DESIGN GUIDE FOR WALLS CONTAINING BOND BEAMS

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ABSTRACT
In 2006 CERAM began an experimental programme to investigate the performance of large blockwork walls, reinforced at intervals up their height by bond beams[1]. The concept was developed by Wembley Innovation as a simple alternative to the use of wind posts. The performance of the walls was very encouraging and lateral loads in the region of 6kN/m^2 were satisfactorily resisted. Since the initial tests various configurations of the walls, for example wall height to length ratios have been varied, the introduction of windows and door openings have been investigated and the connections to the framing elements of the building have been refined. In order to introduce the system to the mainstream of structural engineering consultants in the UK and elsewhere CERAM produced a design procedure which has been developed in conjunction with consultants Jenkins and Potter and Buro Happold. The procedure essentially builds upon the approach in the UK for the design of walls containing prefabricated bed joint reinforcement and incorporated in BS 5628: Part 1. As current UK Codes of Practice[2] are due to be withdrawn in 2010 and the inclusion of new material in the Eurocode (EN 1996-1-1, Eurocode 6)[3] is not yet possible CERAM as an independent body has published a Design Guide[4] for the system. This paper introduces the guide, explains the provisions and shares the supporting test evidence. The system has been used on a number of major schemes and some feedback on the experience so far is given.

KEYWORDS: Aggregate concrete blockwork, lateral load design, bond beams.

INTRODUCTION
This Design Guide has been developed from an extensive series of tests on full size walls generally 8m x 5m (length x height) and reinforced at intervals up their height. It was felt that although walls of this size were fairly large they were typical of non loadbearing walls that might be used for example in large shopping developments or sports facilities. The test walls were made from concrete blockwork and in the case of plain walls reinforced by bond beams at approximately one third and two thirds of the height. The bond beam course was a trough type concrete masonry unit containing two 16mm bars one above the other, the bars fitted into metal cleats fixed to columns at their ends and were concreted into the trough. At intervals vertical

shear transfer rods connected the bond beam to the course above and below it. See Figures 1 and 2.

Figure 1: Bond beam showing concreted section and shear transfer rods

Figure 2: Cleat Welded to Steel Column

For walls with window or door openings the bond beams were at window head and sill level or at door head height respectively. Although the testing programme was carried out using cleats and transfer rods of proprietry design the remainder of the components are readily available and comply with relevant standards and certifications. The Guide has therefore been produced in a relatively general way.

WALL CONSTRUCTION AND TESTING

Rigid steel uprights were 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. A damp proof course was placed on this and the blockwork constructed above. Each of the walls was tied into the steel uprights with 175mm frame ties at 450mm centres and a layer of 12mm x 140mm movement joint filler material was fitted between the blockwork and steel upright.

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 reinforcing bars such that the first was positioned with 47.5mm depth of infill concrete above it and the second with 111mm depth of concrete infill above it and 47.5mm cover beneath it. The bar ran the full length of the wall and at each end was inserted into the cleat to a depth of 85mm. 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 soffit a 20mm movement joint was included. Head restraints were fitted to the steel channel which was acting as a soffit at 900mm centres.

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. Deflections were measured from an independent framework at generally nine positions using linear voltage displacement transducers, in certain tests strains were measured using resistance strain gauges on the reinforcing bars and on the cleats. Figure 3 shows a general view of the loading arrangement.

TEST PROGRAMME

The test programme has evolved over a period of some two years and has consisted of the following phases. The results which are considered here are those which are relevant to the development of the design procedure.

Phase 1: Four walls each 8.1m×5.1m, two containing bond beams and two containing wind posts, one a standard section and one integral to the wall.

Phase 2: Four walls similar to those in Phase 1 except with slightly different detailing and one of the bond beam walls being of wider span, 11.7m×5.1m.

Phase 3: Two walls each 8.1m×5.1m one containing a door opening and one a window opening.

Phase 4: Control wall i.e. no reinforcement and two with single bond beams.

Phase 5: One wall to repeat the 8.1m×5.1m wall on phase 2 and two 8.1m×1.1m walls to focus on spanning capacity of bond beam alone.
Phase 6: Two walls 8.1m×5.1m with improved shear connector design.
In addition various beam tests, wallette tests, shear tests and an impact test have been carried out.

