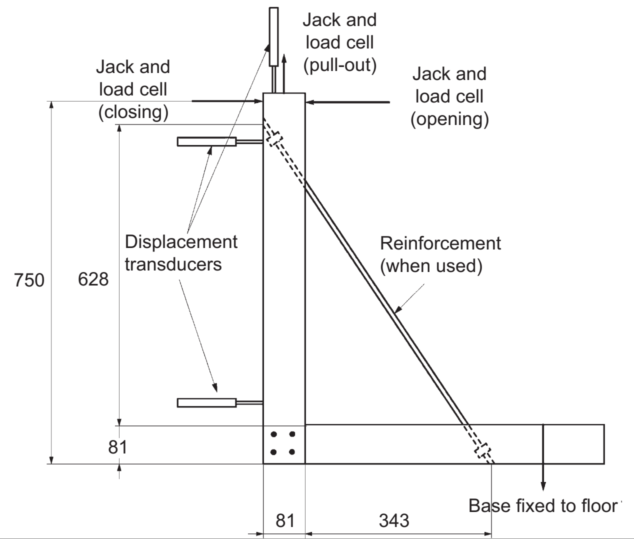

Figure 3. Joint testing set-up (dimensions in mm)

For a given value of θ, assuming that the timber does not flex, deflection at a particular distance from the joint is proportional to deflection at a different distance from the joint in the same ratio. Thus deflection at 3000 mm would be four times the deflection measured at 750 mm.

The load applied was converted to an equivalent load applied 3 m from the joint using the principle of moments. The moment of force (M) is equal to the product of the applied force (F) and the distance from the applied force to the object (r). Thus a force applied at 750 mm from the joint had four times the moment of force of the same applied force at 3000 mm from the joint. A 1 kN force at 750 mm from the joint was therefore equivalent to a 0·25 kN force applied at 3000 mm from the joint. These data are presented in Figure 4. The maximum acceptable deflection according to the design criterion was a 10 mm deflection to the top of the panel when a 10 kN racking load was applied to the top of the panel. Resistance to closing the joint was very low for both reinforced and unreinforced joints. The diagonal braces were bolted from the external faces of the panel, and when the joint was closed, the braces tended to slip through the locating holes with minimal resistance.

Following material tests, three experimental 2 m × 2 m panels were tested under racking shear loads The first panel comprised the reinforced timber frame only, the second comprised the frame filled with pre-compressed straw bales, and in the final test a complete panel with lime render finish was tested. The

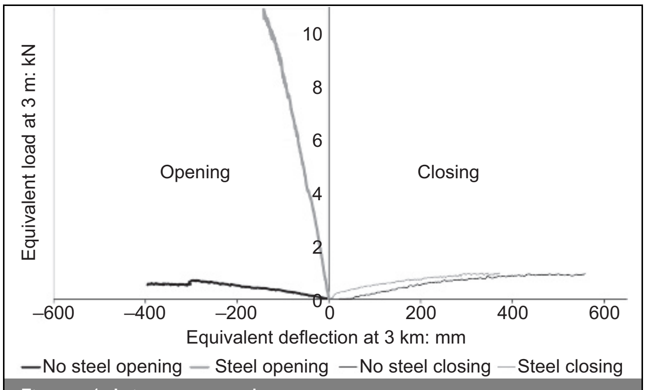

Figure 4. Joint test results

rendered panel was allowed 28 days to cure, at which time the render had a mean compressive strength of 4·44 N/mm^2 and a mean flexural strength of 1·51 N/mm^2. All panels were fabricated in the University of Bath laboratories and tested to assess their relative stiffness under increasing horizontal in-plane loading. These tests were conducted in order to assess the relative influence of the frame, straw and render components. Two-thirds scale panels were used in order to save on materials as they were in limited supply. It was considered that the contribution to racking strength made by the timber joints was independent of the size of the panels. The contribution to racking strength made by the straw bales was likely to be marginally greater in the smaller panels than the full-size ones because fewer joints between bales meant fewer discontinuities along which slippage could occur. The render also provided marginally greater racking resistance in the smaller panels since proportionately there was a greater volume of render. Although the proportionate contributions as measured in the small panels were likely to be marginally different in the full-size panels, the relative contributions would not be expected to differ significantly. The value of these small-scale tests was in improving the understanding of the performance of the composite panel.

