

# **Design Guide for Masonry Reinforced by Bond Beams and Columns to Resist Lateral Load**



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## **Introduction**

This Design Guide has been developed from an extensive series of tests on full size walls, generally 8 m x 5 m (length x height) and reinforced at intervals up their height. The walls were made from concrete blockwork and, in the case of plain walls, were reinforced by Bond Beams located approximately one third and two thirds of the wall height. The Bond Beam course was a trough type concrete masonry unit containing two H16 reinforcing bars placed horizontally, one above the other, in the 'trough'. The bars fitted into metal cleats, fixed to columns at their ends and were concreted into the trough. At specified intervals, vertical shear transfer rods connected the Bond Beam to the courses above and below it. For walls with window or door openings, the Bond Beams were installed at window head and sill level, or at door head height respectively.

The results of the initial tests were very encouraging and as the system was developed, tests were carried out to better understand the performance of the beams and the significance of the contribution of the shear transfer rods. In addition, local vertical reinforced sections have been used to subdivide the length of the wall into smaller panels. BS EN 1996-1-1 (EC6) gives recommendations for the design of "locally reinforced hollow blockwork". Tests have been carried out to confirm the suitability of the provisions for this application and in this Guide locally reinforced hollow blockwork sections are referred to as Bond Columns. Although the testing programme was carried out using cleats, shear transfer rods and continuity ties of proprietary design, the remainder of the components are readily available and comply with relevant standards and certifications. Connections have been designed to connect Bond Beams and Columns to form a complete masonry system. This Guide extends the guidance in EC6 to walls reinforced by Bond Beams and Columns and is justified by the extensive test programme at the Lucideon laboratories (formerly Ceram).

The experimental work was carried out at Lucideon and sponsored by Wembley Innovation Ltd.

The production of this Design Guide has been steered by a working group consisting of:

Andrew Best	Buro Happold
Geoff Edgell	Lucideon
Neil Tutt	Jenkins and Potter
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This Guide can be used in conjunction with the original Lucideon reports, which are available from Wembley Innovation Ltd, and provide a full history of the Lucideon experimental work and test results.

## **Foreword**

This Design Guide provides guidance and recommendations for the design of masonry walls incorporating Bond Beams and Columns to strengthen walls against lateral loading. It should not be quoted as a specification and particular care should be taken to ensure that claims of compliance are not misleading.

This guidance was prepared on the assumption that the execution of its recommendations is entrusted to appropriately qualified and competent people.

References to Code of Practice guidance are to BS EN 1996. BS 5628 was withdrawn on 31 March 2010 although it will continue to be used for some time and earlier versions of this guide gave the appropriate references.

## **Note**

The systems described in this document are known commercially as Wi Beams, Wi Columns, Wi cleats, Wi shear transfer rods and Wi continuity ties, which are the subject of a number of UK and worldwide patents as described in Appendix D. They are available from Wembley Innovation Ltd on +44 (0) 20 8903 4527.

## Section 1: General

### 1 Scope

Recommendations are provided for the structural design of single leaf concrete masonry walls reinforced at determined intervals horizontally by Bond Beams, and vertically by Bond Columns to resist lateral loads. The guidance is generally limited to walls either 140 mm or 190 mm thick, although information is given on the strength of Bond Columns 215 mm thick.

**Note:** Walls designed using this Guide may not always be adequate to satisfy other design requirements e.g. resistance to fire, thermal insulation, sound insulation. Reference should be made to BS EN 1996 and PD 6697 for guidance.

It has been assumed in preparing this guidance that the design of masonry is entrusted to Chartered Structural or Civil engineers or other appropriately qualified persons, for whose guidance it has been prepared, and that the execution of the work is carried out under the direction of appropriately qualified supervisors.

### 2 References

The following documents should be referred to for the application of this Guide. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

<b>BS 4449</b>	Specification for Carbon Steel Bars for the Reinforcement of Concrete
<b>BS EN 206-1</b>	Concrete - Part 1: Specification, Performance, Production and Conformity
<b>BS EN 771-3</b>	Specification for Masonry Units - Aggregate Concrete Masonry Units
<b>BS EN 772-1</b>	Methods of Test for Masonry Units. Determination of Compressive Strength
<b>BS EN 845-1</b>	Specification for Ancillary Components for Masonry - Part 1: Ties, Tension Straps, Hangers and Brackets
<b>BS EN 845-2</b>	Specification for Ancillary Components for Masonry - Part 2: Lintels
<b>BS EN 998-2</b>	Specification for Mortar for Masonry - Part 2: Masonry Mortar
<b>BS EN 1052-2</b>	Methods of Test for Masonry - Part 2: Determination of Flexural Strength

<b>BS EN 1996-1.1:2005+A1 2012</b>	Eurocode 6 - Design of Masonry Structures - Part 1.1: General Rules for Reinforced and Unreinforced Masonry Structures
<b>BS EN 1996-1.2:2005</b>	Eurocode 6 - Design of Masonry Structures - Part 1.2: General Rules for Structural Fire Design
<b>BS EN 1996-2:2006</b>	Eurocode 6 - Design of Masonry Structures - Part 2: Design Considerations, Selection of Materials and Execution of Masonry
<b>NA to BS EN 1996-1-1:2005</b>	UK National Annex to Eurocode 6 - Design of Masonry Structures. General Rules for Reinforced and Unreinforced Masonry Structures (+A1:2012)
<b>NA to BS EN 1996-1-2:2005</b>	UK National Annex to Eurocode 6 - Design of Masonry Structures. General Rules - Structural Fire Design
<b>NA to BS EN 1996-2:2006</b>	UK National Annex to Eurocode 6 - Design of Masonry Structures. Design Considerations, Selection of Materials and Execution of Masonry (Amd Corrigendum 17207)
<b>BS EN 10111</b>	Continuous Hot Rolled Low Carbon Steel Sheet and Strip for Cold Forming: Technical Delivery Conditions
<b>DD 86-1</b>	Damp-Proof Courses - Part 1: Methods of Test for Flexural Bond Strength and Short Term Shear Strength
<b>PD 6697</b>	Recommendations for the Design of Masonry Structures to BS EN 1996-1-1 and BS EN 1996-2

### **3 Definitions**

The definitions given in BS EN 1996 apply, together with those below.

#### **3.1 Bond Beam**

A course of trough shaped units laid as a single or double course in a wall. Reinforcing bars are placed in the void with shear transfer rods connecting the beam to the masonry courses above and below. The trough units are then concreted.

#### **3.2 Bond Column**

A stack bonded column of single cell hollow concrete units which are mortared together. Reinforcing bars are placed in the void which is then concreted.

### 3.3 Shear Transfer Rods

A flat steel section bent through 90° to form a leg and foot. The rod is placed with the foot beneath the course below a Bond Beam with the leg in a cross joint. The rod passes through a hole in the base of the Bond Beam unit, the concrete infill and into the mortar in the cross joint in the course above the Bond Beam. The rod has two slots into which the horizontal bars are located to maintain their correct positioning and to avoid any sagging under their self weight prior to the placing of the concrete.