Figure 3: General Arrangement for Lateral Loading using Air Bags

DESIGN GUIDE PROVISIONS: SCOPE AND PROVISIONS

The scope of the guide restricts the guidance to the structural design of 140mm thick single leaf concrete masonry walls reinforced at intervals up the height with bond beams. Whenever possible the guide refers to the provisions of BS 5628, the materials specifications are by reference to established harmonised European Standards, where possible, and to British Standards. The materials referred to are those which were used in the extensive experimental programme, for example aggregate concrete masonry units of minimum compressive strength $7N/mm^2$ and a 1:1:6, cement:lime:sand mortar. The bond beam courses incorporated 2 no. T16 bars and were covered by a C40 pre-mixed (bagged) concrete infill.

DESIGN RECOMMENDATIONS: PRINCIPLE

The design of concrete blockwork walls to resist lateral loads follows the guidance given in BS 5628 Part 1 and Part 2. In the case of walls containing bond beams the principle is to divide the walls into sub-panels using the bond beams and any vertical supports e.g. wind posts, local vertical reinforcement. Each sub-panel is then designed according to BS 5628-1 using the relevant flexural strengths, support conditions and height/length ratio.
An overall check on the wall strength is made.

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 contiuous support can be assumed.

If there is sufficient precompression 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 at the damp proof course if it has been proven by tests (see DD86-1)[5]. 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.

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

LIMITING DIMENSIONS

The dimensions of the sub-panels are limited in accordance with BS 5628 Part 1, which are given in terms of the effective thickness of the wall. For a single leaf wall of 140mm thickness the limiting dimensions of the sub panels are, for example:-

Panel Supported on three edges;

1) two or more sides continuous = height x length equal to 29.5m^2 or less
2) all other cases = height x length equal to 26.5m^2 or less

CHARACTERISTIC FLEXURAL STRENGTH OF CONCRETE MASONRY

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

Alternatively the value may be determined from Table 1 below which is derived from Table 3 of BS 5628-1.

Table 1: Characteristic flexural strength of masonry f_{kx} N/mm^2

Mortar Strength Class/Designation	**Plane of failure parallel to bedjoints**	**Plane of failure normal to bedjoints**
	M4/(iii)	
Aggregate concrete masonry units 140mm thick and with compressive strength of 7N/mm^2	0.22	0.52

Note: Test to determine the compressive strength of concrete blocks should be in accordance with BSEN 772-1

PARTIAL FACTORS
It is assumed that the recommendations in clause 11 of BS 5628-2 for the special category of construction control for the reinforced elements will be observed and that this control extends to the unreinforced sub-panels. In these circumstances it is recommended that the value for flexure in BS 5628-1 for the special category of construction control i.e. 2.5 may be used. If this level of control cannot be achieved than a value of 3.0 should be used.

DESIGN OF SUB PANELS
The design procedure for the sub-panels should follow the provisions of **32.4** of BS 5628-1. In particular it should be noted that where a subpanel 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 1.75 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 strength may be modified to $f_{kx} + \gamma_m g_d$ where g_d is the design stress due to vertical load (including self weight) normal to the bed joint. The inclusion of the partial factor in the characteristic flexural strength ensures that the design strength is increased by g_d and is not reduced by subsequent division by the partial factor later in the design process
If bed joint reinforcement is used, for example, to limit cracking if is generally ignored for the structural design of the panels.

In the case of walls without openings the subpanels will often be long and shallow and will be considered as vertically spanning. The test programme has demonstrated that the recommended design of shear transfer rods ensures that the bond beam acts compositely with the courses above and below it. Consequently the subpanels are designed as spanning from the top and bottom of those stiffened courses, usually assuming a simple support at the point.

DESIGN OF WHOLE WALL PANELS
As well as ensuring that each of the subpanels can be justified it is essential that the wall as a whole can be. Initially the approach adopted was to treat the bond beam as a simply supported beam resisting loads from the subpanels to which it was bonded. The beam was justified in bending using the provisions of BS5628 Part 2. Although this approach was satisfactory when based upon the steel yielding the design restriction normally applied to ensure that compression failure of the blockwork did not occur proved to be very restrictive. In practice compression failures do not occur and indeed a series of tests on 18 three course high beams tested in four point bending demonstrated that failures were relatively ductile and only at very large deflections did some compression failure of the face shell of the block occur. Although the restriction in the Code design procedure is to guard against a sudden brittle compression failure which did not occur in the test programme it was felt that simply removing the check would be considered to be unacceptable. Consequently an alternative approach was considered and this is now described.