The experimental programme culminated in the testing of four full-size (3 m × 3 m) prototype panels. One reinforced panel and one unreinforced panel were manufactured off-site and transported to the testing laboratory 14 days after manufacture. These were tested alongside similar panels manufactured at the same time in the laboratory. These side-by-side tests were conducted in order to establish whether transportation had any effect on the racking strength of the panels. The panels were subjected to cyclic lateral (racking) loads. Panels were loaded to service load and twice service load and cycled three times at each load. Panels were then loaded through to failure. This test was conducted to establish the deflection caused by loading and the extent of recovery after unloading.

The test set-up used for both the 2 and 3 m square panels is shown in Figure 5. The base of each test frame was fixed to the floor to avoid rotation and sliding during the test. Displacement transducers were attached to the faces of the vertical timbers at both sides of the panel in pairs in order to detect any twisting that might occur during the test. Transducers were placed on the centre line of the top of the panel to measure any vertical displacement within the panel. The horizontal timber at the top of the panel was loaded horizontally using a hydraulic jack, taking care not to put any direct load onto the vertical timbers; the applied force was measured using a load cell. The hydraulic jack was fixed to a strong wall and applied the load to the timber frame at the top of the panel through a ball joint onto a flat plate fixed to the timber frame. A ball joint was used in order to maintain a purely horizontal load. The load was applied exclusively to the top of the panel in order to replicate the racking load that would be transferred to the panel through the floor of a building.

The structural engineering requirement for the full-size 3 m × 3 m panels, as specified by Integral Structural Design, and used to assess performance of the prototype panels, was to sustain an in-service racking shear load of 10 kN (3·1 kN/m) with an in-plane maximum deflection of no more than 10 mm. A racking shear load of 10 kN was considered to be the

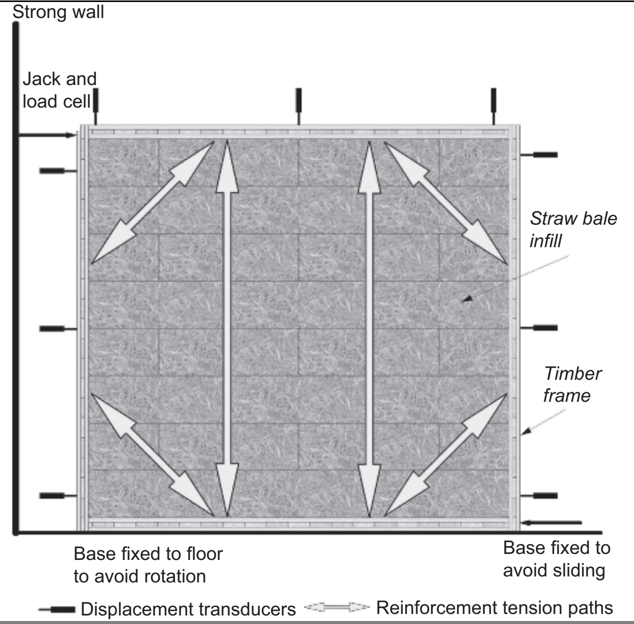

Figure 5. Lateral load test set-up

maximum likely to be encountered during normal service. A maximum 10 mm deflection was required to allow for the 15 mm tolerance designed into the panels to accommodate windows and doors which are installed using a flexible seal. However, in future taller buildings, racking resistance to a higher maximum load may be required.

5. RESULTS

5.1. Render tests

The results are shown in Table 1. It can be seen that the sprayed mortar was significantly weaker than an unsprayed mortar. The sprayed mortar had a compressive strength somewhere between a feebly hydraulic lime mortar (NHL2) and a moderately hydraulic lime mortar (NHL3.5). Although weaker than a lime–cement mortar (typically having a compressive strength of around 5–8 N/mm^2), this mortar will still make a significant contribution to the compressive strength of the composite panel.

5.2. Joint tests

The reinforced joint showed greater resistance to opening loads than the unreinforced joint, but in every case deflections that were greater that 10 mm occurred at loads of less than 0·5 kN. This shows that the joints were acting as hinges and the steel reinforcement, even for an opening joint, did not provide adequate resistance to racking.

Pull-out tests produced a vertical displacement of 7 mm at a load of 4 kN, which is the load that would be applied to each joint on lifting a fully manufactured panel of a typical mass of 16 kN. Failure occurred at a load of 12 kN. The resistance to pull-out shows that the panels should only be lifted from below, as lifting from the top of the panel would result in unacceptable movement.

