

COMPARATIVE TESTS ON AGGREGATE CONCRETE BLOCKWORK WALLS CONTAINING WIND POSTS AND BOND BEAMS

by
EDGELL, G.J.¹ AND CLEAR, L²

*CERAM Building Technology, Stoke on Trent, UK
Pyramid Builders, London, UK*

SUMMARY

The use of steel wind posts to subdivide large panels which are subjected to significant wind loads is common. However they do create issues of cost, correct installation, tying and fire protection. An alternative solution for aggregate blockwork is to introduce horizontal beams at intervals up the height of the wall. This paper describes a programme of tests on eight walls either subdivided by bond beams, by a box section wind post or an integral wind post. Sufficient has been done to demonstrate the satisfactory performance of the bond beam approach and a design procedure has been developed.

INTRODUCTION

When designing large masonry walls it is not uncommon to subdivide the walls into smaller panels by the use of wind posts. These are essentially steel columns fixed back to a beam or slab at head and foot and are regularly tied in to the bed joints of the masonry. In this way the extent of the spans over which the masonry is required to resist wind load are limited. Wind posts can be unpopular as they can be numerous are usually relatively expensive and their installation time consuming on site. Pyramid Builders, a major masonry contractor based in London developed the idea of using bond beams in aggregate concrete blockwork construction. In order to prove the concept a series of four walls were built and tested at CERAM.

The walls tested were substantial, ie 8.1m long and 5.1m high constructed in 140mm blockwork. The performance of the first four walls was encouraging although a cautious approach had been adopted by way of introduction of bed joint reinforcement and the frequency of anchoring to the steel frame used in the laboratory. Consequently a second phase of testing was undertaken on a further four walls. The details of these were far more typical and the opportunity was taken to build one 12m long. All of the walls were tested using a simulated wind load (air bag) The results of the second series were even more encouraging than the first. A design approach has been developed.

TEST PROGRAMME

Phase 1

Positive uniformly distributed loading was carried out on four no. walls of nominal dimensions 8.1m long x 5.1m high constructed from 140mm wide, 7N/mm² medium aggregate blocks and designation (iii) ready mixed retarded mortar. A general description of each wall is given below.

Walls 1 and 2 Bond Beam Walls – tied on 2 sides and top, with bed joint reinforcement in every course.

Walls 3 Wind post Integral 100 x 60 x 6.5mm – tied in on 2 sides and top, with bed joint reinforcement in every course and an additional 2m bed joint reinforcement straddling the wind post.

Wall 4 Standard Box Section Post 100 x 100 x 8mm – tied in on 2 sides and top, with bed joint reinforcement in every course for first 9 courses then every second course full height.

In addition, 2 sets of flexural strength wallettes were tested in accordance with EN 1052-2 as follows:

- Set 1 - parallel to the bed joint containing no reinforcement.
- Set 2 - perpendicular to the bed joint containing bed joint reinforcement every course.

Phase 2

In the second phase three walls similar to those in Phase 1 but with simpler details were built and tested. The fourth was 12m horizontal span.

Wall 5 Bond Beam Wall 8.1m. x 5.1m – tied in on 2 sides and top, with bed joint reinforcement on every course apart from 3 courses above and below the bond beam course.

Wall 6 Bond Beam Wall 12.0m x 5.1m – tied in on 2 sides and top, with bed joint reinforcement on every course apart from 3 courses above and below the bond beam course.

Wall 7(Wall 8.1m x 5.1m) - Wind post Integral 100 x 60 x 6mm – tied in on 2 sides and top, and at the wind post.

Wall 8 (Wall 8.1m x 5.1m) - Standard Box Section Post 100 x 100 x 8mm – tied in on 2 sides and top, and at the wind post.

WALL CONSTRUCTION

Walls Containing Bond Beams

Rigid steel uprights were erected in the Structures lab and bolted down to the laboratory strong floor. A steel channel (200 x 65mm) acting as a head restraint was bolted as a crosspiece to the uprights creating a frame of nominal dimensions 8.1m long x 5.1m high.

A first course of 140mm wide perforated clay bricks was laid off a polyethylene layer. Aquaguard dpc was placed on this and the blockwork constructed above. Each of the walls was tied into the steel uprights with 175mm Ancon frame ties at 450mm centres with a layer of 12mm x 140mm Corofil fitted between the blockwork and steel upright.

BRC 3.5mm bed joint reinforcement was placed in every course in Walls 1 and 2 and in each course except the three above and below the bond beams in Wall 6.