### 3.4 Continuity Tie

A flat steel section bent through 90° at one end to form a leg and doubly slotted at the opposing end. The two slots engage onto the vertical reinforcing bars of a Bond Column. The 90° leg end slots into one of the vertical slots in a Slot Block in the wall panel adjacent to Bond Column. Using the continuity tie at 450mm vertical centres at the Bond Column / panel junction generates a fixing moment in the panel, enabling it to be designed with continuous edge support.

**Note:** Details of the system are illustrated in Figures 1-7.

## 4 Symbols

For the purpose of this Guide the following symbols apply

$f_{xk1}, f_{xk2}$	characteristic flexural strengths of masonry
$f_{xd1}, f_{xd2}$	design flexural strengths of masonry
$f_{xd1,app}$	apparent design flexural strength of masonry when the plane of failure is parallel to the bed joint
$f_d$	design compressive strength of masonry perpendicular to the bed joint
$a_d$	design vertical compressive strength
$\gamma_M$	partial factor for materials
$\gamma_F$	partial factor for load

## 5 Alternative Materials and Methods of Construction

Where materials and methods are used that are not referred to in this Design Guide, their use is not discouraged, however; they are beyond the scope of this guidance, which is based upon the experimental programme. Characteristic flexural strengths for masonry outside the scope of this guidance may be determined in accordance with BS EN 1052-2.

## **Section 2: Materials, Components and Workmanship**

### **6 General**

The materials, components and workmanship used in the construction of masonry laterally loaded wall panels should conform to the appropriate clause in BS EN 1996-2.

### **7 Masonry Units**

Aggregate blocks should conform to BS EN 771-3: Aggregate Concrete Masonry Group 1 Units and have a mean compressive strength of at least  $7 \text{ N/mm}^2$ . This guide generally covers block thicknesses of 140 mm and 190 mm. Information on the strength of 215 mm thick Bond Columns is also included.

### **8 Laying Masonry Units**

Aggregate blocks should be laid on a full bed of mortar. This includes units used to form the Bond Beam and those in the course above. Joints should only be raked out or pointed when approved by the designer.

### **9 Rate of Laying**

The maximum height of blockwork that should normally be built in a day is 1.5 m.

### **10 Forming of Chases or Holes**

Chasing of completed walls or the formation of holes should be carried out only when approved by the designer and then be in accordance with the recommendations in BS EN 1996-1.1.

### **11 Damp-Proof Courses**

The provisions of PD 6697 should be followed.

### **12 Reinforcing Steel**

Reinforcing steel should be two No. H16 bars in each Bond Beam or Column and should conform to the requirements for ribbed weldable steel reinforcing bars in BS 4449.

### **13 Shear Transfer Rods**

Shear transfer rods should be manufactured using 40 mm x 4 mm mild steel to BS EN 10111. The rods are zinc coated. The material/coating reference is



No. 11 to Table A1 of BS EN 845-1. The rods should consist of an 'L' shape with a vertical leg of 610 mm and a horizontal foot of 70 mm in length. Special double length transfer rods are used for double layered Bond Beam construction and short transfer rods over wall penetrations (see Figures 4&5).

#### **14 Masonry Mortars**

The specification for masonry mortar should be in accordance with BS EN 1996-1-1. Mortar of compressive strength class M4 should be used.

**Note:** In the experimental work a ready to use, retarded mortar 1:1:6 cement: lime: sand was used.

#### **15 Concrete Infill**

The specification of concrete infill should be in accordance with BS EN 1996-1-1. C40 pre-mixed 10 mm aggregate (bagged) concrete is recommended, however, quality controlled ready mixed concrete can also be used.

#### **16 Concrete Infill and Mortar for Bond Columns**

The specification of dual purpose concrete infill / bedding mortar for the construction of bond columns should be in accordance with BS EN 1996-1-1. Concrete achieving a characteristic strength of 40N/mm<sup>2</sup> at 28 days should be used.

#### **17 Head Restraint Anchors and Frame Cramps**

Proprietary frame cramps 19 mm x 2 mm in cross section and projecting 175 mm from the frame have proven to be suitable when used with a debonding sleeve.

Proprietary internal head restraints that are bedded in the bed joint beneath the top course and which are suitable for a maximum gap above the top course of 25 mm and permit both vertical and in plane restraint have proven to be suitable.

#### **18 End Cleats**

Bond Beam end cleats should be manufactured using 200 mm x 40 mm x 8 mm mild steel to BS EN 10111. The cleats are zinc coated. The material/coating reference is No. 11 to Table A1 of BS EN 845-1. The end cleats should consist of a base plate with two welded tubes to enable insertion of H16 reinforcement bar. A PVC de-bonding sleeve fits over the tube and reinforcing bar to enable horizontal movement (see Figure 2).

## **19 Head Cleats**

Bond Column head cleats should be manufactured using 340 mm x 60 mm x 8 mm mild steel to BS EN 10111. The cleats are zinc coated. The material/coating reference is No. 11 to Table A1 of BS EN 845-1. The head cleats should consist of a base plate with two welded rods to enable location of H16 reinforcement bars with a pre-welded coupler tube. Appendix B shows the wind loads that were resisted in the various column tests. In each case the reinforcing bars were supported by the proprietary head cleats. The loads resisted were far above design loads and so no design checks are required on the cleats.

**Note:** End cleats and head cleats should generally be regarded as providing simple supports when analysing masonry bond beams and columns unless proven otherwise by calculation or experiment.

## **20 Continuity Ties**

Continuity Ties should be manufactured using 40mm x 6mm mild steel to BS EN 10111. The ties are zinc coated. The material / coating reference is No. 11 to table A1 of BS EN 845-1. The ties should consist of an L shape with a vertical leg of 106mm and a horizontal leg of 457mm.

**Note:** The use of continuity ties at the panel / Bond Column junction allows the designer to take continuous support of the panel across the column.

## **Section 3: Design objectives and general recommendations**

### **21 General**

The design of concrete blockwork walls to resist lateral loads follows the guidance given in BS EN 1996-1-1. In the case of walls containing Bond Beams and Columns, the principle is to divide the walls into sub-panels. Each sub-panel is then designed according to BS EN 1996-1-1 using the relevant flexural strengths, support conditions and height/length ratio. Flexural strengths for UK masonry are given in the National Annex and bending moment coefficients in Annex E to BS EN 1996-1-1.

The lateral loads from the masonry sub-panels are then applied to the Bond Beams, Bond Columns and other vertical supports, which are checked to see that their maximum design moments are not exceeded. When the maximum design moments are based upon the partial factors recommended in this guide, the serviceability limit states of cracking and deflection will be satisfied.

