



Effect of slenderness and eccentricity on the strength of concrete block masonry: An experimental investigation

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Abstract

There are a number of factors which govern the strength of masonry, namely strength and elasticity of masonry units and mortar, slenderness ratio of the wall, nature and magnitude of eccentric loads and masonry efficiency. There have been a number of publications on these factors, although many are with respect to brick masonry or masonry used in western countries. This is perhaps the first study in Indian Literature where a detailed study of hollow concrete block walls has been made. Experimental investigations on the masonry walls with two types of hollow blocks for the influences of slenderness ratio and eccentrically loaded walls have been studied in this paper. It is also necessary to observe that full-scale wall strengths have not been studied adequately in the Indian Context. The entire testing program was concentrated on the cellular concrete block and hollow concrete block masonry. The full-scale walls of two different sizes, i.e., 150-mm blocks and 200-mm blocks, were tested for hollow concrete blocks, whereas only 200-mm-thick blocks were tested for cellular concrete blocks. A total of nine walls were tested, out of which five walls were subjected to the axial type of loading, and the remaining four walls were subjected to eccentric loading. The test results confirm the better performance of hollow concrete block masonry as compared to brick masonry wall specimens tested by previous researchers.

Keywords Slenderness ratio · Eccentricity · Stress reduction factors · Cellular concrete blocks · Hollow concrete blocks · Performance

1 Introduction

Walls are defined as structural members whose dimensions along the width are more than four times their thickness [1]. Based on the structural configurations and functions, walls may be classified as load-bearing walls and non-load-bearing walls. In 1968, 50% of clay blocks were practically replaced by concrete blocks. By 1980, hollow concrete block construction occupied almost half of the housing construction in the USA [2]. Rao et al. [3] in his research noted that adoption of hollow concrete block masonry could result in savings of the order 5 to 15% as compared to brick work. During the same period, hollow

concrete block construction found favor in China and European countries. Switzerland and Denmark, who did not have a steel industry of their own, preferred to build 15- to 16-story buildings using high-strength bricks, which were locally available [4].

IS: 2185-Part-I (2005) [5] also grades the concrete blocks in the same manner as load-bearing and non-load-bearing walls. The strength and behavior of load-bearing masonry may be understood by testing smaller test specimens, namely prisms. The other option is by testing short walls (known as wallettes) or walls of story height dimension (full-scale walls) depending upon the requirement. It is a general practice to test the smaller specimens, viz. individual units,

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mortar and masonry prisms. The results are then correlated to arrive at the strength and behavior of full-scale specimens. However, most of the researchers [6–8] have observed that factors affecting the failure of units or prisms are not the same as those of walls, as the latter is influenced by the effect of slenderness ratio of wall and eccentricity of loading. Hence, this paper deals with the study of the properties of full-scale concrete block masonry walls constructed using two different types of concrete blocks, namely cellular concrete blocks (CCBs) and hollow concrete blocks (HCBs) using cement–sand mortars of proportion 1:6

2 Factors affecting the strength of full-scale masonry walls

There are several factors which influence the strength of a masonry wall. The primary factors are masonry unit characteristics, mortar strength characteristics, slenderness ratio of the wall (ratio of the effective height of wall divided by the effective thickness or effective length divided by the effective thickness, whichever is less), the eccentricity of loading and boundary conditions at the top and bottom of the wall. Apart from these mentioned factors, another significant factor is the effect of workmanship which can influence the strength of masonry [7, 9].

During the construction and testing of full-scale wall panels, all these factors mentioned above are varied, resulting in a more realistic understanding of the design of load-bearing masonry structures. Normally, the load-carrying capacity of masonry can be assessed in three ways, namely (1) tests on masonry constituents, (2) tests on masonry prisms and (3) tests on full-scale wall specimens. If the full-scale walls are tested, then a more realistic understanding of the performance of masonry may be possible as it takes into account all the above-mentioned factors and in addition considers the effect of numerable joints (both vertical and horizontal) within the masonry. In the Indian context, the need for full-scale tests and special testing facility required for the same was discussed extensively by Ambalavanan et al. [8]. As a general recommendation in the modeling of any structure or element, it is recommended to make a full-scale model of masonry which can be practically tested (Ambalavanan [8] and BS 5628 Part 1-1978 [10]).

3 Literature review/earlier investigations of full-scale walls

The studies carried out on full-scale masonry walls, and salient observations made by earlier researchers and code provisions have been highlighted and presented in the following sections.

3.1 Standards/specifications as per various codes

As per the Indian Standards IS: 1905-1987 [1], structural design of load-bearing masonry wall is carried out by working stress approach. The fundamental requirement is to check the stress developed at the base of the wall and compare it with the permissible stress values. The code as reported in Table 8 of IS: 1905-1987 [1] gives values of the basic compressive stress of masonry which confirms to brick masonry unit dimensions. However, for the block masonry, alternatively, based on the experimental results on masonry prisms (height-to-width ratio between 2 and 5), one may also arrive at the basic compressive stress. The basic compressive stress recommended by the code [1] is taken as 0.25 times the prism strength with a correction factor for various h/t ratios of prisms. The code [1] specifies that permissible stress arrives through basic compressive stress computed earlier and applying correction factors, viz. shape modification factor, area reduction factor and stress reduction factor. The shape modification factor takes into account the shape of the unit, i.e., height-to-width ratio (as laid), the area reduction factor takes into consideration the smallness of the sectional area of the element and is applicable when sectional area is less than 0.2 m², and the stress reduction factor, which is an important factor, takes into account the slenderness ratio of the element and eccentricity of loading. Table 9 of IS: 1905-1987 [1] indicates the stress reduction factor for a maximum slenderness ratio of 27, and it may be noted that with an increase in the value of slenderness and eccentricity ratio, there is a decrease in the stress reduction factor.

Unlike the Indian standards [1], the British code BS-5628 [10], the Eurocode EN-1996-1-1 [11], the Canadian code CSA 304.1 [12] and the Australian code AS 3700 [13], the structural design of masonry is based on the limit state approach. The stress reduction factor is represented by the term capacity reduction factor in these codes. The design strength in the British standards is determined by the characteristic strength multiplied by a capacity reduction factor and divided by the partial safety factor of the material. However, the British code is now obsolete and is superseded by the Eurocode.

3.2 Earlier research on masonry prisms and full-scale walls

A literature review on studies done on full-scale masonry has been reported in this section. Several investigations have been carried out on full-scale masonry wall panels especially in the UK with a specially manufactured

machines of 1000 ton capacity [8] (for example, Amsler machine at Building research station, UK), assembled machine of 1000 ton capacity at Edinburgh University and 750 ton capacity testing machine at Cement and Concrete Association of Wrexham University [14]. The size of the wall panels recommended for testing is around 1.2 m to 1.8 m wide and 2.4 m to 2.7 m high which require special facilities for testing [8].

