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STRENGTH CHARACTERISTICS OF INTERLOCKING DRY-STACKED CONCRETE BLOCK MASONRY

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ABSTRACT

A dry-stacking, interlocking type of block, known by the trade name of Sparlock™, was tested to determine loadbearing properties for axial compression and out-of-plane bending about both the horizontal and vertical axes. Prisms and small walls were built as plain masonry, surface bonded masonry and grouted and reinforced masonry. Loadbearing capacities comparable to traditional masonry were obtained for the latter two forms of construction and even the interlocking in the plain form of construction (i.e., no mortar, surface bonding, or grout) provided significant stability and strength.

INTRODUCTION

To achieve increased efficiency and to reduce the need for highly skilled tradesmen, many types of dry-stacked masonry have been developed (Glitza 1991, Harris et al. 1992). Sparlock™ block masonry is one of the newer mortarless systems and is unique because of both horizontal and vertical interlocking and use of special blocks for corners and ends of walls at openings. Vertical interlocking is achieved by starting with a half height unit on one side of the wall and then building with full height units to the top of the wall where another half height unit is required to finish off the wall level. Each block is laid directly on top of the block below in a stack pattern so that dimension control to maintain uniform heights of block is not as critical as for cases of dry-stacked

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masonry in running bond. Horizontal interlocking is achieved by the shear keys where the units from opposite sides of the wall lock together as shown in Fig. 1(b).

The research reported here follows initial elastic analysis and tests carried out by Hatzinikolas et al. (1986) to investigate basic structural properties. The capacities of the Sparlock™ block system under axial load and flexure are the focus of this presentation. In addition to the inherent strength and stiffness resulting from the interlocking feature, surface bonding with a modified portland cement parging containing a fibreglass mesh or filling the cells of the hollow units with grout were investigated to document the improvements in strength and stiffness. Only stretcher units were used in the tests.

PROPERTIES OF SPARLOCK BLOCKS

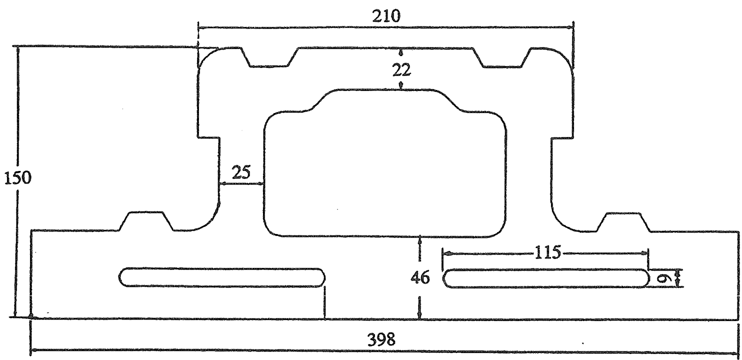
The average dimensions from five randomly selected stretcher units are shown in Fig. 1(a). These values are for the top of the block whereas the dimensions of the solid parts at the bottom of the block are generally about 2 mm larger. The average height of the block was 196 mm (7.72 in.). The 0.6% coefficient of variation for height is not a problem because of the stack pattern of construction. For the same five blocks, the average density was 2248 kg/m^3 (140 lb/ft^3) with C.O.V. of 1.6%. During the tests, it was observed that the masses of two blocks with a slightly brownish colour were slightly higher than the other more usual grey blocks. This observation plus strength tests indicated that two different batches of block were used in the test program.

Using five block samples, the average masses for the brownish and grey samples were 11.250 kg (24.76 lb) and 10.87 kg (23.91 lb), respectively. Compression tests using hard capping and solid steel capping plates gave compressive strengths of 28.4 MPa and 19.87 MPa and splitting tension tests across the 70 mm (2.76 in.) face shell (conforming to ASTM C 1006) gave tensile strengths of 4.44 MPa and 3.54 MPa, respectively, for the brownish and grey samples.