### **22 Division into Sub-Panels**

The Bond Beam may be taken as consisting of the reinforced course acting together with the courses above and below it, i.e. it is three courses deep. The sub-panel is then taken as receiving simple support at one course above or one course below the reinforced course. Alternatively, if the designer carries out a more detailed analysis and can justify continuity across all three courses, then enhanced support can be assumed; in this case the support is

considered to act at the mid height of the Bond Beam. The width of a Bond Column is considered to be the length of a single block. The support at the side of a Bond Column to a sub-panel is considered to be simple unless continuity ties as described in **20** above are used in which case full fixity can be assumed. If there is sufficient pre-compression due to self-weight of the masonry above then continuous support at the DPC at the base of the wall may be assumed. Alternatively, flexural tension should only be relied upon at the damp proof course if it has been justified by tests (see DD 86-1). If the damp proof course is provided by damp-proof course bricks continuous support may be assumed. If flexural tension cannot be relied upon at the damp-proof course then simple support should be assumed.

The designer will need to consider whether the head restraint can provide continuous support and if not should assume simple support.

**Note:** In the experimental work, simple support was generally achieved at the head of the wall. Where attempts were made to provide moment restraint, cracking tended to occur prematurely along the bed joint at the base of the top course.

## **23 Limiting Dimensions**

The limiting dimensions for walls set out in Annex F of EN 1996-1-1 should be observed, in order to avoid undue movements due to deflections, creep, shrinkage, temperature effects and cracking.

The limiting dimensions of the sub panels which contain bed joint reinforcement should be in accordance with PD 6697.

### **23.1 Limiting Height of Columns**

The design data in this guide is based upon a maximum column height of 7m. This should only be exceeded with careful consideration.

## **24 Compressive Strength of Blockwork**

When using the materials specified for this form of construction there is no need to check against compression failure.

**Note:** Tests show that Bond Beams and Bond Columns fail in flexure after extensive cracking and have effectively failed by excessive deflection before fairly slow and localised compression failure occurs.

## **25 Characteristic Flexural Strength of Concrete Masonry**

The characteristic flexural strength of masonry for use in design ( $f_{xk1}$ ,  $f_{xk2}$ ) may be determined by tests according to BS EN 1052-2.

Alternatively, the value may be determined from Table 3 below which is derived from Table NA6 of the National Annex to BS EN 1996-1-1. These values have been verified by test.

**Table 3 - Characteristic Flexural Strength of Masonry,  $f_{xk1}, f_{xk2}$  N/mm<sup>2</sup>**

Mortar Strength Class/Designation	Wall Thickness (mm)	Plane of Failure Parallel to Bed Joints, $f_{xk1}$	Plane of Failure Perpendicular to Bed Joints, $f_{xk2}$
M4	100	0.25	0.59
	140	0.22	0.52
	190	0.19	0.44
	215	0.17	0.40

**Note:** Tests to determine the compressive strength of concrete blocks should be in accordance with BS EN 772-1.

## 26 Partial Factors

It is assumed that the recommendations for workmanship given in BS EN 1996-2 are followed including appropriate inspection and supervision.

In this case the partial factor for materials,  $\gamma_M$ , may be taken as 2.0 for the design of the sub-panels. If this level of control cannot be achieved a value of 2.4 should be used. These values are consistent with the 2013 revision to the National Annex to BS EN 1996-1-1. They apply to panels when removal of the panel would in no way affect the overall stability of the building.

The mortar should either a) comply with the requirements of BS EN 998-2 or b) be a site mixed mortar where preliminary compression strength tests are carried out in accordance with BS EN 1015-2 and BS EN 1015-11 to indicate that the strength requirements of BS EN 1996-1-1 are met and regular testing of the mortar used on site in accordance with BS EN 1015-2 and BS EN 1015-11 show that these requirements are being met.

As the recommendations for workmanship given in BS EN 1996-1-1 are assumed to be followed including appropriate inspection and supervision for the reinforced elements the partial factor for materials,  $\gamma_M$ , can be taken as 2.0. If this recommendation is followed the requirements for the serviceability limit states of deflection and cracking will be met.

## 27 Design of the Sub-Panels

The design procedure for the sub-panels should follow the provisions of Clause 5.5.5 and 6.3 of BS EN 1996-1-1, although it should be noted that if sub-panels are formed by Bond Beams only, they are normally designed as spanning one way, vertically. In particular, it should be noted that where a sub-panel has a height: length ratio of less than 0.3 it should be designed as spanning vertically. Where a panel has a height: length ratio of greater than 2.0 it should be designed as spanning horizontally.

The vertical load in the sub-panel acts so as to increase the flexural strength normal to the bed joint and the design strength may be modified to  $f_{xd1,app} = f_{xd1} + a_d$  where  $a_d$  is the design stress due to vertical load (including self-weight) normal to the bed joint,  $a_d$  is not to be taken as greater than  $0.2 f_d$ .

## 28 Design of Bond Beams

### 28.1 Design for Bending

The ultimate bending moment for the Bond Beam has been derived through testing, and is given in Table 4. The basis of this assertion is given in Appendix A. This should be modified by the partial factor for materials,  $\gamma_M$ , (2.0) to give the maximum design bending moment for 140 mm and 190 mm Bond Beams.

**Table 4 - Maximum Bending Moments for Bond Beams of Different Thickness**

Wall Thickness (mm)	Ultimate Bending Moment (kN-m)	Maximum Design Bending Moment (kN-m)
140	60	30
190	80	40
215	80	40

The bending moment applied to the Bond Beam should be calculated assuming that the Bond Beam is loaded by the characteristic wind loads on the adjacent masonry sub-panels, modified by the appropriate partial factor for loads of 1.5 and the Bond Beam is considered to be simply supported.

As shown in Appendix A, when the recommended partial factors are used, there is no need for a further check against cracking or excessive deflection.

### 28.2 Design for Shear

The shear resistance of bond beam sections may be to Annex J of BS EN 1996-1-1 using the enhancement for longitudinal steel. In general design for shear is not required as the load transferred to the beam from the sub panels is limited by their own capacity.

## 29 Design of Bond Columns

### 29.1 Design for Bending

The ultimate bending moment for Bond Column sections has been derived through testing and this is given in Table 5. The basis of this assertion is given in Appendix B. This should be modified by the partial factor for materials,  $\gamma_M$ , (2.0) to give a maximum design bending moment for the section. The bending moment applied to the section should be calculated assuming

that the section is loaded by the characteristic wind load on the adjacent masonry sub-panels modified by the appropriate partial factor for loads of 1.5 and the Column is considered to be simply supported.

**Table 5 - Maximum Moments for Bond Columns of Different Thickness**

Wall Thickness (mm)	Ultimate Bending Moment (kN-m)	Maximum Design Bending Moment (kN-m)
140	36	18
190	44	26
215	60	30

As shown in Appendix B, when the recommended partial factors are used, there is no need for a further check against cracking or excessive deflection as the limiting bending moments are based upon a serviceability criterion, a lower moment due to the limitations of the testing or a factored ultimate load at which serviceability limits are met.