Prasan et al. [15] have reported results of investigations carried out on seventeen numbers of full-scale (story height which is approximately 3 m in height) brick masonry walls of 4 1/2" thick (105 mm thick). The effect of factors such as brick strength, mortar strength, slenderness ratio and eccentricity of loading has been reported. The authors indicated that the masonry strength is weakly related to mortar strength. The strength comparison made by the authors [15] showed in a few cases the eccentrically loaded walls show almost the same or higher strengths than the axially loaded walls.

Sinha [16] has reported the test results on a one-sixth-scale model brick walls, equivalent to 230-mm-thick English-bonded full-scale walls. A major argument put forward by his research is that comparing the masonry strengths from the factor brick work strength/brick compressive strength may not be realistic. On the other hand, it is proposed to have a comparison of masonry strengths based on factor brick work strength to tensile strength of the bricks. It is shown that the initial tangent modulus of elasticity of brick work in compression varies linearly with the tensile strength of brick.

In the Indian context, however, there have been several studies made at Indian Institute of Science (IISc) (Bangalore) and BMS College of Engineering (Bangalore) on testing and understanding the behavior of full-scale wall panels. In addition to this, the building and construction management division of IIT Madras was involved in series of research work carried out on hollow concrete block masonry [17]. Ambalavanan et al. [8] have presented in their paper the need for the full-scale tests on wall panels and illustrated the design, fabrication and installation of a special purpose wall testing facility. During the same period, Ramamurthy and Ganesan investigated the behavior of hollow concrete block masonry under axial compression [17]. The authors have also reviewed finite element analysis techniques for load-bearing masonry [18]. Further, Ramamurthy reported the behavior of grouted hollow concrete block masonry prisms [19].

A similar study was carried out by Khalaf et al. [20] on strength of concrete block masonry prisms. The strength of hollow concrete block prisms with and without grout was investigated. The hollow concrete blocks had gross area compressive strength of 11.3 MPa, while the corresponding prisms had the strength of 10.0 MPa, indicating an

efficiency of 89% of concrete block masonry. The grouted blocks had an average strength of 22.3 MPa, while the grouted prisms had an average strength of 15.0 MPa. The efficiency is apparently reduced to 67% when the blocks were grouted.

Gumaste [21] had conducted compression tests on three full-scale walls. Table molded bricks were used in two walls of which one was 105-mm-thick stretcher bond wall and the other being 230-mm-thick English-bonded wall. The third wall was again 105 mm thick, constructed using wire cut bricks. The ultimate strengths were compared with the respective wallette strengths to arrive at the actual reduction factor for understanding slenderness effect. The reduction factors were compared with that arrived from the code specifications [1]. The results are tabulated in Table 1.

Mangala Keshava [22, 23] conducted full-scale wall testing on fourteen walls, out of which eight walls were tested with concentric loading and the remaining six walls were subjected to eccentric loading. The full-scale walls were made up of table molded bricks (11), solid concrete blocks (1) and hollow concrete blocks (2). Further tests were carried out on walls with different slenderness ratios and eccentricity ratios. The results are reported in Table 1.

Recently, in addition to the above-mentioned research, there have been several studies made on the effect of full mortar bedding and face shell bedding and effect of thickness of mortar joints as reported by Thamboo et al. [24]. Fortes et al. [25] reported a relationship that was established statistically between compressive strength of the masonry and masonry units with the comprehensive test results by other researchers. It was reported for ungrouted masonry, the compressive strength ratios were approximately 0.8 and 0.5 for low- and high-strength units, respectively. Further, the same researchers [26] in order to study the compressive strength of high-strength concrete masonry grouted prisms devised an experimental program. They concluded that the efficiency of grouted masonry depends on many factors such as shrinkage characteristics of grout, bond between block and grout in addition to other factors mentioned earlier and recommended a more comprehensive study to understand the behavior of high-strength masonry.

4 Experimental investigation

As discussed earlier, the testing of full-scale masonry walls requires special arrangements and facilities. The testing of these walls is also very complex in nature. It requires a high degree of caution and cares for both men and equipment like strain gauges, demec gauges, etc., as the failure of masonry specimens are unpredictable. The details of

Table 1 Summary of tests on full-scale masonry walls from previous researchers

Sl. no.	Researcher	Materials	Unit strength (MPa)	Prism strength (MPa)	Dimensions of the wall $l \times t \times h$ (mm)	Slenderness ratio	Eccentricity (e)	Wall strength (MPa)
1	Gumaste [21]	Table molded brick	5.70	1.83	720 × 105 × 2770	22.42	0.0	1.08
			5.70	1.38	970 × 230 × 2770	10.24	0.0	1.32
2	Mangala Keshava [22, 23]	Wire cut bricks	23	7.40	750 × 115 × 2770	20.47	0.0	6.64
			5.85	1.73	950 × 225 × 1550	5.8	0.0	1.64
		Table molded brick	5.85	1.73	950 × 225 × 1550	5.8	0.0	1.72
			5.85	1.73	1050 × 230 × 2430	15.85	0.0	1.62
			5.85	1.73	950 × 220 × 2500	9.66	0.0	1.28
			5.85	1.78	1060 × 105 × 2440	19.75	0.0	1.29
			4.57	4.32	1030 × 150 × 2560	14.51	0.0	3.06
			6.08	3.83	830 × 150 × 2600	14.73	0.0	4.22
			6.08	3.83	830 × 150 × 2600	14.73	0.0	3.61
			5.85	1.73	950 × 225 × 1550	6.0	0.25 t	1.06
			5.85	1.73	950 × 225 × 1550	6.0	0.25 t	1.16
			5.85	1.73	1050 × 230 × 2430	15.85	0.67 t	0.64
5.85	1.73	950 × 220 × 2500	9.66	0.25 t	0.91			
5.85	1.73	950 × 220 × 2500	9.66	0.25 t	0.97			
			5.85	1.73	950 × 220 × 2500	9.66	0.25 t	1.06

the reaction loading frame, loading arrangements and lateral restraints are discussed in detail in the subsequent sections.