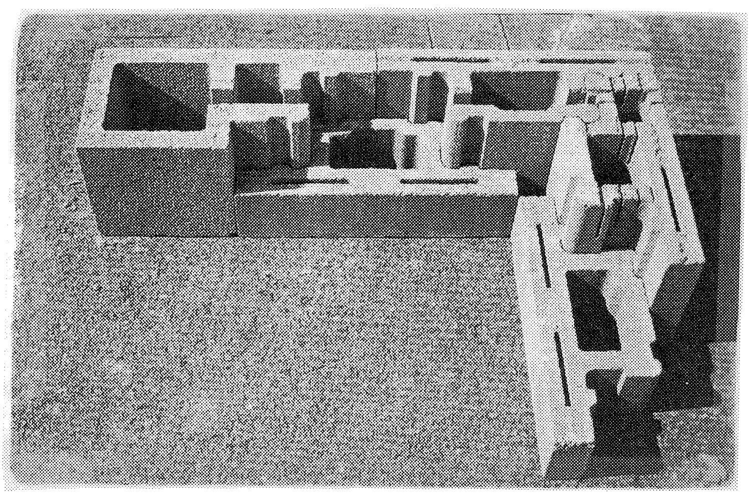
CONSTRUCTION OF TEST SPECIMEN

Test specimens were constructed with random mixes of brownish and grey stretcher units before the difference was noticed. (Interpretation of how this affected the results is discussed later.) Because only stretcher units were used, ends of walls were finished evenly using half blocks which were obtained by cutting whole units vertically in half. The bottom course started with a half height unit on one face of the specimen and another half height unit was used at the top course to end up with uniform height.

The dry stacking of the blocks was done with reasonable care but using untrained personnel. Generally the specimens were built to within 6 mm of plumb but two of the surface bonded walls were measured to be up to 12 mm out-of-plumb. This quality of construction is thought to be of lower quality than would be expected of trained blocklayers in the field.



(a) Plan view of a Stretcher Block.



(b) Combination of different blocks.

Fig. 1 Sparlock™ Interlocking Block System.

Specimens were made using the following three types of construction:

Plain: Blocks were simply dry stacked.

Surface Bonded: The fibreglass mesh used in the surface bonding was FIBACRETE IM with nominal strengths of 6.4 kg/mm (360 lb/in.) and 7.2 kg/mm (400 lb/in.), respectively, in the warp and weft directions. The fibreglass mesh was embedded in a thin layer of DUROCK Prep-Coat™ which had been trowelled on the face of the block. Then a final layer was trowelled over the surfaces resulting in an overall thickness of approximately 2 mm (0.08 in.). The fibreglass mesh was lapped 50 mm (2 in.) at joints between the horizontally placed 965 mm (38 in.) wide strips of fibreglass mesh. The surface bonding layer was allowed to cure in the dry laboratory conditions for 28 days prior to testing. The final architectural finish coat was not applied.

Grouted: For grouted specimens, the hollow cells of the blocks were filled with grout consisting of one part portland cement to five parts concrete sand with sufficient water added to produce a 200 mm (8 in.) slump. The grout was consolidated using a 29 mm (1 1/8 in.) diameter pencil vibrator. Grout cylinders, 100 mm diameter by 200 mm long (4 in. by 8 in.), were air cured and tested at the same time as the grouted specimens. The average compressive strength was 23.8 MPa (3450 psi) with a coefficient of variation of 6.3%.

TEST RESULTS

Prism Tests

Five four-block high by one-block long prisms were tested for each type of construction. The bottom of each prism was set in a thin layer of hydrostone on the base of the test machine and a 51 mm (2 in.) thick steel plate was hard capped to the top of the prism. Load was applied through a 229 mm (9 in.) diameter spherical seat. For each type of construction, two prisms were gauged over the middle 200 mm (8 in.) height on both sides for strain measurements using a demountable mechanical strain indicator.

The compressive strengths from the prism tests are summarized in Table 1 where the net area of 55690 mm^2 (86.3 in.^{2}) was used for the plain and surface bonded prisms and the gross area of 80,000 mm^2 (124 in.^{2}) was used for the grouted prisms. As can be seen, the surface bonding and grouting resulted in compressive strength increases of 20% and 118%, respectively. However, because of the larger effective area, the grouted prisms had 3.1 times the compression load carrying capacity of the plain construction.