## 29.2 Design for Shear

The shear resistance of bond column sections may be to Annex J of BS EN 1996-1-1 using the enhancement for longitudinal steel. In general design for shear is not required as the load required to be transferred to the column from the sub-panels to reach its shear capacity will not be reached by the design air pressure. Comparison with pressures achieved in the test programme demonstrate how conservative this approach is.

## 29.3 Lintels

Where bond beams are required to act to resist both in plane and out of plane loads, for example as lintels it is reasonable to follow the procedure in BS EN 845-2.

The load capacity is generally controlled by the value in flexure. Table 6 and 7 recommends some values for design.

**Table 6 - Load Capacity of WI Lintels in flexure 1.8m clear span**

Flexural tests were undertaken to determine the failure load and mode for 1800 mm span WI Lintels, with detailed results given in Tables 1-4, Appendix C

Block Size (mm)	Load Capacity (kN)
	Flexural Failure
100	150
140	160
190	200
215	200

**Table 7 - Load Capacity of WI Lintels in shear 1.35m clear span**

Shear tests were undertaken to determine shear failure and mode for 1350mm span WI Lintels, with detailed results given in Tables 5-8, Appendix C

Block Size (mm)	Load Capacity (kN)
	Shear Failure
100	38.0
140	50.0
190	65.0
215	70.0

The experimental results are included in Appendix C

### **30 Detailing**

The detailing of the restraints, bar placement etc. shall be such that the design strength of the wall can be achieved.

#### **30.1 Head Restraints and Frame Anchors**

In general, manufacturers' recommendations should be followed. Frame anchors should be used to restrain the sides of the wall and spaced vertically at maximum 450 mm centres. Head restraints should be placed no more than 900 mm apart horizontally at the head of the wall.

#### **30.2 Shear Transfer Rods**

Shear transfer rods shall be placed at 900 mm centres. These are placed in the cross joints of the courses above and below the Bond Beam.

#### **30.3 Location of Reinforcement**

The location of the end cleats and any intermediate supports, e.g. at the transfer rods, should be so as to ensure sufficient clearance from the side of the Bond Beam block for adequate cover and compaction of concrete infill. In both the 140 mm and 190 mm wide by 215 mm high Bond Beam configurations, two H16 bars should be placed vertically above one another with equal spacings between the base of the unit and the first bar, the two bars and the second bar and the top of the unit, as shown in Figures 1 & 4. The sole difference is that for the 190 mm blocks, the cover to the side of the bars is increased. The provisions of BS EN 1996-1-1 regarding cover, bar spacing etc, should be followed.

In the case of Bond Columns, two vertical H16 bars should be used and the provisions of BS EN 1996-1-1 regarding cover, bar spacing and void filling should be followed.

## **31 Workmanship**

Workmanship should generally conform to the requirements of BS EN 1996-2.

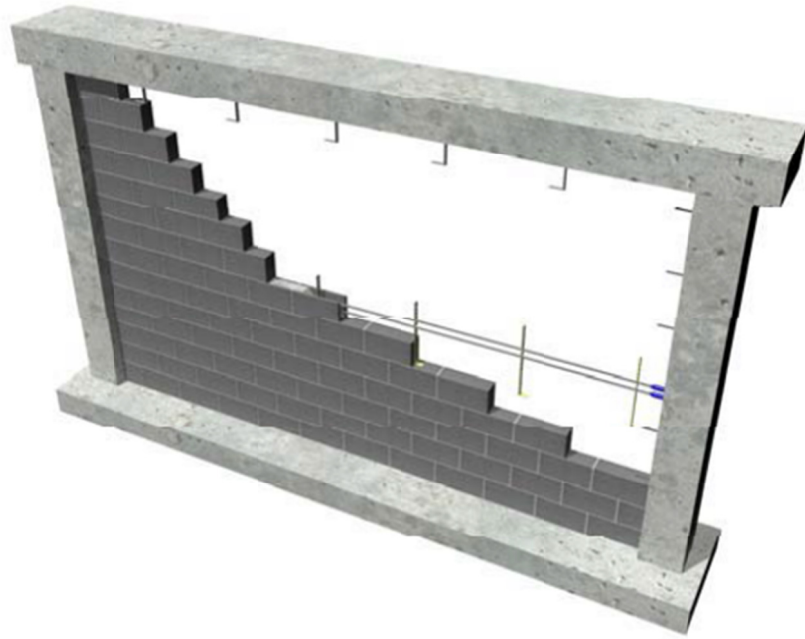
Concrete infill for Bond Beams should be in accordance with **15** and for Bond Columns with **16**. Special care should be given to the workability of concrete and the height of pour to ensure complete filling without spillage on the face of the units.

Reinforcement should be in accordance with **12** and fixed as shown on the detail drawings.

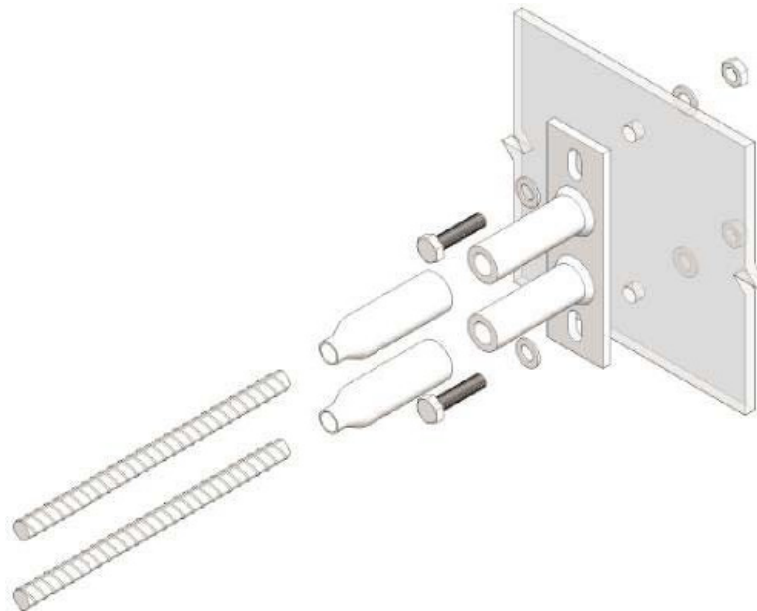
Care should be taken to ensure that the specified cover to the reinforcement is maintained, e.g. by insertion of reinforcing bars into the slots in the transfer rods. Where spacers are used, they should be of such a type and the reinforcement so positioned that compaction of the infill concrete is not prevented.

Reinforcement should be free from mud, oil, paint, retarders, loose rust, loose mill scale, snow, ice, grease or any other substance that may adversely affect the steel or concrete chemically, or reduce the bond. Normal handling prior to embedment is usually sufficient for the removal of loose rust and scale from reinforcement.