4.1 Details of the loading frame

Full-scale masonry walls were tested on a reaction loading frame of 2000 kN capacity which was designed, procured and erected in the Department of Civil Engineering, R.V. College of Engineering. The frame is a part of the financial aid under the Research Grants scheme program, funded by Visvesvaraya Technological University, Belgaum, Karnataka. The loading frame was designed as a reaction-type loading frame. It has a clear height of 3.75 m and a clear width of 2 m. The full-scale masonry walls were constructed and tested in the frame. The loading frame used for the test program involving full-scale/story height walls of approximately 3 m height is as shown in Fig. 1.

4.2 Details of the loading arrangement

To ensure a uniform loading on masonry specimen, a staircase system of loading is adopted on wall specimen. The loading arrangement comprised two-tier systems: an upper tier loading beam and a lower tier loading beam both made of steel sections. Steel rollers, MS plates of 25 mm thickness and a rich mix of the concrete bed are used to distribute the load. The entire assembly of loading



Fig. 1 Details of the loading frame arrangement

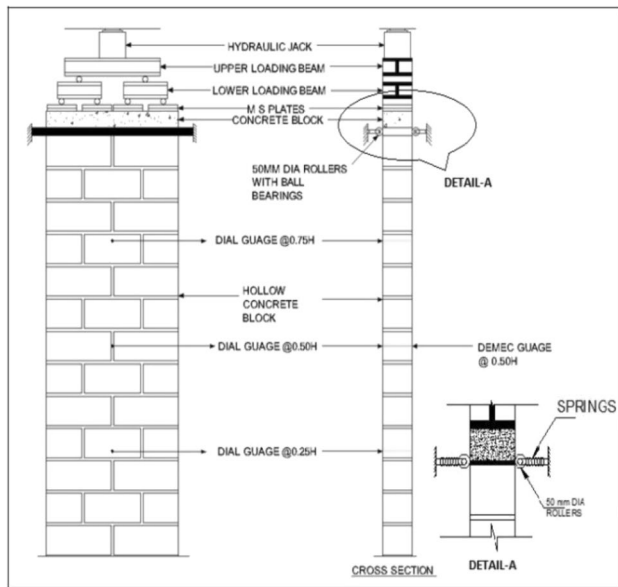


Fig. 2 Typical test setup of full-scale walls

arrangement for testing both axial and eccentric loads is as shown in Fig. 2.

4.3 Details of lateral restraint

To ensure that the wall has lateral and rotational restraint at the bottom (fixed condition) and only lateral restraint at the top, arrangements were made as shown in Fig. 3. Adequate care is taken to avoid out of plane movement of the wall at the top while allowing compression and rotation at higher loads. The lateral restraints are achieved by providing a beam of cross section on either side of the wall which is fixed on rigid supports. Two ball-bearing rollers are fixed onto each beam with a spring, such that the rollers are abutting the concrete blocks. The cross section of steel channels is stiff enough to avoid any lateral movement at the free end of the wall.

4.4 Details of testing

Two types of concrete blocks that are commercially available are identified in the present study. The first type, which is more common, is a cellular concrete block (CCB) which will have four or more holes in each block. These blocks have one bed face closed and are laid with this 'blind end' upward, preventing wastage of building mortar which would otherwise drop into cavities [2]. The other type is a hollow concrete block (HCB) which will have 2 or 3 holes with hole sizes generally larger than the ones used in cellular blocks and unlike the cellular blocks, hollow concrete blocks are open at both sides. Figure 3a shows



Fig. 3 Typical test setup for testing concrete block prism

typical CCBs and HCBs. Here, the word hollow concrete block will be used only to mean blocks with number of holes less than or equal to three. The void ratio for the HCB was determined to be 35% and 40% for 150-mm and 200-mm blocks, respectively, whereas for 200-mm-thick CCBs the value of void ratio was 27.34%.

The experimental program consists of two stages: in the first stage the basic properties of the concrete blocks and its prism compressive strength were determined, and in the second stage, the full-scale wall specimens were tested. The entire testing program is concentrated on the cellular concrete block and hollow concrete block masonry. The full-scale walls of two different sizes, i.e., 150-mm blocks and 200-mm blocks, were tested for hollow concrete blocks, whereas only 200-mm-thick blocks were tested for cellular concrete blocks.

The basic properties of the cellular concrete blocks (CCB) and hollow concrete blocks (HCB) used in prism and wall construction were determined as per the methodology in Indian code [1, 5] and ASTM [27] as applicable and are tabulated in Table 2. The results reported by Mangala Keshava [22], Somanath [36] and Manjunath

et al. [28] show a wide variation in the compressive strength and masonry efficiencies of two block prisms with cement–sand mortar of proportion 1:6. In the west, Drysdale et al. [29] and Hamid et al. [30, 31] also indicate the inadequacy of two-block prism test. However, three-block prism test results show lower variation in strength and masonry efficiencies. Recommendations made by the BIS 1905:1987 [1] with the use of correction factors are concerning h/t ratio of 2. The correction factor indicated in code sometimes leads to a prism strength which is more than the block strength. This is unlikely. Hamid et al. [32] and Bartolome et al. [33] also suggest that the slenderness ratio of 2.0 for normalization of prism strength is not satisfactory. Therefore, three-block prisms were considered

and tested for its compressive strength. The typical test setup is as shown in Fig. 4. The prism compressive strength is tabulated in Table 3. The correction factor is applied for the prism strength as per the codal recommendations [1]. However, in the construction stage of the wall, the representative mortar cubes were tested for its 28-day compressive strength (5.37 MPa).

A total of nine walls were tested, out of which five walls were subjected to the axial type of loading, and the remaining four walls were subjected to eccentric loading. All the nine wall specimens were tested on a loading frame as shown in Fig. 1, at the laboratory after curing the specimens for a minimum period of 28 days. To ensure proper curing, jute cloth which retains water for a longer time was

Table 2 Basic properties of concrete blocks

Type	N*	Cellular blocks		Hollow blocks	
		200 mm	N*	150 mm	200 mm
Dimensionality $L \times H \times B$ (mm)	20	387 × 203.8 × 200	20	398 × 200 × 149	400 × 200 × 200
Bulk density Kg/m^3 (% CoV)	20	1670.4 (2.43)	20	1245 (2.70)	1176 (2.6)
IRA $\text{Kg/m}^2/\text{min}$ (% CoV)	20	2.64 (17.10)	20	0.51 (42)	0.57 (28.88)
Water absorption (%) (% CoV)	20	5.81 (17.27)	20	2.72 (25.4)	5.03 (12.37)
Compressive strength* (MPa) (% CoV)	20	7.06 (14.66)		13.77 (8.84)	6.60 (20.3)
Initial tangent modulus (MPa)		4453		15,197	9373
Secant modulus @25% ultimate stress (MPa)		4258		14,652	9066
Chord modulus (CHM)		4154		14,362	8903