At the seventh course a bond beam was built into the wall incorporating a 7N/mm^2 140 x 214 x 440mm medium density hollow concrete block filled with concrete with 2 no. 16mm diameter rebars such that the first rebar was positioned with 47.5mm depth of infill concrete above it and the second rebar was positioned with 111mm depth of concrete infill above it and 47.5mm cover beneath it. The rebar ran the full length of the wall and at each end was inserted into the pocket of a cleat allowing 85mm penetration of the rebar. The cleat was welded to the steel uprights.

Transfer rods were cast into the bond beams and built into the cross joints of the course above.

The bond beam construction was repeated at course 15.

At the junction of the soffit and the blockwork a 20mm deflection joint filled with Corofil was included. An HRV head restraint was fitted to the steel channel which was acting as a soffit at 900 centres.

In Walls 5 and 6 300mm wide x 100mm deep and 8mm thick galvanised steel angles were welded to the channel/soffit at 900mm centres staggered front and rear.

Wind Post Wall – Integral Post

The details for these walls were similar to those containing the bond beams other than the 100mm x 60mm x 6.5m wind post was incorporated within the thickness of the wall at mid span. In Wall 3, 175mm ties at 225mm centres and an extra 2m of bed joint reinforcement was placed in every course and straddling the wind post. In Wall 7 the spacing of the ties at the wind post was expanded to 450mm and the bed joint reinforcement was omitted.

Wind Post Wall – Standard Wind Post

In Wall 4 bed joint was used in each of the first nine courses and then every second course to be full height. In Wall 8 this was excluded. In Wall 4 the wind post was tied to the blockwork every 225mm vertically, in Wall 8 this spacing was extended to 450mm.

METHOD OF TEST

Flexural Strength of Masonry

The wallettes were tested in accordance with BS EN1052-2 : 1999 Determination of flexural strength of masonry.

Positive Uniformly Distributed Loading

A series of airbags were positioned on the face of the wall and a reaction board was placed over this butting up to and tied to the steel frame. A series of steel uprights were bolted to the laboratory strong floor behind the reaction boards and props were used to brace the boards back to the uprights. Plate 1 shows a general view of the loading arrangement.



Figure 1 : General View of Loading Arrangement

A series of linear displacement transducers were located on an independent scaffold frame reading onto the rear of the wall at the positions shown in Figure 1.

A uniformly distributed load was applied to the walls via the air bags in 0.2kN/m^2 increments to failure.

RESULTS

The failure loads on the walls in Phase 1 are given in Table 1, those from walls in Phase 2 in Table 2.

Table 1 : Summary of Failure Loads of Walls Under Positive Udl : Phase 1

Wall Number	Description	Load at First Crack (kPa)	Failure Load (kPa)
1	Bond Beam	5.0	5.0
2	Bond Beam	5.8	6.0
3	Integral Wind Post	3.6	5.2
4	Box Section Post	5.6	5.6

Table 2: Summary of Failure Loads of Walls Under Positive Udl : Phase 2

Wall Number	Description	Load at First Crack (kPa)	Failure Load (kPa)
5	Bond Beam 8m long	6.00	6.50
6	Bond Beam 12m long	4.20	4.88
7	Integral Wind Post	6.00	6.50
8	Box Section Post	7.50	7.50

The results from the wallette tests are given in Table 3.

**Table 3 : Results from Flexural Strength Wallettes
(parallel and perpendicular to bed joint)**

Sample Number	Flexural Strength N/mm ²	
	Parallel	Perpendicular
1	0.17	0.79
2	0.18	0.71
3	0.21	0.86
4	0.20	0.81
5	0.19	0.81
Mean	0.19	0.80

DISCUSSION

The interesting feature of the failures in Phase 1 were that they all showed some indication that the restraints at the head of the wall were fairly ineffective and this was borne out by the deflected profiles. Also the walls seem to have as a whole entity rather than being really subdivided by either the bond beams or the wind posts. There were “back of the envelope” crack patterns although these were not as fully developed as in most single leaf walls without beams or posts. See Figures 1-4.