	Standard lime render mix with a 125% flow	Wet lime render mix as used for spray application
Compressive strength: N/mm^2	5·88	2·44
Flexural strength: N/mm^2	2·14	1·16

Table 1. The 90-day compressive and flexural strength data for mortars

5.3. Initial panel tests

The joint tests suggest that the frame alone, with steel reinforcing, is unlikely to provide sufficient racking resistance. This was confirmed by the 2 m × 2 m frame tests. The racking load–peak horizontal displacement results for the three experimental panels are shown in Figure 6. The empty frame lacked stiffness, deflecting 6·5 mm under a load of just 0·5 kN/m. Although inclusion of straw increased stiffness, a loading of 0·5 kN/m produced a deflection of 2·7 mm; this improvement was insufficient to meet structural requirements. The complete rendered panel demonstrated the highest values of stiffness and resistance. For a peak deflection of 2·7 mm the complete panel sustained 1·8 kN/m, an increase of 350% compared with the straw in-filled panel. It is noticeable that in the rendered panel, and later in the full-size panels, there was a distinctive initial phase of very low stiffness for displacements to around 1 mm. This is attributed to a small shrinkage gap, of approximately 0·5 mm, that developed between the render and timber surround.

5.4. Full-size prototype panel performance tests

Data presented below are for one reinforced and one unreinforced panel. The data are from the worst performing of each type, although the differences between the two specimens of the same type were marginal. Cyclic movement data were almost identical, and the loading at failure was within 5% for the two tests. It was concluded that transportation had no effect on the structural strength of the panels.

Figure 7 shows the displacement measured at the top of the panel when subjected to a lateral load at the opposing top corner of the panel. Loading was taken to the service load of 3·1 kN/m and then taken off. This cycle was repeated three times for both the reinforced and the unreinforced panels. In both cases there was approximately 1 mm of 'bedding-in' after the first load was applied and removed. The reinforced panel deflected by around 2 mm under load, returning to an approximate 1 mm residual deflection at the end of each cycle. The unreinforced panel

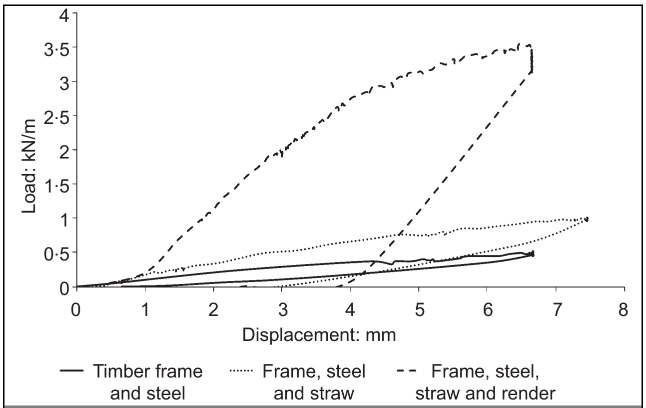

Figure 6. Racking shear–displacement performance for 2 m × 2 m panel tests

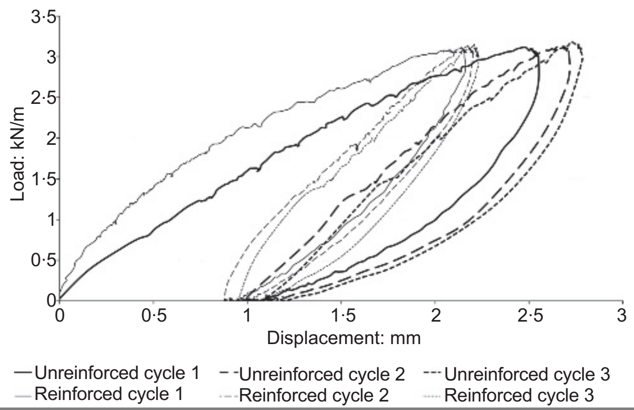

Figure 7. Racking shear cycling to service load of 3·1 kN/m

deflected between 2·5 mm and 2·75 mm under the same load, also returning to around 1 mm residual deflection.

The load test was then repeated at twice the service load (6·2 kN/m). Figure 8 shows the results of these tests. The reinforced panel deflected to around 10 mm under peak load, returning to a residual deflection of 2 mm from the original position. The unreinforced panel deflected by approximately 12 mm in the first cycle, increasing by around 2 mm on each subsequent loading. The residual deflection after the first cycle was 4 mm, increasing by around 1 mm after each subsequent cycle.

Both reinforced and unreinforced panels were then loaded to failure. Cracks appeared in the render of the unreinforced panels at 6·3 kN/m and failure occurred at 6·6 kN/m. Cracks appeared in the render of the reinforced panels at 7·9 kN/m with failure occurring at 19·5 kN/m (Figures 9 and 10).