The walls containing bond beams all failed at considerable lateral loads, generally in excess of 5/kN/mm^2. A similar wall without the bond beams resisted on ultimate load of 1.78kN/m^2. The load at which cracks first appeared in the case of the reinforced walls were at least 3.6kN/m^2. In most cases the crack patterns indicated that at failure there was some element of two way spanning, see Figure 4.

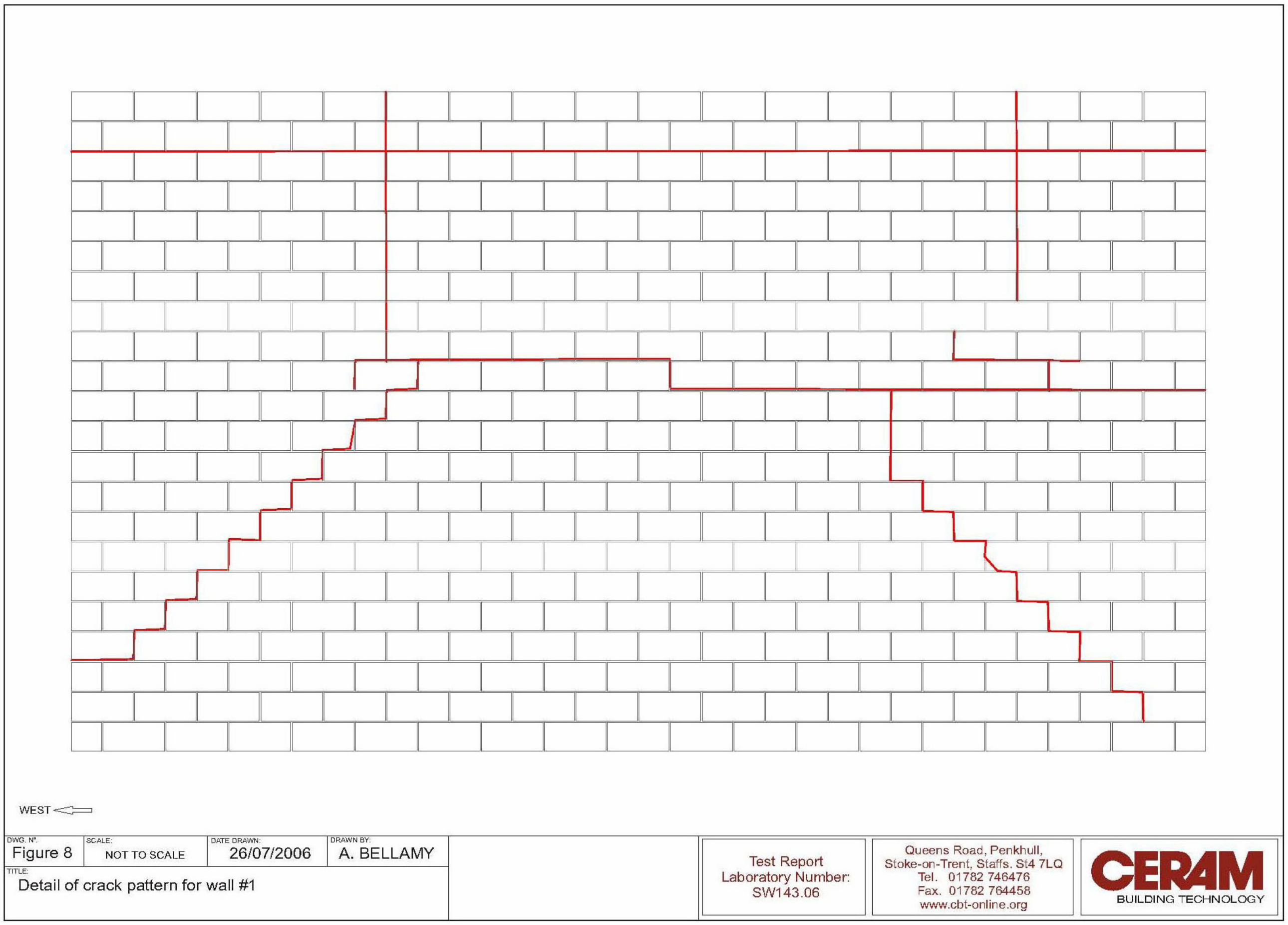

Figure 4: Crack Pattern at Failure of Wall Containing Two Bond Beams

Consequently it seemed reasonable to adopt one of the approaches used in BS5628 Part 2 to justify the design of walls containing bed joint reinforcement. The approach is based upon the observation that the load which caused cracking in the walls containing bed joint reinforcement is at least as large as that to cause failure in unreinforced walls. Consequently the failure load for the unreinforced wall is taken as the basis for the serviceability limit for the reinforced wall. The ultimate that can be resisted when not containing any reinforcement is calculated in accordance with clause 32.4 of BS 5628 Part 1 taking γ_m and γ_f as unity. This is then considered to be the load to cause first cracking in the wall containing bond beams. The ultimate load for the wall is then taken to be 1.5 (the partial factor for serviceability) times this calculated load. The vertical load in the panel may be used to enhance the characteristic flexural strength as for the subpanels but in this case as the calculation is for at failure, the characteristic flexural strength normal to the bed joint is taken as $f_{kx} + g_d$ where g_d is the design stress due to vertical load, in many cases just that due to the self weight.

This approach is inevitably conservative as the true cracking load of the walls containing bond beams exceeds the failure load of the unreinforced wall by a significant margin. In any further refinement for walls of this type it must be recognised that the actual difference between cracking load and failure load is generally quite small and does not reflect a partial factor of 1.5.

COMPARISON WITH TEST RESULTS

The results of the walls tested to date are given in table 2 together with the estimated failure loads. In phases 1-3, 4 wallettes were tested accompanying the walls and these results have been used in the calculations. The first two phases gave very similar wallette strengths and wallettes were not repeated for the later phases. In the case of all of the phases partial fixity has been assumed at the side of the walls, the top is assumed to be a simple support as in practice there was minimal rotational restraint and the damp proof course was assumed to provide simple support. The calculated failure loads are those based upon the whole wall design described earlier.