For the surface bonded prisms, initial failure was evident from bulging and separation of the surface bonding layer from the blocks at the horizontal joints between blocks. The final failure of the blocks in the prism was similar to the plain prisms with vertical cracks developing in the webs and/or the face shells. Figure 2 is a photograph of a prism after final failure. For grouted prisms, failure was accompanied by development of vertical

Table 1 Average Tests Results

Type of Construction	Compressive Strength, MPA (C.O.V.), %	Flexural Capacity, kN-m/m (C.O.V.), %		Eccentric Load Capacity, kN/m (C.O.V.), %
		Normal to Bed Joints	Parallel to Bed Joints	
Plain	6.74 (12.3)	2.13 (26.7)	3.23 (22.7)	299 (7.8)
Surface Bonded	8.08 (6.8)	5.50 (9.1)	7.43 (2.6)	399 (13.9)
Grouted	14.7 (4.8)	16.02 (1.3)	16.09 (11.8)	706 (4.7)

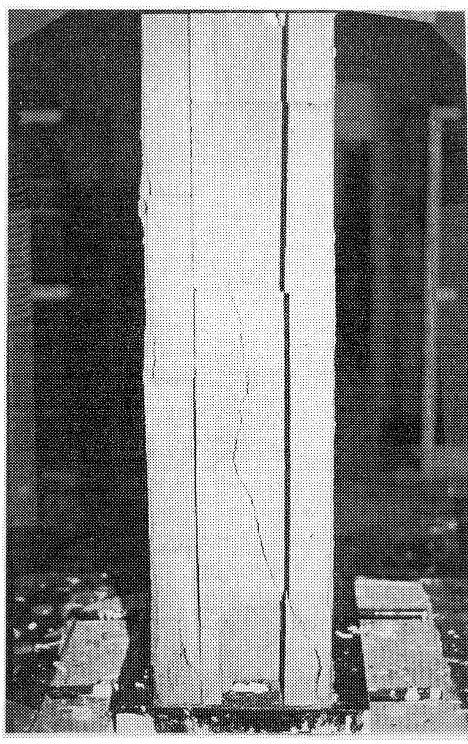


Fig. 2 Web Cracks and Face Shell Cracks in a Surface Bonded Prism.

cracks which propagated through the face shells rather than running parallel to the face shells as was the case for the plain and surface bonded prisms.

The average stress-strain relationships for each type of construction were plotted in Fig. 3. The initially lower slope for the plain construction indicates closing up of the horizontal joints between blocks at low stresses. After this, a stiffer behaviour is observed. The difference between the relationship for the surface bonded construction versus the plain construction is due to the stiffening effect of the surface bonding layers at early stages of load. The results for the grouted prisms are typical of grouted blockwork where a gradual softening is observed at loads above approximately half the capacity. The continuity provided by the grouted cells seems to have prevented any initial closing of the horizontal joints.

Flexural Tests of Walls

Two sets of 3 walls were tested for flexural strength for each type of construction. For tension normal to the bed joints, the wall specimens were 2 1/2 blocks long and 8 blocks high (nominally 1 × 1.6 m or 3 ft. - 4 in. by 5 ft. 4 in.). For tension parallel to the bed joints, the wall specimens were 4 blocks long by 5 blocks high (nominally 1.6 × 1 m or 5 ft. 4 in. by 3 ft. 4 in.). For the six grouted wall specimens, two number 15 bars (total A_s = 400 mm^2 or 0.62 $in.^2$) were placed at the mid-thickness of the wall in the direction of bending. For tension parallel to the bed joint, slots had to be cut into the webs of the block to permit the reinforcement to be placed horizontally. The bars were located 200 mm (8 in.) from the edges in the 1 m (3 ft. 4 in.) direction.

The walls were tested in a horizontal position as simply supported beams with a clear span of 1.5 m (5 ft.) between the pin and roller support system. All of the plain and surface bonded walls and one grouted wall tested for tension parallel to the bed joint were tested using two symmetrically placed line loads located at 0.55 m (1.8 ft.) from each support as shown in Fig. 4. Because of concern that shear failure might control capacity for such a small shear span, the remaining five grouted walls were tested using a single mid-span line load. Hollow structural steel sections were used to create the line loads and combinations of pins and rollers were used to ensure that neither beneficial arching nor detrimental torsion were introduced during loading. Mechanical dial gauges were used to measure midspan deflections. Initial deflections due to self-weight were also measured.

Ultimate moment capacities for tension both normal and parallel to the bed joints are summarized in Table 1. The weights of the specimen and the loading apparatus were included in the calculated moment capacities.