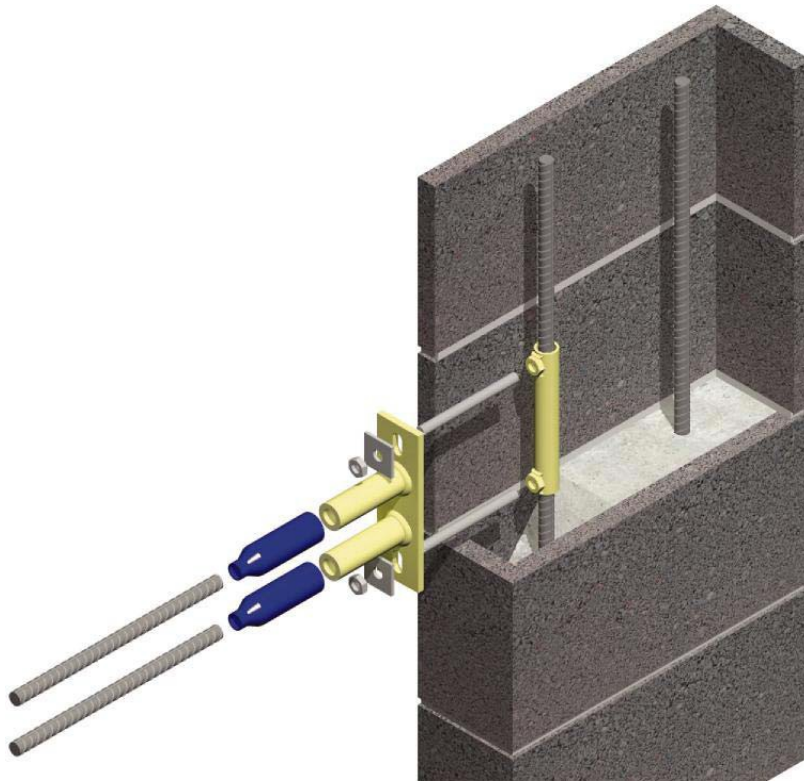




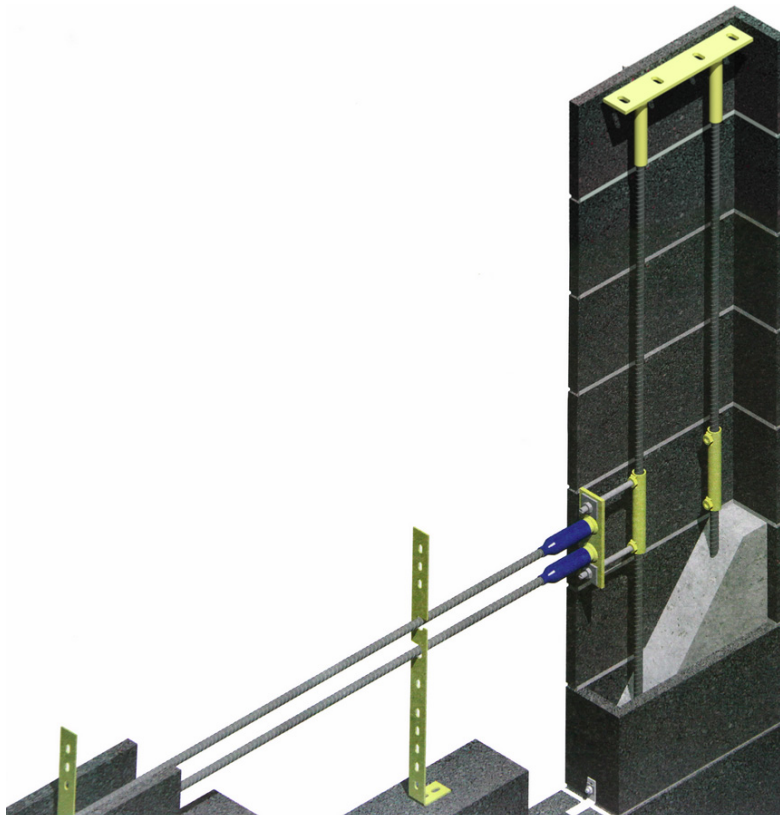
**FIGURE 1.** Bond Beam System



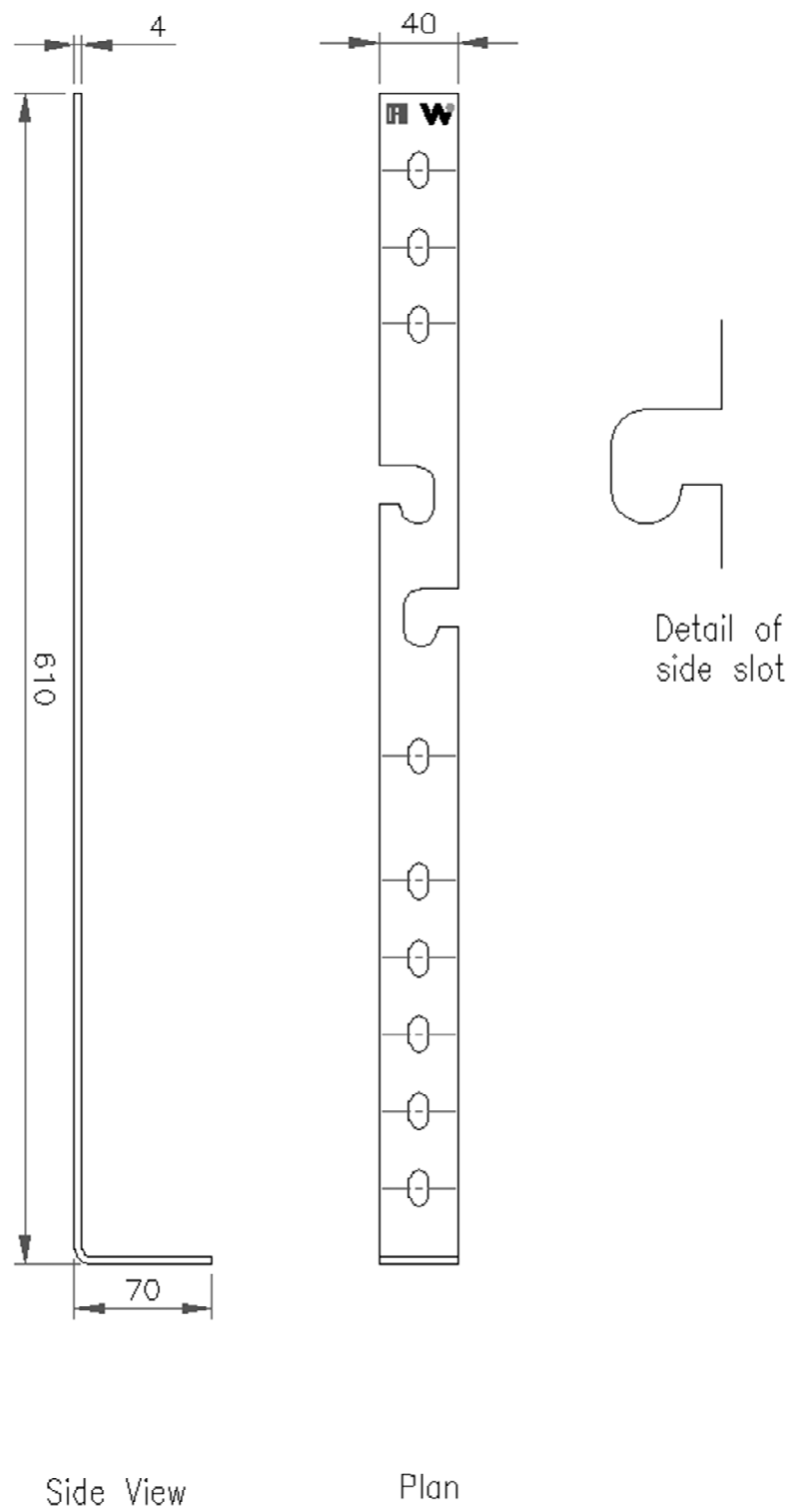