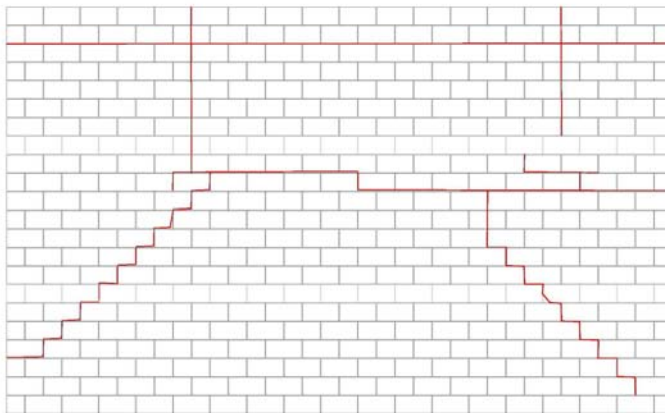


Figure 2 : Failure Pattern – Wall 1

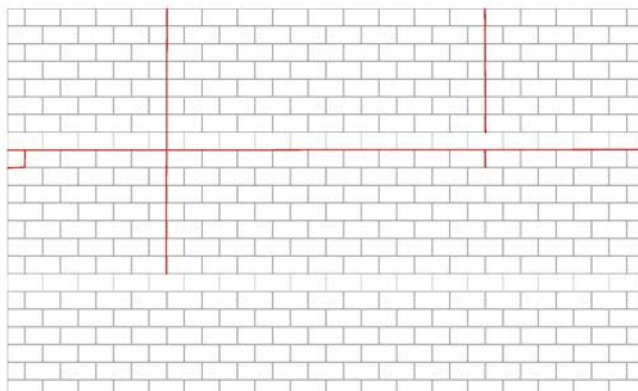


Figure 3 : Failure Pattern – Wall 2

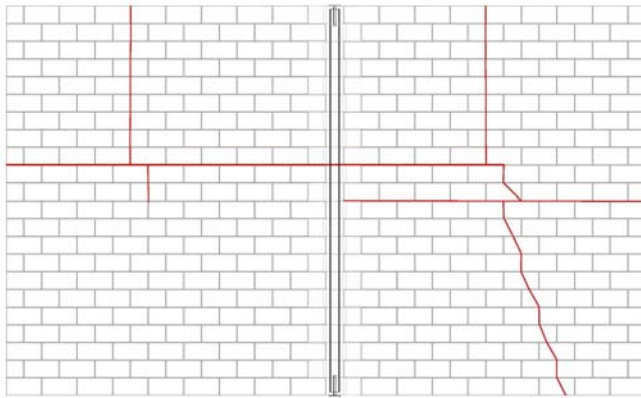


Figure 4 : Failure Pattern – Wall 3

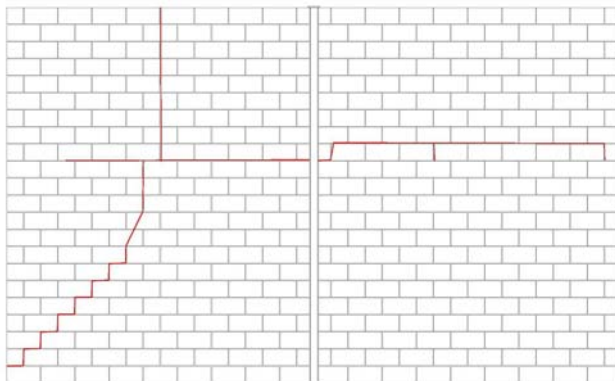


Figure 5 : Failure Pattern – Wall 4

Overall the picture seems to be that the failure and deflected profiles indicate that walls of the configurations tested are very strong and behave like two way spanning walls with, at failure, little head restraint. The wall with the integral wind post cracked before failure, the others either did not or did close to failure. Nevertheless the lateral loads involved to cause cracking and failure were considerably above design values. Deflections at working loads were low.

In Phase 2 the test results for the 8m span walls were slightly improved above those in Phase 1 despite a reduced and more realistic amount of tied connections and bed joint reinforcement. The heavier duty head restraints did little to improve behaviour and it was particularly noticeable that in the 12m wall rotation at the head of the wall led to a horizontal crack beneath the top course which rendered the restraint ineffective. The crack patterns again showed that there were elements of two way spanning and in the walls with wind posts although the cracking occurred preferentially in one half of the wall it did suggest two way spanning in the whole of the wall. The crack patterns are shown in Figures 5-8.

The location of the deflection transducers is shown in Figure 10. A typical deflection profile up the height of the wall is shown in Figure 11 (Wall 2, Phase I).