As expected, there was a much greater scatter of the results for the plain walls and the addition of surface bonding substantially increased the moment capacities, as well as producing more consistent results. In both cases, the moment capacities for tension parallel to the bed joints were higher, probably because the vertical joints are not continuous through the thickness. A 14.1 kN-m (10.4 ft-kip) moment capacity, from the first test of a grouted wall with tension parallel to the bed joint, appeared to have been

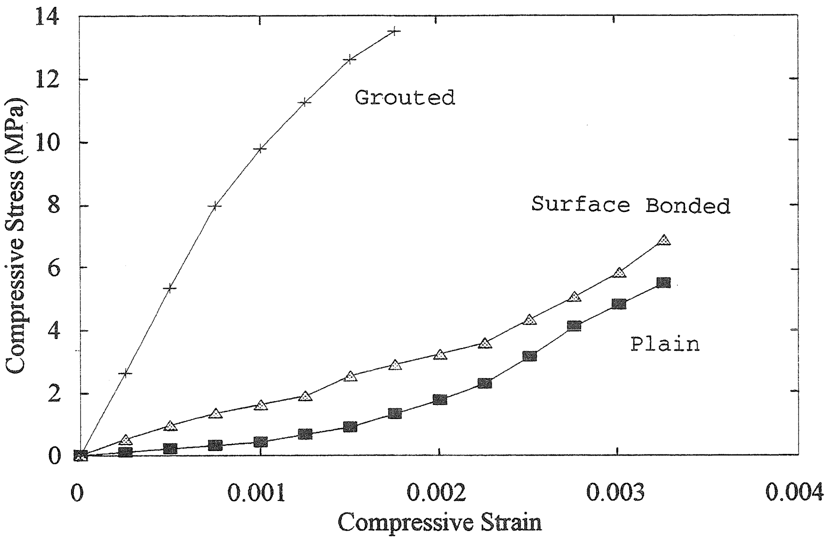


Fig. 3 Compressive Stress-Strain Relationships from Prism Tests.

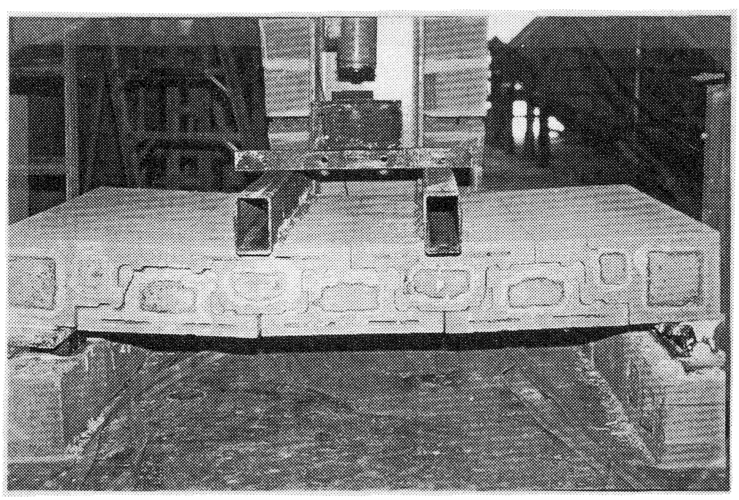


Fig. 4 Flexural Test of Grouted Wall with Two Line Loads and Tension Parallel to the Bed Joints.

controlled by shear failure. Although this premature shear failure is a lower bound on the moment capacity, this result was included in the 16.09 kN-m average for the three tests.

Average results for midspan deflection versus midspan moment were plotted for each type of wall in Fig. 5. In Fig. 5(a) for tension normal to the bed joints, the sudden change in the slope for the surface bonded wall, at around 1.4 kN-m (1.03 ft-kips) bending moment, corresponds to tensile cracking of the blocks after which the fibre mesh provides the tensile force. The very stiff behaviour of the grouted wall up to about 6 kN-m (4.4 ft-kips) indicates the contribution of the grout prior to cracking. Following cracking of the grout, the reinforcing bars provided the tensile force. Failure corresponds to yielding of the reinforcement. In Fig. 5(b), similar results were observed for tension parallel to the bed joints.