N* number of blocks tested, IRA initial rate of absorption conducted as per ASTM C67 [27], CHM chord modulus is the ratio of stress to strain measured between 5 and 33% of ultimate stress (MPa) [35]

*Compressive strength is computed based on the gross area of the block

Fig. 4 Typical cellular concrete blocks and hollow concrete blocks



Table 3 Strength of three block prisms

Type	Cellular blocks	Hollow blocks	
	200 mm	150 mm	200 mm
No. of specimens tested	5	5	5
Average prism strength (MPa) (% CoV)	3.91 (19.3)	6.80 (13.1)	5.41 (19.7)
h/t	3.25	3.94	2.94
Corrected prism strength (MPa)	3.41	6.32	4.60

used. Fishburn [34] emphasizes the importance of proper curing in improving the bond strength of walls. The bond strength of block assemblies which is representative of the compressive and flexural strength of walls is ensured with proper curing conditions. The mean temperature at the laboratory was around 27 °C to 30 °C which is within the range of temperature as suggested in his research document.

In order to measure the deflection at various heights of the wall, digital dial gauges are placed at three different heights, one at 0.25 H, the second at 0.5 H and the third dial gauge at 0.75 H (where H is the height of the wall) as shown in Fig. 2. In addition to this, strains are measured using the mechanical demec gauge, fixed on one face of the wall in case of axially loaded walls and on both faces of the wall in case of eccentrically loaded walls.

When the wall has lateral and rotational restraint at one end and only lateral restraint at the other, the effective height is 0.85H. The effective height of the wall may be taken as $0.85 \times$ actual height as per IS: 1905-1987 [1]. The slenderness ratio of 200-mm-thick masonry wall specimens was maintained up to 11.3 and 15 for 150-mm-thick masonry walls. For the wall specimens of hollow concrete block masonry, an eccentricity of 0.1 t (t = thickness of wall) is chosen which is in the range of values specified by

IS:1905-1987 [1]. Since eccentricities induce both compressive stress and bending stress, demec gauges are put on both sides of the wall. The details along with a description of wall specimens are presented in Table 4.

5 Results and discussions

5.1 Axially loaded walls

The test results of the five full-scale walls, viz. the ultimate load, first crack load, deflection at various incremental loads and ultimate stress, have been discussed for axially loaded walls in detail as below. Table 5 gives the ultimate strength, and Table 6 shows the stress reduction factors for the five walls tested. Due to unpredictability in the failure of specimen, the lateral deflection was recorded at a load of 180 kN for all specimens.

5.1.1 Wall no. 1 (CCB-200-A1)

The wall was subjected to axial compression, and the load was applied initially in increments of 10 kN up to the failure load. The longitudinal strains were recorded over a gauge length of 200 mm at the center on the one face of

Table 4 Details and descriptions of full-scale wall specimens

Sl. no.	Designation/ notation of the wall	Description	Dimensions $l \times t \times h$ (mm)	Slenderness ratio#	Eccentricity
1	CCB-200-A1	Cellular concrete block walls of 200 mm thick, axially loaded: wall no. 1	802×200×2667	11.28	0
2	HCB-200-A2	Hollow concrete block walls of 200 mm thick, axially loaded	822×201×2680	11.33	0
3	HCB-200-A3	wall nos. 2 and 3	800×200×2660	11.3	0
4	HCB-150-A4	Hollow concrete block walls of 150 mm thick, axially loaded	830×150×2680	15.18	0
5	HCB-150-A5	wall nos. 4 and 5	830×150×2680	15.18	0
6	HCB-200-E6	Hollow concrete block walls of 200 mm thick, eccentrically	843×199×2660	11.36	0.1 t
7	HCB-200-E7	loaded wall nos. 6 and 7	842×202×2620	11.02	0.1 t
8	HCB-150-E8	Hollow concrete block walls of 150 mm thick, eccentrically	823×150×2640	14.96	0.1 t
9	HCB-150-E9	loaded wall nos. 8 and 9	820×150×2620	14.84	0.1 t

Table 5 Test results of axially loaded masonry walls

Designation of walls	Dimensions of the walls (mm)	Block strength (MPa)	Prism strength (MPa)	Corrected prism strength (MPa)	SR	e	Wall compressive strength (MPa)	Avg. wall compressive stress (MPa)
CCB-200-A1	802×200×2667	7.06	3.91	3.41	11.28	0	3.30	3.30
HCB-200-A2	822×201×2680	6.60	5.41	4.60	11.33	0	3.28	3.11
HCB-200-A3	800×200×2660	6.60	5.41	4.60	11.3	0	2.94	
HCB-150-A4	830×150×2680	13.77	6.80	6.32	15.18	0	5.54	5.79
HCB-150-A5	830×150×2680	13.77	6.80	6.32	15.18	0	6.04	

Table 6 Stress reduction factor for axially loaded walls (experimental)

Designation of walls	Dimensions of the walls (mm)	Wall compressive strength (MPa)	Average wall compressive strength (MPa)	Average stress reduction factor Wall strength/corrected prism strength
CCB-200-A1	802×200× 2667	3.30	3.30	0.96
HCB-200-A2	822×201× 2680	3.28	3.11	0.68
HCB-200-A3	800×200× 2660	2.94		
HCB-150-A4	830×150× 2680	5.54	5.79	0.91
HCB-150-A5	830×150× 2680	6.04		

the wall, and on the other face, deflections were measured at the height of 0.25 H, 0.5 H and 0.75 H ('H' being the height of the wall) up to a load of 200 kN. The wall was further loaded continuously at the same rate up to failure. The height of the wall was maintained at 2.667 m indicating a slenderness ratio of 11.28. The first vertical crack was observed on the first two courses from the bottom of the wall at a load of 460 kN. This was followed by a horizontal crack in between the second and third course from the bottom of the wall. A vertical split along the thickness of the wall was also observed in all the bottom five courses. The various stages of crack formation and ultimate mode of failure are shown in Fig. 5. The strains and the lateral displacement were recorded up to a load of 180 kN, and a maximum deflection of 5.28 mm was observed at ¾th height of the wall. The ultimate stress at the base of the wall was about 3.30 MPa. The stress reduction factor which is defined as the ratio of wall strength to prism strength is recorded experimentally as indicated in Table 6.