It is evident that the major contribution to shear resistance is made by the lime render. Previous tests have shown the importance of renders to shear resistance of straw-bale walls. Ash *et al.*[6] demonstrated that changing the type of render (from earth to cement stucco), and including wire reinforcement, within the cement render increased the racking shear strength of a rendered straw-bale panel by nearly 600%. The wire reinforcement increased the tensile strength of the renders. The render used in the panels under investigation did not contain any mesh reinforcement; however, the vertical steel ties acted to improve the tensile strength of the composite panel. As the panel moves in shear, so the distance between the top and bottom timber panels reduces. This tends to put the render under compression, thereby reducing the tensile stresses, and hence increasing the racking shear resistance. Under racking shear the joints act as hinges, and it is only when they are torn apart that the render fails through tensile stress. The presence of the vertical steel reinforcement improves the resistance to pull-out of the joints, thereby improving the racking shear resistance of the composite panel. Similarly, if the joints had a greater pull-out resistance, this would tend to transfer the load into the render as a compression load, which would further improve the racking resistance.

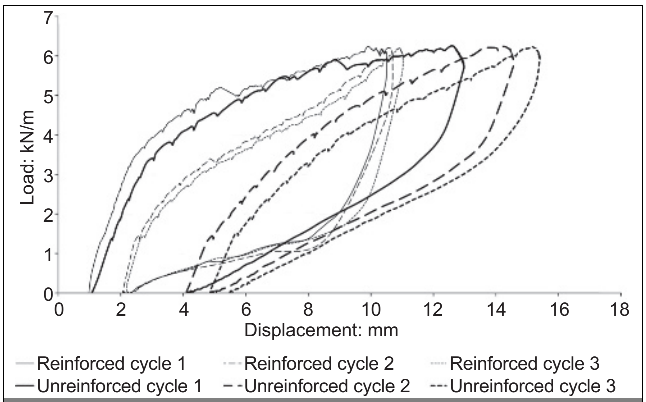

Figure 8. Racking shear cycling to 6·2 kN/m

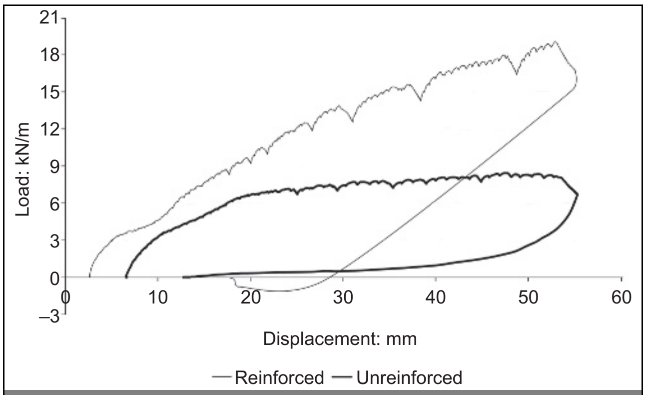

Figure 9. Racking shear to failure

6. SUMMARY AND CONCLUSIONS

Prefabricated straw-bale panels combine the benefits of straw-bale construction (use of renewable low-energy material, high thermal insulation) with the benefits of prefabrication and modern methods of construction. Although uses to date have been limited to non-load-bearing cladding applications, testing reported here shows that such panels are suitable for a range of modest structural applications. Seven racking load tests have demonstrated adequate performance and confirmed the significance of internal bracing and vertical reinforcement, and external render, to structural performance.

The racking shear strength resistance of the panel is derived from four different components: the timber frame; steel reinforcement; straw bales; the render. Both the joint tests and reinforced empty frame tests indicate the timber surrounds alone

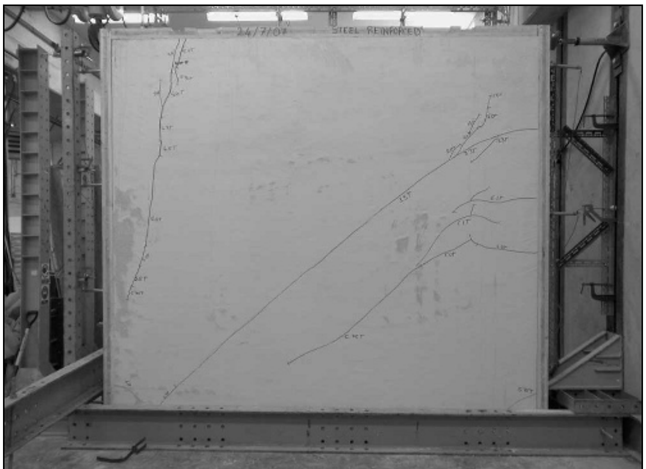

Figure 10. Crack pattern on panel loaded to failure. (All four sides of the frame remained true; curvature seen here is distortion caused by camera lens)

make little contribution to the resistance to racking shear of the panels. Although the straw improves stiffness, its contribution is insufficient to meet structural requirements. The lime render makes a significant structural contribution, as well as protecting the straw from decay and providing fire resistance.