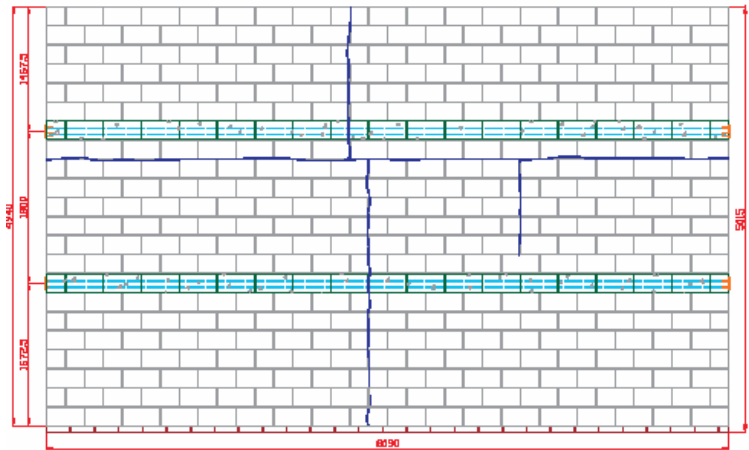


Figure 6 : Failure Pattern – Wall 5

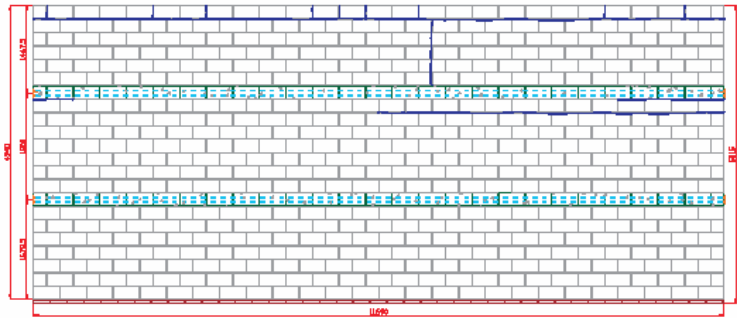


Figure 7 : Failure Pattern – Wall 6

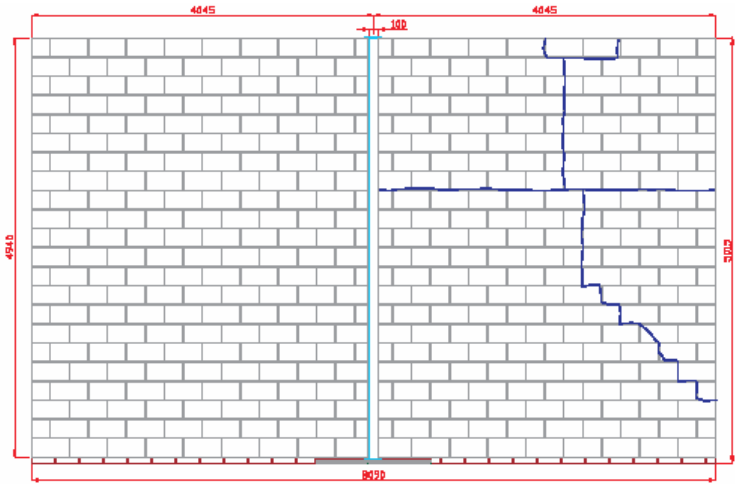


Figure 8 : Failure Pattern – Wall 7

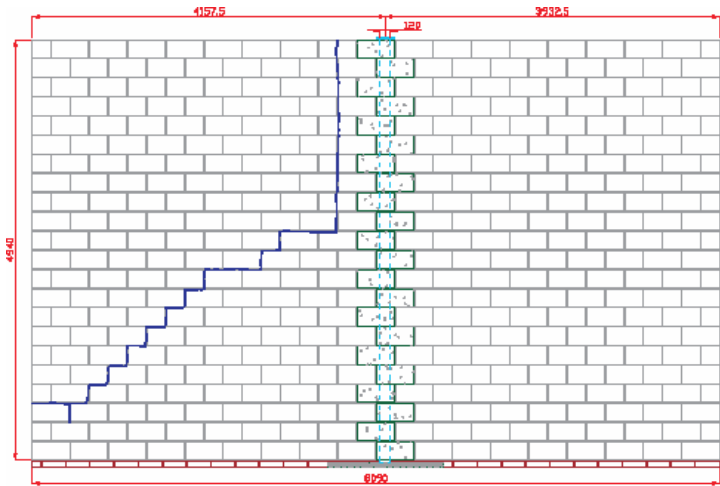


Figure 9 : Failure Pattern – Wall 8

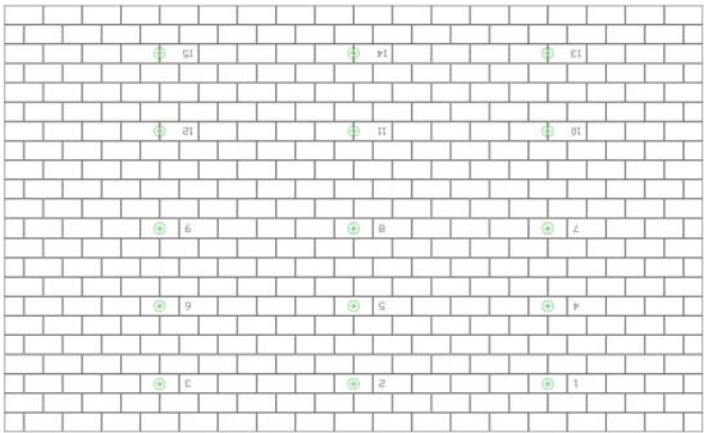


Figure 10 : Transducer positions for all tests

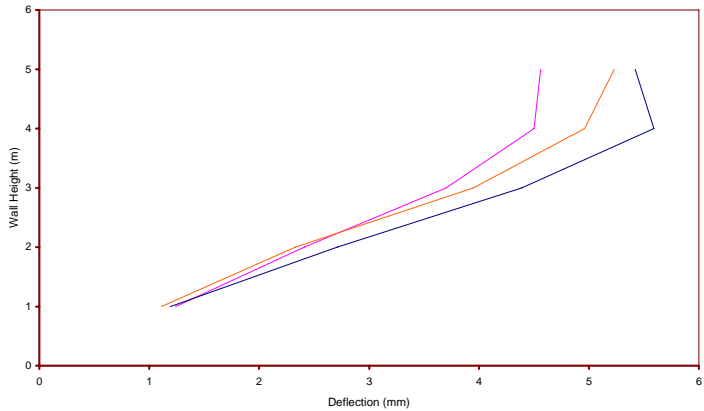


Figure 11 : Wall 2 Bond Beam
Vertical Deflection Profile under 3kPa Uniformly distributed load

DESIGN PROCEDURE

A design procedure has been developed for the walls with bond beams which is realistic in that the design moments are less than design actions and the failure loads predicted using the approach are very conservative. The basic steps in the procedure are as follows:

1. Subdivide the wall into imaginary sub panels, the subdividers being the bond beams.
2. Design each panel as vertically spanning blockwork using the characteristic flexural strength values from BS 5628 Part 1.
3. Consider there is moment restraint at the top of a bond beam if there are vertical steel connectors from bond beam concreted to course above.
4. Treat the bottom of a bond beam as a simple support.
5. Decide whether head restraints can provide moment restraint. If it is not clear assume simple support.
6. The γ_m and γ_f values are 3.0 and 1.2 respectively.
7. The lowest panel capacity determines the characteristic wind load which can be resisted.
8. Check the bending capacity of the bond beams treating them as locally reinforced blockwork, ie beam width is three times wall thickness.