5.1.2 Wall no. 2 (HCB-200-A2)

The wall was constructed using 200-mm-thick hollow concrete blocks whose average unit compressive strength was 6.60 MPa. The load arrangements were similar to the previous wall. The load was applied axially using a 100-ton capacity hydraulic jack. The deflections were measured up to the load of 300 kN, and after that further loading was applied removing the dial gauges. Figure 6 shows the deflection profile of the wall at various stages of loading along with the failure pattern. The wall exhibited a classical mode of failure due to the typical tensile splitting of blocks. The first vertical crack was observed at 480 kN at the top of the wall, and on further increase in the load, the vertical cracks developed throughout the height of the wall. The ultimate stress recorded was 3.28 MPa. The maximum deflection recorded was around 1.50 mm at the ¾th height of the wall at a load of 180 kN.

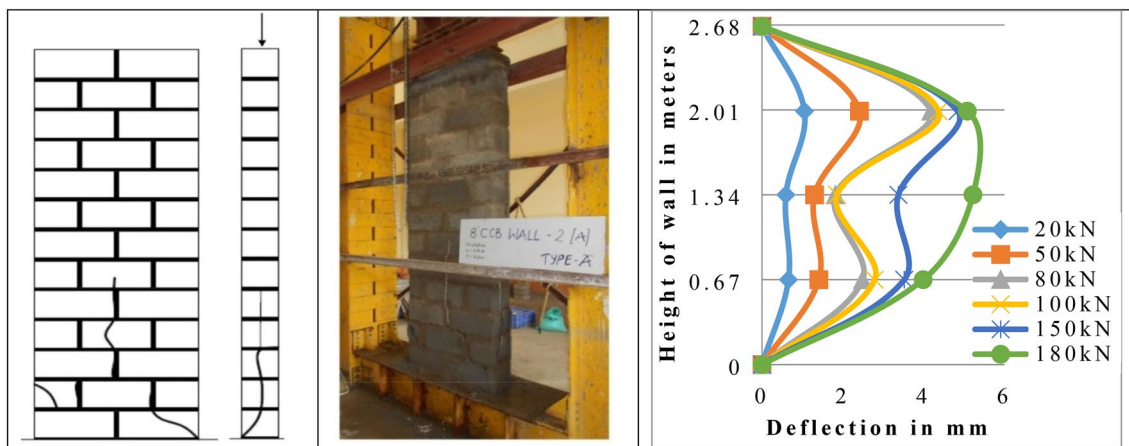


Fig. 5 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 1 (CCB-200-A1)



Fig. 6 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 2 (HCB-200-A2)

5.1.3 Wall no. 3 (HCB-200-A3)

The wall was constructed similarly to the wall no. 2. The wall is constructed to a height of 2.66 m with slenderness ratio of 11.3. The crack originated from the perpend joint of the second course of the wall. On further loading, the vertical crack propagated up to the eighth course from the top. Similar to the previous wall results, considerable fixity was observed at the base of the wall resulting in no rotation. The maximum deflection recorded was around 3.67 mm at $\frac{3}{4}$ th height of the wall at a load of 180 kN. The ultimate stress recorded at the base of the wall was 2.94 MPa. The crack pattern and deflections recorded are indicated in Fig. 7. Wall no. 2 and wall no. 3 were

constructed using similar block type and dimensions. The average compressive strength is calculated to be 3.11 MPa. The stress reduction obtained experimentally is reported in Table 6.

5.1.4 Wall no. 4 (HCB-150-A4)

The wall was constructed using 150-mm-thick hollow concrete blocks. The mean compressive strength of the block was 13.77 MPa. The cross section dimension of the wall was 830 mm × 150 mm with a height of the wall being 2.68. The slenderness ratio of the wall was 15.18. The wall was tested under axial compression, and deflections were recorded at three different heights, viz.

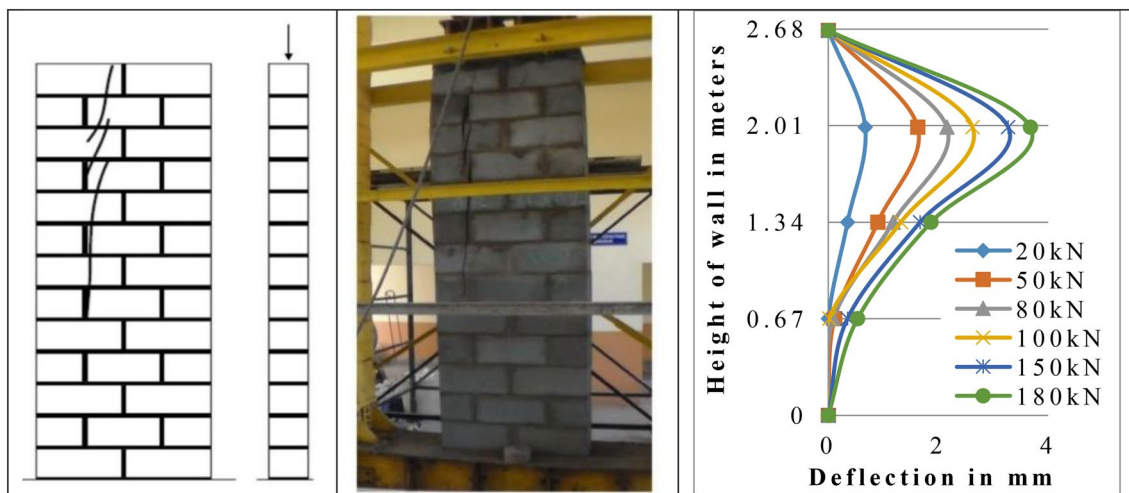


Fig. 7 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 3 (HCB-200-A3)

0.25 H, 0.5 H and 0.75 H (*H* being the height of the wall). The first crack was observed at the top most course of the wall at the load of 540 kN. On further increase in the load, crack propagated up to the fifth course from the top of the wall as shown in Fig. 8. The maximum deflection of 2.56 mm was recorded for the load of 180 kN. The ultimate load of the wall was 690 kN indicating an ultimate stress of 5.54 MPa. It is interesting to note that the wall strength constructed using 150-mm blocks is better than the 200-mm-thick blocks. This may be attributed to the shorter length of the web, consequently increasing its stiffness.

5.1.5 Wall no. 5 (HCB-150-A5)

All the specifications for this wall were similar to that of the wall no. 4. A faint horizontal crack was observed at the top of the third course from the bottom of the wall at a load of 620 kN. The deflection at 180 kN was observed as 1.00 mm. The wall was further loaded and finally collapsed suddenly without caution at a load of 752 kN, into masonry units as shown in Fig. 9. This indicates the unpredictability in failure of masonry specimen. The ultimate stress recorded at the base of the wall was 6.04 MPa. Both wall no. 4 and wall no. 5 were with similar specifications. The average compressive strength is recorded as 5.79 MPa as indicated in Table 5.