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REFERENCES

1. Department for Communities and Local Government. The Code for Sustainable Homes – Setting the Standard for Sustainability in New Homes. DCLG, London, 2008. See http://www.communities.gov.uk/documents/planningandbuilding/pdf/codesustainhomesstandard.pdf. Accessed 07/07/2009.
2. King B. *Design of Straw Bale Buildings*. Green Building Press, San Rafael, CA, USA, 2006.
3. American Society for Testing and Materials. *Fire Tests of Building Construction and Materials: 1 Hour Fire Resistance Test of a Non-load-bearing Straw Bale Wall* ASTM, West Conshohocken, PA, USA, 2006. See http://www.ecobuildnetwork.org/pdfs/Non-Bearing_Clay_Wall.pdf. Accessed 05/10/2008, ASTM E119-05a.
4. American Society for Testing and Materials. *Fire Tests of Building Construction and Materials: 2 Hour Fire Resistance Test of a Non-load-bearing Straw Bale Wall*. ASTM, West Conshohocken, PA, USA, 2006. See http://www.ecobuildnetwork.org/pdfs/Cement_Stucco_Wall.pdf. Accessed 05/10/2008, ASTM E119-05a.
5. Summers M. D., Blunk S. L. and Jenkins B. M. *How Straw Decomposes: Implications for Straw Bale Construction*. Ecological Building Network, San Rafael, CA, USA, 2003. See http://old.strawbalebuilding.ca/pdf/How%20Straw%20Decomposes.pdf. Accessed 07/07/2009.
6. Ash C., Aschheim M. and Mar D. *In-plane Cyclic Tests of Plastered Straw Bale Wall Assemblies*. University of Illinois, Chicago, IL, USA, 2003. See http://www.ecobuildnetwork.org/pdfs/InPlane_Wall_Tests_Large.pdf. Accessed 07/07/2009.
7. Carrick J. and Glassford J. *Preliminary Test Results Straw Bale Walls*. Building Research Centre, University of New South Wales, Sydney, Australia, 1997. See http://strawbalebuilding.ca/pdf/Carrick%20Test%20results.pdf. Accessed 05/10/2008.
8. Faine M. and Zhang J. A pilot study examining the strength, compressibility and serviceability of rendered straw bale walls for two storey load bearing construction. In *Proceedings of the Fifth International Conference on Ecological Building Structure*, San Rafael, CA, USA, 2001. See http://strawbalebuilding.ca/pdf/Two-Storey%20LB.pdf. Accessed 5/10/2008.
9. Steen A. S., Steen B., Bainbridge D. and Eisenberg D. *The Straw Bale House*. Chelsea Green Publishing Company, White River Junction, VT, USA, 1994.
10. Magwood C., Mack P. and Therrien T. *More Straw Bale Building*. New Society Publishers, Gabriola Island, Canada, 2005.
11. King B. *Load-bearing Straw Bale Construction*. Ecological Building Network, San Rafael, CA, USA, 2003. See http://strawbalebuilding.ca/pdf/Load-Bearing_SB_Const.pdf. Accessed 05/10/2008.
12. Walker P. *Compression Load Testing of Straw Bale Walls*. Report by Department of Architecture and Civil Engineering, University of Bath, 2004. See http://people.bath.ac.uk/abspw/straw%20bale%20test%20report.pdf. Accessed 05/10/2008.
13. British Standards Institution. *Methods of Test for Mortar for Masonry Part 11: Determination of Flexural and Compressive Strength of Hardened Mortar*. BSI, London, 1999, BS EN 1015-11.
14. British Standards Institution. *Method of Testing Mortars, Screeds and Plasters: Part 1 Physical Testing*. BSI, London, 1998, BS 4551-1.
15. Allen G., Allen J., Elton N., Farey M., Holmes S., Livesey P. and Radonjic M. *Hydraulic Lime Mortar for Stone, Brick and Block Masonry*. Donhead Publishing, Shaftesbury, 2003.