**FIGURE 2.** Exploded View of Cleat



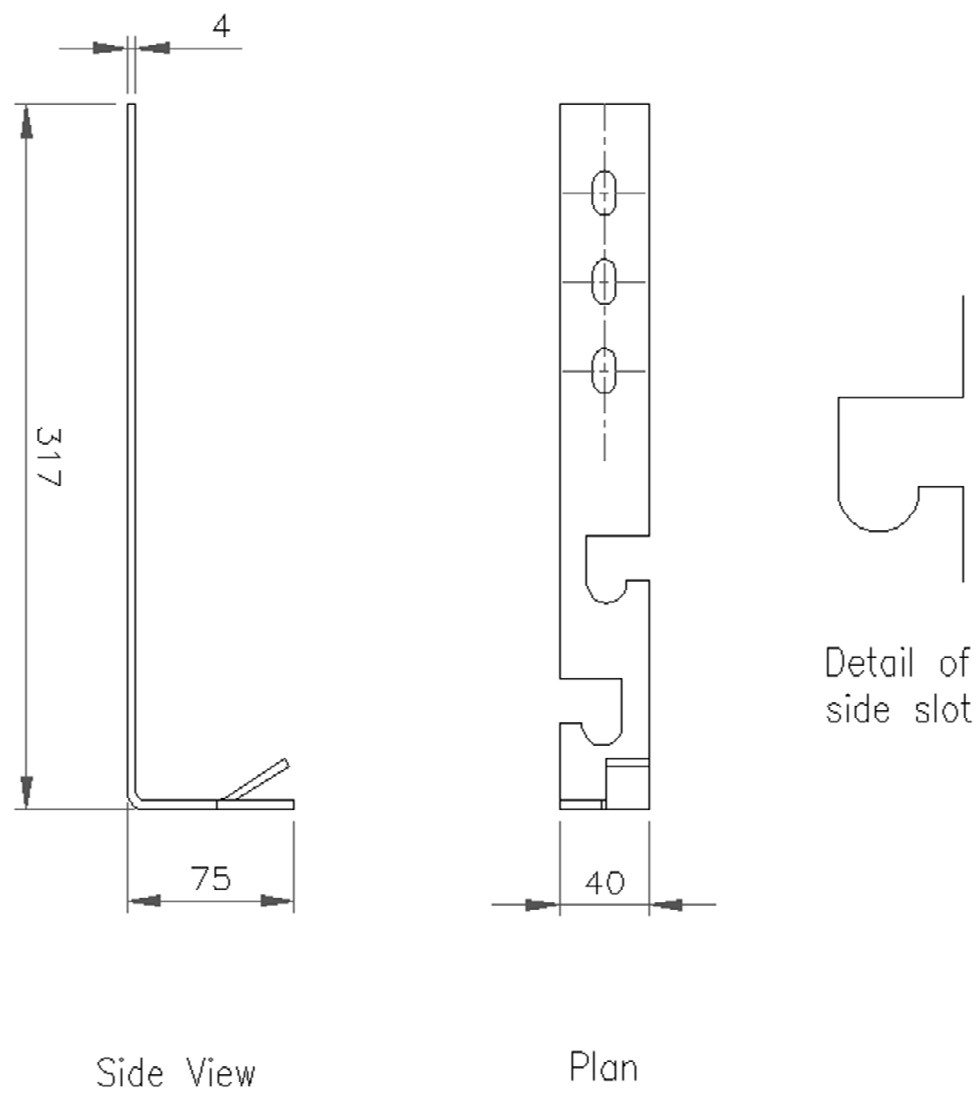
**FIGURE 3.** Bond Column System



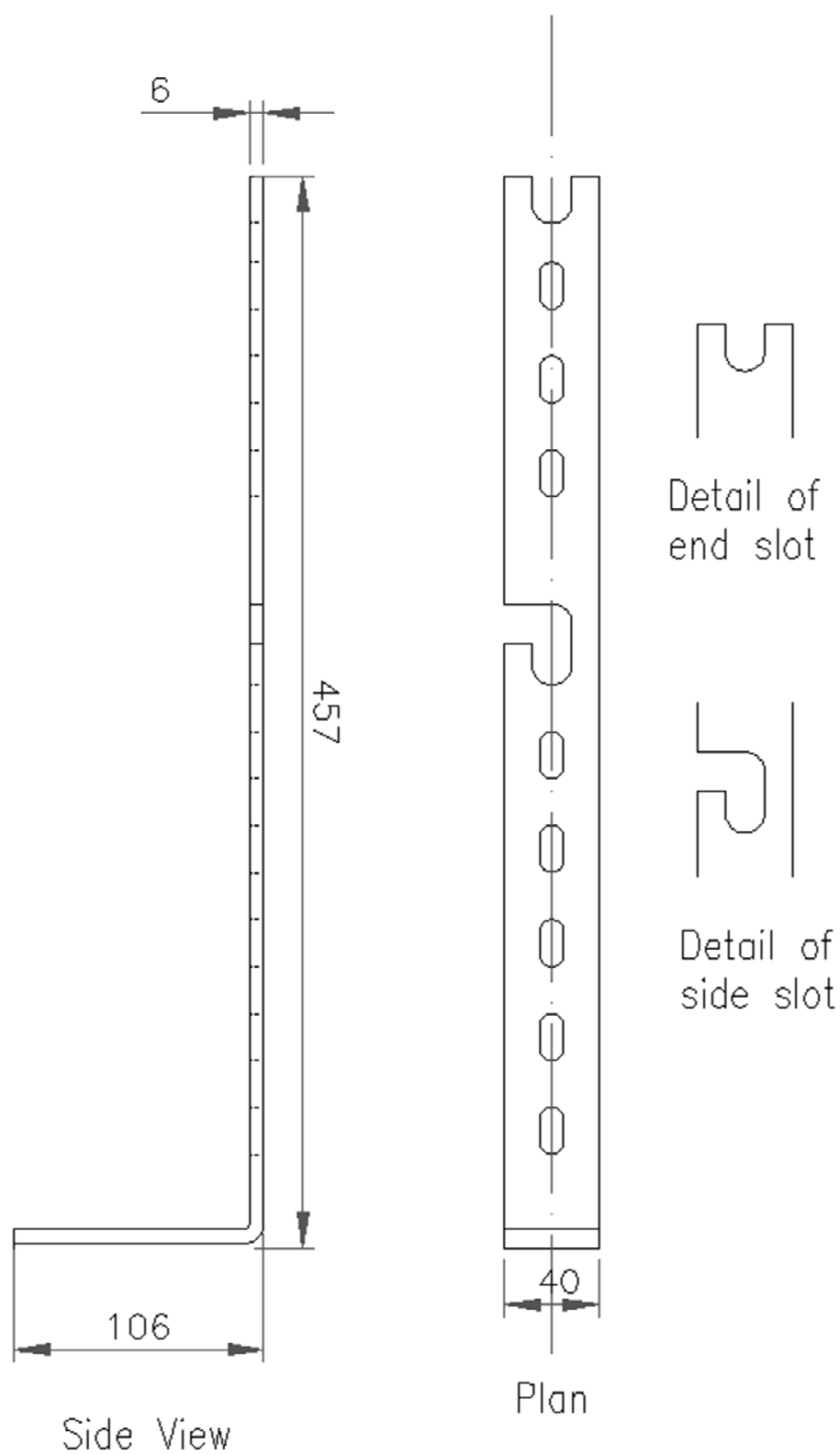
**FIGURE 4.** Complete Beam and Column System



