THE DEVELOPMENT OF MASONRY REINFORCED BY BOND BEAMS AND BOND COLUMNS TO RESIST LATERAL LOAD

Geoff, Edgell¹; Andrew, Best²

¹ PhD, Director, CERAM, geoff.edgell@ceram.com
² Group Director, Buro Happold, andrew.best@burohappold.com

In 2009 the Design Guide for Masonry Reinforced by Bond Beams to resist Lateral Loads was published. This represented the culmination of a series of tests on full size walls, small beams and low height walls. The tests demonstrated that large walls could be subdivided into smaller panels by the use of bond beams and that the lateral load resistance was considerable and comparable to walls subdivided by wind posts. The system has now been further developed to include the use of reinforced hollow blockwork columns, which enables walls to be subdivided by both horizontal and vertical reinforced elements. This paper describes the column tests and the development of a revised and extended design guide. A major application of the system is at the Aquatic Centre for the 2012 Olympic Games in London. In this iconic structure the internal blockwork walls are up to 7 m high and are required to accommodate numerous openings for services. The system enables this to be done in an elegant and efficient way and the paper describes how this was achieved.

Keywords: blockwork, lateral loading, bond beams, columns

Theme: Innovative products and system

INTRODUCTION

In 2006 Ceram were approached by a major masonry contractor, who had developed a means of reinforcing large blockwork walls with a view to eliminating wind posts. The system essentially consisted of a horizontally reinforced blockwork course (a bond beam), at intervals up the height of the wall, so as to subdivide the wall into a number of smaller vertically spanning panels see Figure 1. The motivation for the development was the fact that wind posts were relatively difficult to install and the tendency seemed to be to use them at ever closer horizontal spacing, which had an inevitable upward effect on prices. As a result, a series of laboratory tests were commissioned to investigate the structural performance of the system.
BACKGROUND
Initially four walls were tested. These were each 8 m long, 5 m wide and 140 mm thick, solid aggregate concrete blockwork. Bond beams were introduced at approximately one third and two thirds of the wall height. Each contained two 16 mm diameter high yield reinforcing bars, placed one above the other at the midpoint of the wall, placed within a series of trough shaped units, which were subsequently concreted, with 40 N/mm² concrete. The walls were built within a steel frame and the reinforcing bars fitted into simple cleats attached to the columns. Shear transfer rods connected the bond beam to the course above and below. An early test wall is shown in Figure 2 and the typical loading arrangement in Figure 3. Two further walls containing wind posts at the wall centre line were tested for comparison purposes. One wind post was a 100 mm box section and the other, an 80 mm section, placed in a vertical hollow blockwork void, i.e. it was within the wall thickness.

The initial results were very encouraging, in that the walls with the bond beams gave similar results to those with the wind posts. However, the details that were used, for example, inclusion of bed joint reinforcement and closer than usual spacing of ties, meant that the designs were deliberately conservative. As a result a second, similar phase was carried out where these conservative measures were not taken. The results were similarly encouraging, in fact, slightly improved in the case of the bond beams, one of the walls having been extended to a 12 m horizontal span. There was no evidence of cracking due to shrinkage.
One key observation in these initial phases of the work was that when the walls containing the wind posts failed, they did so by one half of the wall cracking. However, the crack pattern was one half of that which would have been expected in a wall not subdivided by the wind post. This seems to suggest that the wind posts were providing an improvement in resistance to wind load, but were not fully subdividing the panel as is assumed in design. It is probably
that the stiffness of the post in a fairly tall wall is not sufficient to alter the failure crack pattern.

In total, some eighteen wall panels were tested, supplemented by eighteen smaller beam tests. The details of the full programme are given elsewhere.

The design approach for the sub-panels between the bond beams is straightforward and follows the principles of BS 5628-1 or EN 1996-1 and PD 6697. However, design of the bond beams to BS 5628-2 proved to be more difficult, in particular, as for the 140 mm thick blockwork, the effective depth of the beam is only 70 mm. This led to the check against a sudden compression failure, which is incorporated in BS 5628-2, controlling the design and would have led to span limitations. There was no sign of sudden compression failure in any of the tests, and consequently a series of tests on low height walls was undertaken, to determine some limiting bending moments that could be used for design. From the four tests carried out, the lowest result was used to define a maximum ultimate bending moment, for use in design, which was such that, when taken with the recommended partial safety factors, ensured that no cracking occurred and deflections were limited, and hence a serviceability check was not required.

The specification of materials, design and execution were all brought together, in a Design Guide, which was published by Ceram in 2009. This was developed with collaboration from practicing engineers and the sponsoring contractor, and so although retaining a perceived authoritative and independent approach, it included realistic and achievable guidance.

INITIAL APPLICATION
The first significant application of the bond beam system was in a large Data Centre constructed in South East England. The Centre was constructed as a robust two storey steel and concrete ‘bunker’ built entirely within a large steel framed building. The interior was broken down into smaller rooms, with a requirement that a fire or explosion in any room did not spread into the adjacent areas.

The solution adopted for constructing the walls was to use long runs of 140 thick block walls and a design lateral load of 0.5 kN/m². These walls were typically 6 m high, and the original design featured bed joint reinforcement in every course plus 200 x 200 Square Hollow Section windposts at maximum 4.5 metre centres. The total amount of blockwork used was 11,000 m².

The governing factor for the spacing of the wind posts was the ability of the blockwork walls to span laterally under the 0.5 kN/m² lateral load. The introduction of bond beams substantially increased the allowable span of the walls. In most cases this allowed the walls to span the 8.5 metres between primary steel columns, eliminating interim wind posts. In total some 650 wind posts were omitted, although there were still some windposts required adjacent to doorways and on unusual spans etc. The masonry contractor reported that the use of the bond beam system achieved cost savings of approximately 15% over a traditional wind post system.