Eccentric Axial Load Tests of Walls

Three sets of three walls were tested under axial loads positioned at an eccentricity of one-third the wall thickness (t/3 = 66.7 mm or 2.6 in.) from the centre of the wall. This line load condition at the top of the wall and the uniform initial bearing at the base of the wall conform with the requirements of ASTM E72. The line load, along the 1 m (3 ft., 4 in.) top of the wall, was created using a load distribution beam, a full length roller and a 25 mm (1 in.) thick steel capping plate on top of the wall as shown in Fig. 6,

The ultimate compression loads are listed in Table 1. As shown in Fig. 6, the capacity of the plain wall was controlled by compression failure of the face shell on the compression side of the wall. As can be seen in the load-deflection plots in Fig. 7, closing of the bed joints on the compression side and opening on the tension side at the early stage of loading resulted in an initial large deflection after which a stiffer behaviour prevailed.

The average ultimate load for the surface bonded walls was 33% higher than for the plain walls. As can be seen in Fig. 7, the initial stiffness was also much higher where the fibreglass mesh restricted opening of the joint on the tension side of the wall and the surface bonding prevented closing of the joint on the compression side. Following local buckling of the surface bonding on the compression side, the observed reduced stiffness was consistent with the prism tests.

The failure pattern and load-deflection relationships for the grouted walls are very similar to those observed for traditional grouted blockwork. The 136% increase in wall capacity compared to plain walls is less than for prisms because the eccentric load results in a smaller share of the load being resisted by the grout in the cells along the centre of the load.

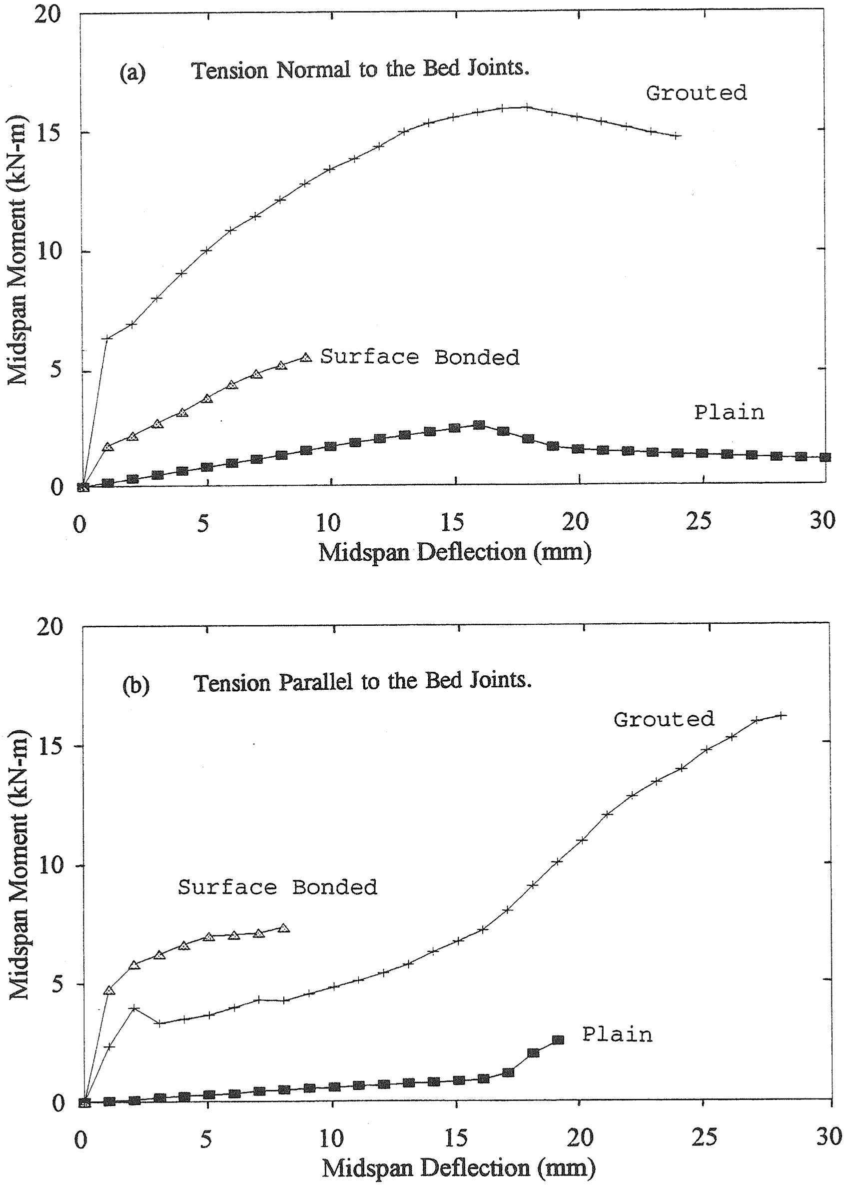


Fig. 5 Midspan Moment vs. Midspan Deflection Relationships from Wall Bending Tests.