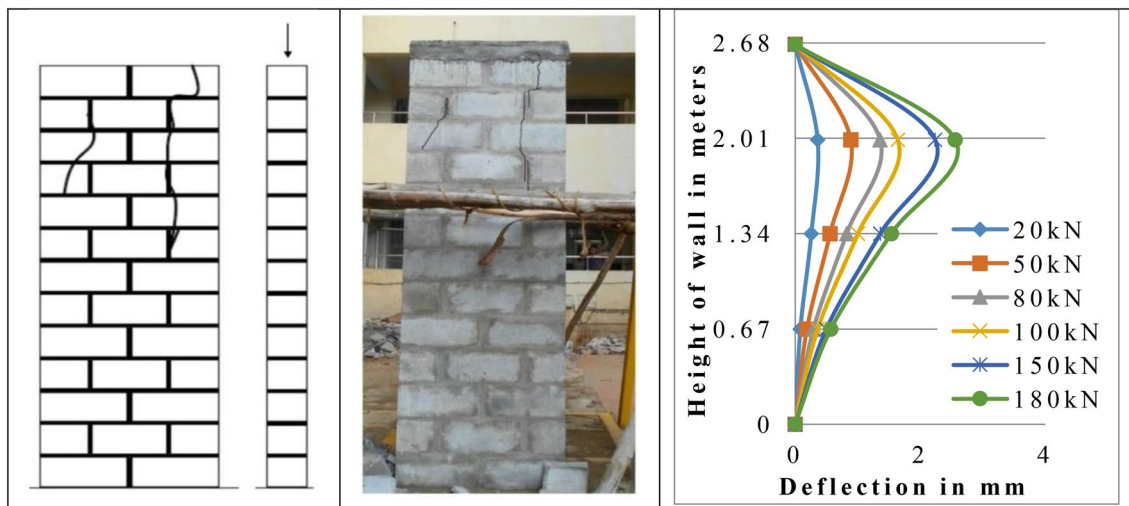


Fig. 8 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 4 (HCB-150-A4)

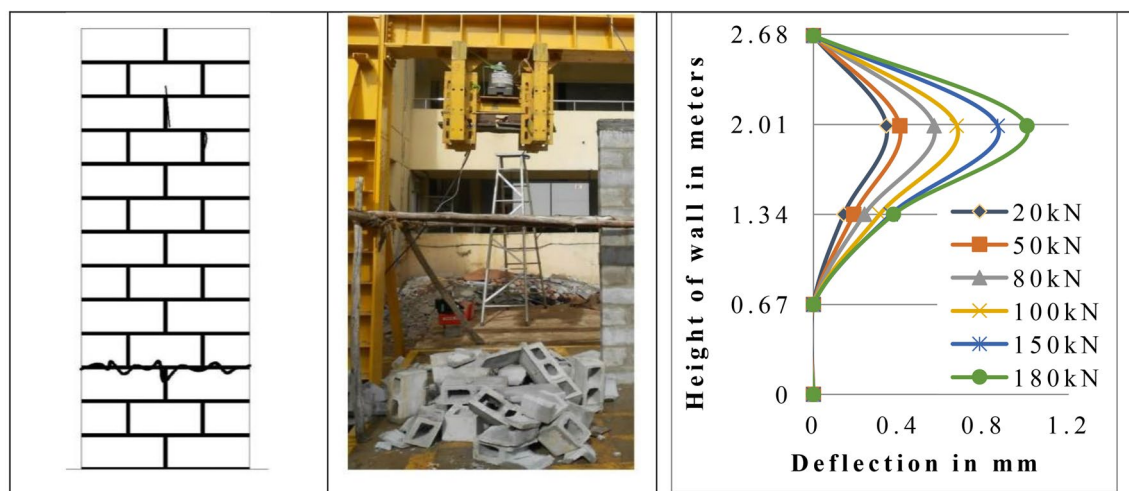


Fig. 9 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 5 (HCB-150-A5)

5.2 Eccentrically loaded walls

The strength of a wall depends on the strength of masonry, slenderness ratio and eccentricity of loading. The Indian code BIS: 1905: 1987 [1] specifies reduction factors due to slenderness ratio and eccentricity. In the previous section, the effect of slenderness was investigated on axially loaded walls. However, in this section, the effect of eccentricity is investigated experimentally.

According to Bureau of Indian Standards (BIS), the point of application of the load on a wall is the center of the bearing on the wall; if the span of roof slab is less than 30 times the wall thickness. When the span is more than 30 times the wall thickness, the point of application of load is considered to be displaced from the center of bearing toward the span of the floor to the extent of 1/6th the bearing width.

In the present study, the effect of eccentricity is investigated on hollow concrete blocks (HCBs) only. Table 7 presents the test results of four eccentrically loaded hollow concrete block masonry specimens, of which two walls are of 150 mm thickness, and other two walls are of 200 mm thickness. The eccentricity for all the four specimens is taken as 0.1t as shown in Fig. 10. The test results of the four full-scale walls, viz. the ultimate load, first crack load, deflection at various incremental loads and ultimate stress, have been discussed for eccentrically loaded walls in detail as below.

5.2.1 Wall no. 6 (HCB-200-E6)

The wall was constructed using 200-mm-thick hollow concrete blocks. The cross section dimension of the wall was 843 mm × 199 mm with a height of wall being 2.66 m. The wall constructed had slenderness ratio of 11.36. The wall was subjected to moment, induced due to eccentric loads. Eccentricity ratio of 1/10 was maintained for the wall. The wall developed vertical and horizontal cracks on the tensile face of the wall as indicated in Fig. 11. The cracks extend up to the base. The maximum deflection recorded

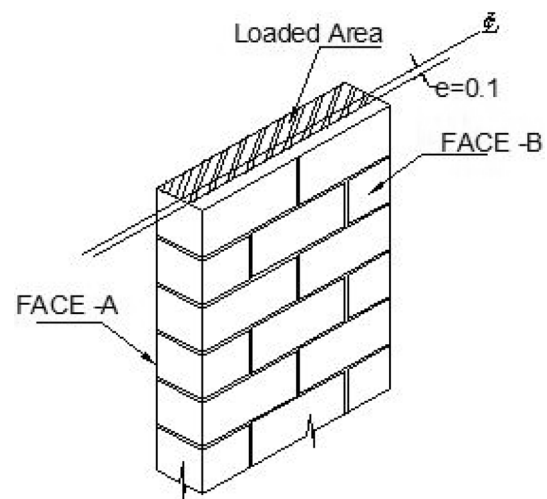


Fig. 10 Schematic representation of the eccentrically loaded wall

was around 5.93 mm at the 3/4th height of the wall from the base at a load of 180 kN. The wall failed at a stress of 2.20 MPa as indicated in Table 7. It may be noted from Table 7 that there is a reduction in the ultimate compressive strength of the wall when eccentric loading is induced on the wall.