This Data Centre was an ideal use for the bond beam system, as it featured long, uncomplicated runs of high walls, loaded by a moderately high lateral load. It is interesting to compare this to construction of the Athletes Village for the 2012 London Olympics,
constructed soon after the completion of the Data Centre. The Athletes village buildings were concrete framed, and made widespread use of blockwork for facade and internal wall elements. The cellular nature of the rooms and low lateral loads meant that few of the walls required reinforcement, and whilst it was possible to replace most of the few wind posts this was not the ideal project for the system.

The bond beam system was very successfully used on the Data Centre, but that project also pointed out the shortcomings of the system. Whilst the bond beams allowed for a much wider spacing of wind posts, it was not possible to entirely omit them. The obvious next step in the development of the product was to investigate a vertical version of the bond beam.

**COLUMN TESTS**
Column tests were carried out on what were essentially locally reinforced hollow blockwork. The columns were 0.89 m long and either 3 m or 5 m high, in both 140 mm blockwork and 190 mm blockwork. Two block sections were used, either one; with two formed voids, separated by a central web and one where the central web was removed. In each case two vertical steel bars were used, but the system eventually was designed about the single voided block.

In the case of the 140 mm blocks, only one of the four columns failed by the section failing, in the remaining three, the air bags in the loading system failed prematurely. The ultimate failure moment, was based on that which actually failed, although one which subsequently failed by an air bag, burst, did so at a higher bending moment.

Consequently, the design bending moments to be used in design, were fixed based upon these columns, and a check at working loads, showed that the deflection in all cases was acceptably low.

In the case of the 190 mm columns, all of the failures were by the air bags bursting, and so although the actual failure moments were in some cases extremely high, they did not really provide a sound basis for fixing a limiting ultimate moment.

Consequently, the moments to achieve a span/500 deflection were determined and the lowest value used to define the maximum moment permitted in service. This is a very conservative approach, and it was anticipated that it would be revised upwards in the light of more relevant test results.

The completed system now consists of both beams and columns, shear transfer rods, cleats to fix reinforcing bars to building columns, and to the vertical bars in the columns. All of the details, together with the limiting moments to be used for both beams and columns are available in a revised design guide.

**APPLICATION AT THE 2012 AQUATICS CENTRE**
The first application for the completed system occurred on the London 2012 Aquatic Centre. This is an architecturally impressive building constructed for the Olympic Games, and it features a large lower level containing large areas of plant rooms, changing areas and various other back of house functions see Figure 4.
The lower level is divided up into the various functions by 9,500 m² of 140 mm thick blockwork walls. The design of the walls was complicated by several factors:

- The lower level is a cavernous space and most of the walls were 6-7 m high.
- The designers had specified a lateral design load of 0.5 kN/m², with higher loads at balustrade level along escape corridors.
- Many of the walls were not full height and hence, had no restraint at their head.
- There were large amounts of large services distributed at high level in the spaces, creating many penetrations through the full height walls.

The original design proposals featured bed joint reinforcement, wind posts, and a large amount of head restraint connections to the ceiling slab. For the partial height walls these head restraints were spaced at 600 mm centres, and cantilevered over 3 m from the soffit of the slab down to the head of the wall. This created a forest of steel components making it very difficult for services distribution.

The bond beam and column system was an ideal solution for stiffening and restraining these walls. The bond columns were able to span the full 6.5 m from floor to soffit level, and the beams and columns could be positioned to avoid the ductwork and cabling runs see Figure 5 and 6. Bond beams could be located at the head of partial height walls to provide restraint, and also at balustrade height along escape corridors to resist the unusually high loads in these areas.
As a very high profile publicly funded project, the Aquatic Centre was under much more scrutiny than normal construction projects, and many different regulatory bodies had an interest in approving and then monitoring the performance of the bond beam and column system. Through a combination of test results, visits to completed buildings and then excellent construction quality on site all of these parties agreed to the use of the system. The Olympic Delivery Authority contained a team of ‘Innovation Champions’ who were very supportive of the system.

As with all masonry applications, the length of blockwork panels is limited by the need for movement joints to manage shrinkage. It is certain that the use of bond beams will reduce the amount of shrinkage in a wall panel; however this is not yet documented by test results and could not be relied on. In designing the Aquatic Centre masonry the strength added by introducing bond beams meant that the walls were able to span significantly further than the recommended spacing of movement joints. In order to avoid this limitation it was necessary to introduce bed joint reinforcement to allow the movement joint spacing’s to be increased.
POTENTIAL FUTURE DEVELOPMENTS
The application of the system at the Aquatic Centre was a great demonstration of how well the system worked, however it revealed the shortcomings of the system with regard to blockwork shrinkage and movement joint spacing. Whilst the introduction of bed joint reinforcement was able to overcome this problem, further testing would enable the investigation of the effect of bond beams on reducing shrinkage, allowing the reduction or omission of bed joint reinforcement on future projects.

CONCLUSIONS
The subdivision of large blockwork walls subjected to high lateral loads can be achieved without the extensive use of wind posts. The combination of the design of the subpanels using Code Guidance and a ‘design by test’ approach for the bond beams and columns has enabled an ultimate limit state design approach to be developed, without the need for serviceability checks. Throughout the test programme progressive improvements were made to the components of the system.

The system has been used very successfully on the complex 2012 Aquatic Centre project. The masonry contractor has reported that the block walls were erected faster and cheaper than using comparable traditional systems. This project demonstrated the value of the system for applications with long or high walls with significant lateral loads. Further investigation would lead to improved guidance on provision for the effects of shrinkage.

REFERENCES


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