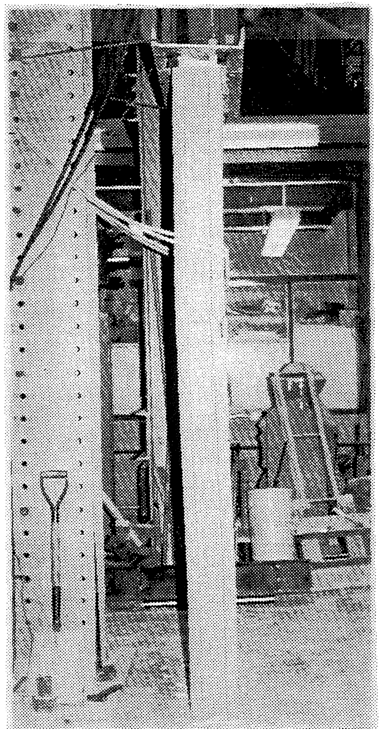


Fig. 6 Photograph of Failure of a Plain Wall under Eccentric Axial Compression.

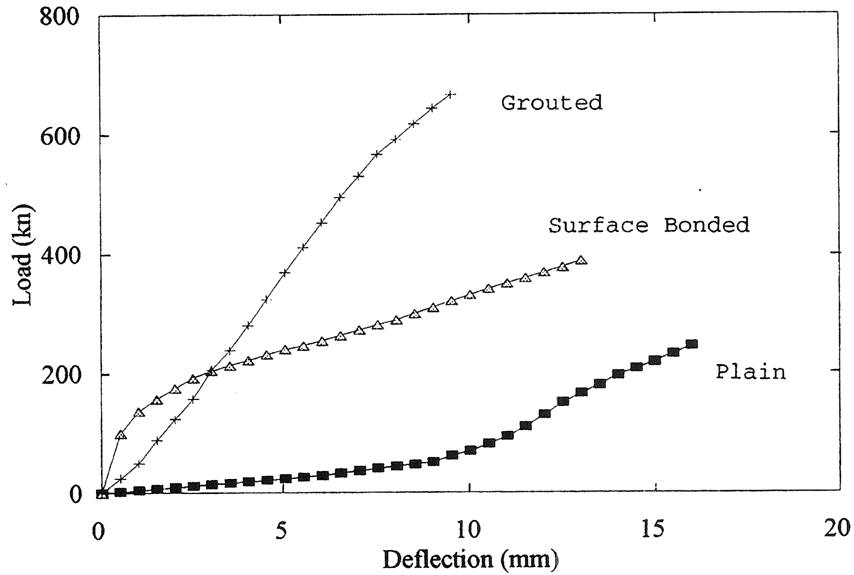


Fig. 7 Load vs. Deflection Relationships from Eccentric Axial Compression Tests.

DISCUSSION AND CONCLUSION

The results clearly indicate the structural potential for this innovative interlocking dry stacking block system. Surface bonding resulted in significant enhancement of strength and stiffness and grouting provided capacities and behaviour similar to traditional block masonry. The test data was used to develop simple design procedures for walls under combined axial load and out-of-plane bending.

Plain SparlockII walls are easy to build and, if the initial deformation can be tolerated, this provides a very quick and economical form of construction where capacity requirements are small. Otherwise, surface bonding or partial grouting (grouting only some of the cells) can provide significant improvements in both strength and stiffness. Reinforced and fully grouted walls produce strengths and stiffnesses comparable to traditional mortared block masonry.

ACKNOWLEDGEMENTS

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CEMENT BONDED WOOD PARTICLE BLOCKS FOR DRY-STACKED, SURFACE BONDED WALL SYSTEMS

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ABSTRACT

An experimental program was conducted to investigate the structural behaviour of a dry stacked surface bonded Durisol block wall system. The Durisol block is composed of cement bonded wood particles. Included in the tests were fourteen three-course prisms tested under concentric and eccentric compression loads and three eight-course walls tested in bending. The test results confirmed very significant axial compression capacities and very ductile behaviour under shear and flexural loads. The need to develop details for transferring floor loads to the wall system was identified.

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