**FIGURE 5.** Shear Transfer Rod



**FIGURE 6.** Short Shear Transfer Rod



**FIGURE 7.** Continuity Tie

## Appendix A: Bond Beam Test Data

### 140 mm Thick Walls

During the experimental work of Phase (V), two low height (1.1 m) walls containing a single Bond Beam with pinned end supports were built on a slip plane and loaded with a uniformly distributed load over their face area until failure occurred. The results of these two tests are given in Table A1.

**Table A1 - Test Phase V - Experimental 140 mm Bond Beam Results  
with Pinned End Cleat Connection**

Wall No.	Load at First Crack (kPa)	Maximum Load (kPa)
2	2.4	8.54
3	2.6	7.05

From these results, the characteristic maximum load on the Bond Beam had been taken to be 7.05 kN/m<sup>2</sup>. The wall in this case spans 7.9 m and assuming the ends are simply supported the maximum allowable bending moment resistance may be taken to be 60 kN-m.

Applying the partial factors,  $\gamma_M = 2.0$  and  $\gamma_F = 1.5$ , the design maximum load is derived as 2.35 kN/m<sup>2</sup> which is lower than either of the loads in the tests of 2.4 kN/m<sup>2</sup> and 2.6 kN/m<sup>2</sup> to cause cracking.

From the load vs. deflection curves in the Phase (V) report, the deflection at 2.04/m<sup>2</sup> is 14 mm (wall 3) which is span/564. The general criterion for deflection is that the final deflection will not exceed span/250, including long term effects. In order to avoid damage to finishes it is recommended that lateral wall deflections, after they are constructed, do not exceed span/500. It is concluded that the deflections recorded in the tests are small enough that the serviceability limit state of deflection will be satisfied.

Phase (VII) of the work included two further low height (1.1 m) walls in this case also with a single central Bond Beam. The difference between this and Phase (V) is that the cleats were firmly mounted onto the columns and in Phase (V), they had been set from the column face by bolted connections. The span in this case was 8.1 m. The results were as follows:

**Table A2 - Test Phase VII - Experimental 140 mm Bond Beam Results  
with Fixed End Cleat Connection**

Wall No.	Load at First Crack (kPa)	Maximum Load (kPa)
1	4.0	9.0
2	4.2	11.6

In this case, which more accurately models real life detailing, the greater fixity had clearly an effect in delaying cracking and allowing greater loads to be achieved. Using the same partial factors as above gives an estimate of the characteristic load as  $3.0 \text{ kN/m}^2$  and  $3.9 \text{ kN/m}^2$  which are again below the cracking loads. The deflections at these loads were  $\text{span}/485$  and  $\text{span}/2410$ . This additional data enables an enhanced, but still conservative, design bending moment of  $30 \text{ kN-m}$  to be determined for the  $140 \text{ mm}$  Bond Beam.

### 190 mm Thick Walls

During phase (IX) three  $190 \text{ mm}$  thick Bond Beam walls each  $8.1 \text{ m} \times 1.1 \text{ m}$  were tested and produced a wide variance in first crack and ultimate loads. The results of these three tests are given in Table A3.

**Table A3 - Test Phase IX - Experimental 190 mm Bond Beam Results  
with Fixed End Cleat Connection**

Wall No.	Load at First Crack (kPa)	Maximum Load (kPa)
1	10.8	21.1
2	14.9	21.7
3	3.0	5.5

The results from the first two walls gave loads at first crack of  $10.8 \text{ kN/m}^2$  and  $14.9 \text{ kN/m}^2$  respectively, with failure loads close to  $22 \text{ kN/m}^2$ . Due to loading and construction issues with the third wall a very conservative view has been taken of these results.

The maximum allowable bending moment has been taken to be  $190/140$  times that for the  $140 \text{ mm}$  wall, i.e.  $80 \text{ kN-m}$ . This equates to a load of  $11.1 \text{ kN/m}^2$  so is close to the minimum recorded load at which the wall first cracked. Bearing in mind that this would mean the design load would be limited to  $4.8 \text{ kN/m}^2$  and taking into account  $\gamma_F$  the characteristic lateral load would be limited to  $3.2 \text{ kN/m}^2$  which is below the lowest cracking load experienced and close to that experienced by wall 3 in Table A3.

Consequently the recommended values are very conservative and there is clearly no need for a serviceability check.

## Appendix B: Bond Column Test Data

### 140 mm Thick Columns

During Phase (VIII) of the experimental work, four columns, each 140 mm thick, 0.89 m wide and either 2.7 m or 5 m high were tested under lateral load.

The columns consisted of one reinforced block with two 16 mm reinforcing bars, either one in each of the blocks two cells or two spaced similarly within a single rectangular cell. The results are given in Table B1.

**Table B1 - Test Phase VIII - Experimental Results for 140 mm Thick Columns**

Column No.	Cell Type	Height (m)	Maximum Load (kN/m)	Failure Mode	Maximum Bending Moment (kN-m)
3	Double	2.7	35.6	Air Bag	32.4
4	Single	2.7	32.0	Air Bag	29.2
5	Double	4.95	8.45	Air Bag	25.9
6	Single	4.95	10.45	Section Failure	31.9

Two further tests were carried out in Phase (IX) on 140 mm columns and a nominal height of 7 m. In this case specially reinforced air bags were used to overcome the earlier problems of the bags bursting. In this case, both tests gave ultimate failure loads of 7 kN/m<sup>2</sup>. Ultimate failure was taken to be excessive deflection (span/100) although there was no physical indication of failure. These two columns are numbered 7 and 8 and the failure data is given in Table B2.