Table 2: Failure Loads

Phase	1st crack kN/m^2	Failure kN/m^2	Predicted (kN/m^2)
1	5.0	5.0	4.5
	5.8	6.0	4.5
2	6.0	6.5	4.5
	4.2	4.88	4.5
4 (Control wall – no beams)	1.6	1.78	3.0(2.1)
5	3.02	3.48	4.5 (3.2)
6	4.5	5.5	4.5(3.2)
	4.75	6.5	4.5(3.2)

Overall the estimates are fairly reasonable although they can be greatly affected by the assumptions about side restraint. It should also be noted that in Phase 5 a relatively low result was achieved in the test. It was observed during the test that the character of the deflections was unusual, being quite large in the upper areas of the wall. Subsequent investigation showed that the head restraints were relatively loosely connected and hence this result, although included for completeness, should be discounted when considering the overall analysis. For phases 4-6 the figures in parenthesis give the calculated failure strengths based upon Code guidance figures as opposed to actual measured wallette strengths. The reinforced walls would be designed to resist a characteristic wind pressure of $2.5kN/m^2$ allowing for a partial factor for materials of 1.5 (serviceability) and 1.2 as the partial factor for wind load. However the subpanel design is the controlling factor and it is the vertical spanning of the top subpanel which is critical and the characteristic wind load is calculated to be $1.45kN/m^2$. At this load deflections were typically 1.5mm which is span/5400, i.e. minimal.

In the phase 3 of the work panels with window and door openings were introduced. Although the subpanels are again the factor restricting the design the failure pressure may be estimated and are 3.1kN/m^2 (window opening) and 3.85kN/m^2 (door opening) which compare to the actual failure loads of 3.3kN/m^2 and 4.2kN/m^2.

CONCLUSIONS

After an extensive testing programme a design procedure has been developed for aggregate concrete blockwork walls containing bond beams. The approach is practical and leads to estimates of the lateral strength of walls that are reasonable. In practical design it is the subpanels that are critical in that they limit the characteristic wind load that can be resisted although in test the failure of a subpanel does not lead to complete failure of the wall.

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