The procedure has been applied to the test walls. In this case the mean measured flexural strength (Table 3) has been used instead of the code values. The results of applying the design procedure to Wall 5, ie one without bed joint reinforcement and assuming simple restraint at the head of the wall was limited by the vertically spanning capacity of the upper panel of blockwork. The predicted strength was 3kN/m^2 compared with an actual strength of 6.5N/mm^2 . Whilst this is very conservative it is not unusual as this is often the case in walls reinforced with bed joint reinforcement to resist wind load. In the case of Wall 8 the predicted strength of the wall was the same based upon the same limiting panel (setting gamma factors to unity). However the bond beam was considered to be relieved of load over 10% of its length at each end, it being assumed that this load was being transmitted directly to the vertical supports by the ties at the sides. This was considered to be reasonable as there would clearly be an element of two way spanning close to the supports, the critical area for the vertical spanning of the masonry being remote from the supports. The failure load in this case was 4.88kN/m^2 compared to the prediction of 3kN/m^2 .

CONCLUSIONS

The form of construction using bond beams instead of wind posts to subdivide spans in order to resist wind loads in large panels is practical. The failure loads of the walls are very high and comparable to those of the panels containing wind posts. A demonstration of a wall with 12m span was completely successful. All of the walls demonstrate an element of two way spanning and although those containing the wind posts tended to develop cracking preferentially in one half the pattern indicated that the panel had not been subdivided into two panels which behaved independently. The failure loads were very high in relation to realistic characteristic wind loads. A conservative design procedure has been proposed.

REFERENCES

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NOTE:References 3 and 4 contain much more information about the experimental details, materials, performance and can be made available by CERAM Building Technology.