5.2.2 Wall no. 7 (HCB-200-E7)

All the specifications for this wall were similar to that of the wall no. 6. A faint vertical crack was observed at the top courses initially followed with vertical cracks propagating to the middle and bottom courses. It is interesting to note that uniform maximum deflections were observed at 1/4th and 3/4th height of the wall indicating rotations at both ends of the wall. It was also observed that due to eccentric loading most of the cracks appeared to be concentrated at the top three courses of the wall. The deflections at various heights and loadings and the failure pattern of the wall are indicated in Fig. 12. Both wall no. 6 and wall no. 7 were with similar specifications. The average compressive

Table 7 Test results of eccentrically loaded masonry walls

Designation of walls	Dimensions of the walls (mm)	Block strength (MPa)	Prism strength (MPa)	Corrected prism strength (MPa #)	SR	<i>e</i>	Wall compressive strength (MPa)	Avg. wall compressive stress (MPa)
HCB-200-E6	843 × 199 × 2660	6.60	5.41	4.60	11.36	0.10 t	2.2	2.78
HCB-200-E7	842 × 202 × 2620	6.60	5.41	4.60	11.02	0.10 t	3.36	
HCB-150-E8	823 × 150 × 2640	13.77	6.80	6.32	14.96	0.10 t	5.11	4.74
HCB-150-E9	820 × 150 × 2620	13.77	6.80	6.32	14.84	0.10 t	4.37	

#Correction factor [1]

SR slenderness ratio, *e* eccentricity, *t* thickness of wall

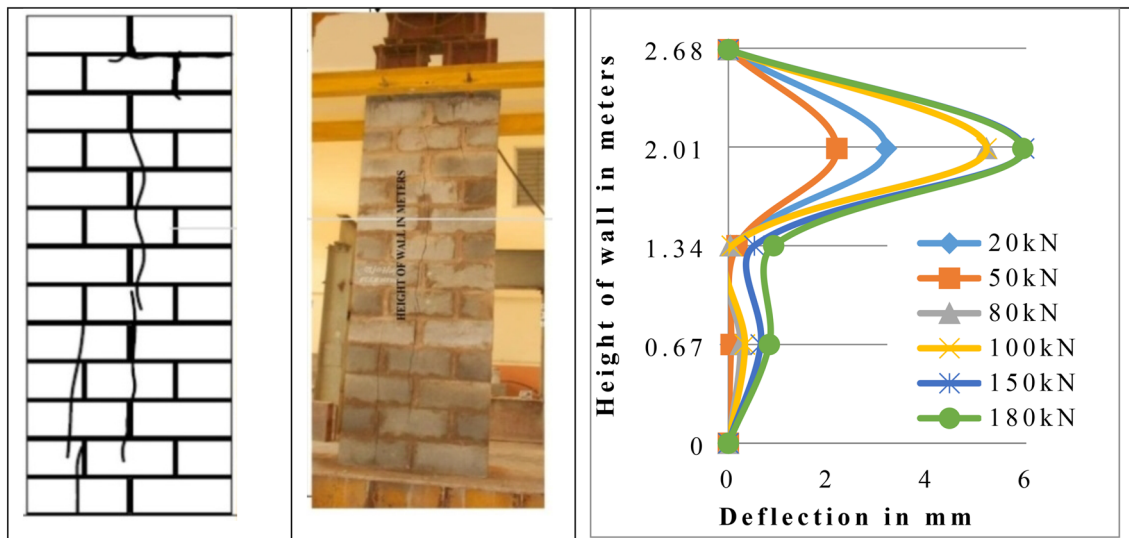


Fig. 11 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 6 (HCB-200-E6)

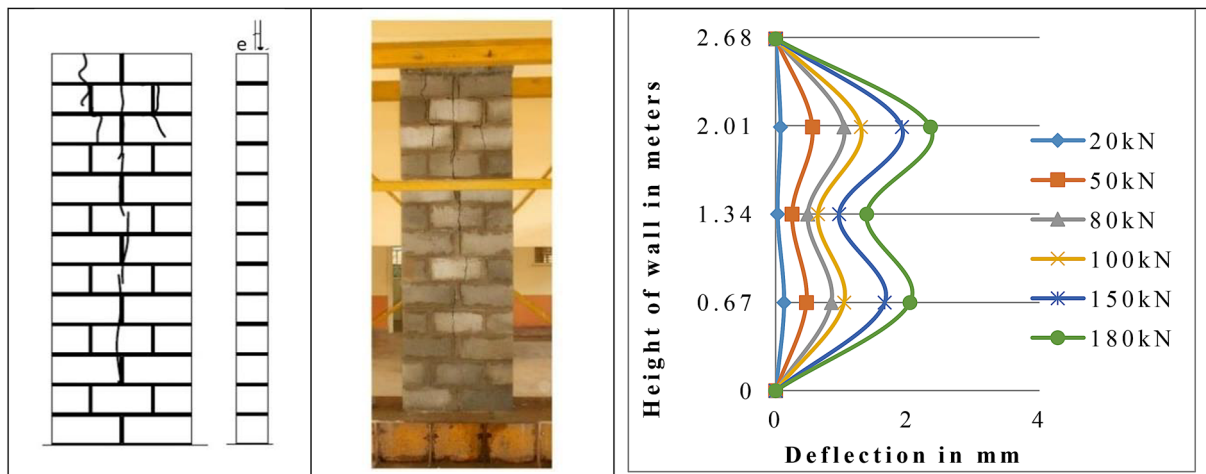


Fig. 12 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 7 (HCB-200-E7)

strength was recorded as 2.78 MPa as indicated in Table 5. The average stress reduction factor obtained experimentally is determined as 0.60 for these walls as indicated in Table 8.