**Table B2 - Test Phase IX - Experimental Results for Extra 140 mm Columns**

Column No.	Cell Type	Height (m)	Maximum Load kN/m	Failure Mode	Maximum Bending Moment (kN-m)
7	Single	7.0	6.23	Deflection	39.9
8	Single	7.0	6.23	Deflection	39.9

The mean results from the 140 mm columns that were classed as failing, i.e. 6, 7 and 8 was 37.2 kN-m. Although one failure moment is below the value, it is suggested that an ultimate moment of 36 kN-m be adopted. This view has been reached based upon the difficulty of air bag failure, the limitation of failures to deflection giving a conservative result and the results on the thicker columns. This gives a design moment of resistance of 18 kN-m, based on a partial factor of 2.0. Taking the partial factor for actions as 1.5 the deflections at the characteristic bending moment would be as given in Table B3.



**Table B3 - Test Phases VIII and XI - Serviceability Deflections**

Column No.	Deflection (mm)	Ratio to Span
3	3.8	710
4	4.0	675
5	5.0	990
6	8.0	614
7	2.4	2901
8	1.7	4057

Clearly in all cases the deflections are very small and occur long before any sign of cracking, even for wall 6, consequently, the design bending moment resistance can be taken as 18 kN-m, no serviceability check is needed and the maximum column height may be increased to 7 m.

### 190 mm Thick Columns

During Phase (VIII) of the experimental work a further four 190 mm thick columns were tested. The results are given in Table B4.

**Table B4 - Test Phase VIII - Experimental Results for 190 mm Thick Columns**

Column No.	Cell Type	Height (m)	Maximum Load (kN/m)	Failure Mode	Maximum Bending Moment (kN-m)
1	Single	2.7	35.3	Air Bag	32.2
2	Double	2.7	34.9	Air Bag	31.8
7	Double	4.95	22.6	Air Bag	61.6
8	Single	4.95	35.1	Air Bag	107.5

All of the above results (Table B4) show different maximum bending moments due to ultimate failure of the loading air bags. Both of the 2.7 m high columns achieved similar maximum bending moments of approximately 32 kN-m. There was variation in the 5 m high column results which has led to the development of a conservative alternative design approach (Table B5). This shows the applied test loads and bending moments achieved at a deflection of span/500.

**Table B5 - Test Phase VIII - Actions to Cause Limiting Deflection**

Column No.	Span/500 (mm)	Load (kN/m)	Bending Moment (kN-m)
1	5.4	26.7	32.2
2	5.4	24.6	31.8
7	9.9	6.5	19.9
8	9.9	4.0	12.3

The results here show a fairly homogeneous set of bending moments with column 8 giving a slightly low result. If we take column 8 as the critical one, here a design bending moment of  $\gamma_F \times 12.3$ , i.e. 18.4 is indicated. Clearly given the other results if this is taken as 19 kN-m, it provides a conservative limit. In addition this is very close to that which might be expected from the 140 mm columns where the lowest actual measured design bending moment was 16.0 kN-m and simply multiplying this up by the wall width ratio 190/140 gives a value of 21.7 kN-m and at that value a serviceability check is not needed. As all the maximum bending moments were limited by the failure of the loading air bags and given the extraordinary strength of column 8, the critical one, a value of 22 kN-m can be recommended as the design bending moment.

**Note:** The experimental data in Tables B1 and B4 on the columns relates to those where an internal web separated the two formed voids and those where it had been removed (referred to single and double cell type). The preferred method of construction is to now use units with a single formed void and no internal web.

### 215 mm Thick Columns

In Phase (IX) of the work two columns 215 mm thick and 7 m high were tested. The results are given in Table B6.

**Table B6 - Test Phase IX - Experimental Results for 215 mm Thick Columns**

Column No.	Height	Maximum Load (kN/m)	Maximum Bending Moment (kN-m)
9	7.16	10.68	68.4
10	7.16	10.23	65.5

From these results it is recommended that the ultimate bending moment be taken conservatively as 60 kN-m which gives a design moment of 30 kN-m. At the characteristic load of 3.5 kN/m<sup>2</sup> the deflections at a maximum of span/450 are relatively small. In neither case was there any indication of cracking, and hence a serviceability check is not needed.

## Appendix C: Lintel Results

**Table 1 - Summary of Results for WI Lintel Flexural Testing of 100 mm Blocks  
1.8m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	162	Flexural Failure

**Table 2 - Summary of Results for WI Lintel Flexural Testing of 140 mm Blocks  
1.8m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	194	Flexural Failure

**Table 3 - Summary of Results for WI Lintel Flexural Testing of 190 mm Blocks  
1.8m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	254	Flexural Failure

**Table 4 - Summary of Results for WI Lintel Flexural Testing of 215 mm Blocks  
1.8m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	222	Flexural Failure

**Table 5 - Summary of Results for WI Lintel Shear Testing of 100 mm Blocks  
1.35m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	43.59	Shear Failure
2	48.00	Shear Failure
3	38.80	Shear Failure
Mean	43.46	-

**Table 6 - Summary of Results for WI Lintel Shear Testing of 140 mm Blocks  
1.35m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	54.09	Shear Failure
2	70.00	Shear Failure
3	72.00	Shear Failure
<b>Mean</b>	<b>65.36</b>	-

**Table 7 - Summary of Results for WI Lintel Shear Testing of 190 mm Blocks  
1.35m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	80.37	Shear Failure
2	66.64	Shear Failure
3	68.80	Shear Failure
<b>Mean</b>	<b>71.93</b>	-

**Table 8 - Summary of Results for WI Lintel Shear Testing of 215 mm Blocks  
1.35m clear span**

Test No.	Failure Load (kN)	Failure Mode
1	74.85	Shear Failure
2	74.82	Shear Failure
3	77.37	Shear Failure
<b>Mean</b>	<b>75.68</b>	-

## Appendix D: Wi Beam and Column Patents

“Wi Column”, “Slot Block”, “U-Block”, “Wi Beam and the “Wi” device are registered Community trademarks of Wembley Innovation Ltd. The Wi Beam / Wi Column building system and its components are the subject of a number of Community design registrations (CDRs), patents and patent applications, including CDRs 000881263-0001 to 0005, 000992136-0001, 002391136-0001 to 0002 and 1126635-0001; **005810579-0001 to 0004**; UK patent nos. 2440531, 2442543, 2054563, 2313575, 2469272, 2250323, 2485397, 2726735, 2313575, 2013366093 and 2509149, 2935715, 2892704, 1711069.3; International patents 2007280305, 2659536, 575214, 9127449, 2054563, 2009254997, 2726735, 590165, 2313575, 2013366093, 708518, 9523194, 2935715, and applications 2892704, 1711069.3

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