5.2.3 Wall no. 8 (HCB-150-E8)

To study the effect of eccentricity on 150-mm hollow concrete blocks, similar tests as conducted earlier were carried out. The wall was constructed using 150-mm hollow concrete blocks. The wall dimension was 823 mm × 150 mm with the height of the wall as 2.64 m. The slenderness ratio was calculated as 14.96. The eccentricity ratio was maintained as 1/10 similar to that of 200-mm hollow concrete blocks tested earlier. Due to the application of eccentric

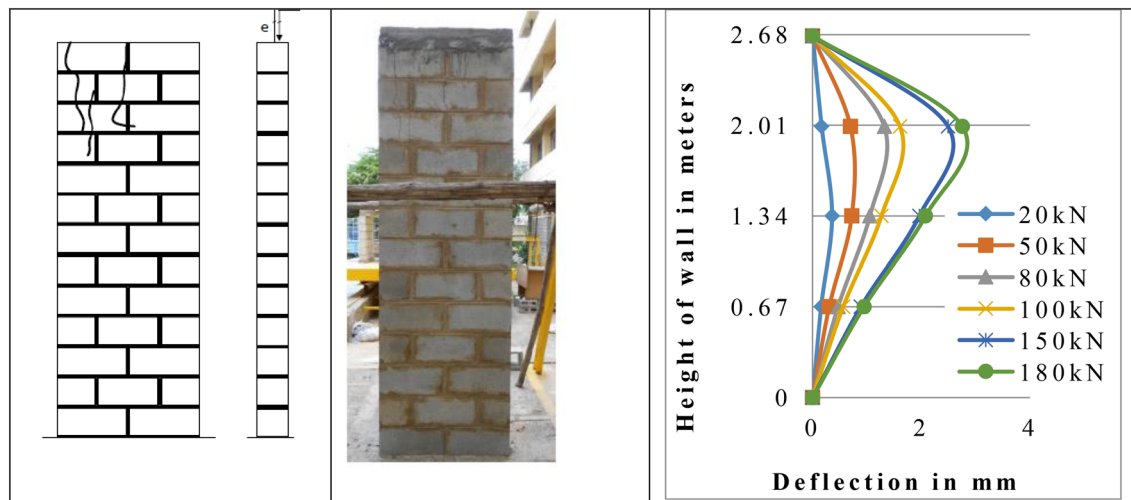
loading, the damage is confined to the top four courses of the wall only. The failure patterns and the distribution of lateral deflections are indicated in Fig. 13. The maximum deflection recorded was around 2.75 mm at the 3/4th height of the wall from the base at a load of 180 kN as shown in Fig. 13. The maximum stress recorded at the base of the wall is 5.11 MPa which is lesser than that obtained for axially tested walls.

5.2.4 Wall no. 9 (HCB-150-E9)

All the specifications for this wall were similar to that of the wall no. 8. This wall also experienced similar failure pattern as that of the previous wall no. 8. The formations of faint vertical cracks were observed at the top three courses only.

Table 8 Stress reduction factor for eccentrically loaded walls (experimental)

Designation of walls	Dimensions of the walls (mm)	Wall compressive strength (MPa)	Average Wall compressive strength (MPa)	Avg. stress reduction factor Wall strength/ corrected prism strength
HCB-200-E6	843 × 199 × 2660	2.2	2.78	0.60
HCB-200-E7	842 × 202 × 2620	3.36		
HCB-150-E8	823 × 150 × 2640	5.11	4.74	0.75
HCB-150-E9	820 × 150 × 2620	4.37		

**Fig. 13** Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 8 (HCB-150-E8)

It was also interesting to observe material failure regarding crushing of topmost course on the loading face of the wall. The significant rotation was also observed at both ends of the wall from load deflection curves evaluated for various stages of loading. The load deflection and failure pattern are as indicated in Fig. 14. The stress developed at the base of the wall is 4.37 MPa.

Both wall no. 8 and wall no. 9 were with similar specifications. The failure patterns observed were almost similar with considerable damage at top courses only. The average compressive strength was recorded as 4.74 MPa as indicated in Table 7. The average stress reduction factor obtained experimentally is determined as 0.75 for these two walls as indicated in Table 8.

For the eccentrically loaded walls, as shown in Fig. 10, strain measurements were made on both faces of walls. A 200-mm demec gauge was placed on face A, and a 150-mm demec gauge was placed on face B to measure the strains. The strains were measured up to a load of 200 kN. Despite eccentric loading, it was observed that the walls

experienced compressive strains on both faces. Along the face of the wall where eccentric loading was applied (face A), the strains measured were found to be higher than the strains on the opposite face (face B) as indicated in Table 9. This may be attributed to the fact that a couple generated at the top of the wall due to eccentricity is inducing bending in the wall. This may be validated by the local failure confined at the top of the wall as observed in Figs. 11, 12, 13, 14.

It may be noted from Tables 5 and 7 that the strength of cellular concrete block wall of 200 mm thickness is comparable with the strength of hollow concrete block wall of 200 mm thickness, even though the hollow concrete blocks are stronger than a cellular concrete block. The cellular concrete block can be profitably used for load-bearing masonry if there is no need for vertical reinforcement for walls.

The considerable weakening of the hollow concrete block in the thinness of its web is probably the reason. The 150-mm hollow concrete block wall, however, is stronger

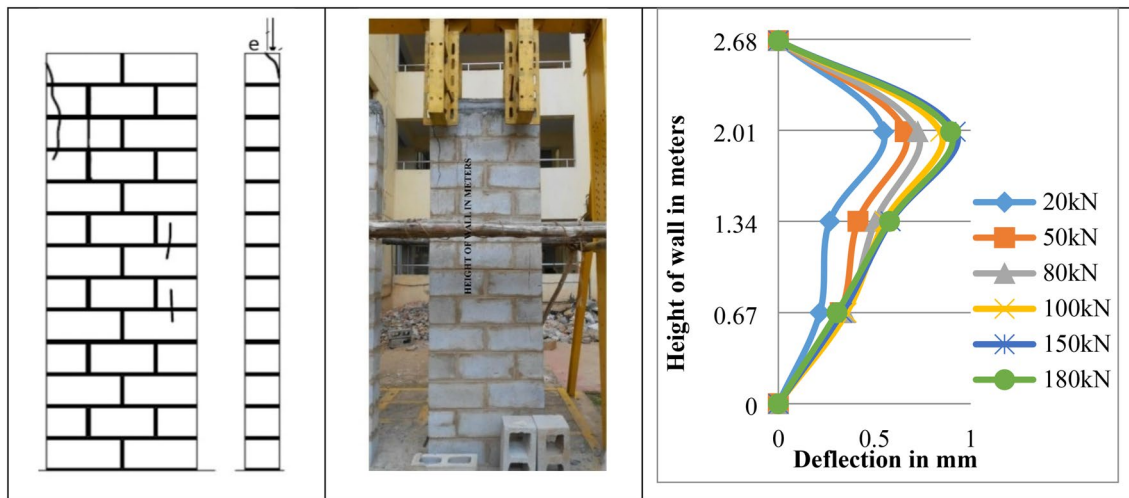


Fig. 14 Views of failure patterns and distribution of lateral deflection along the height at various stages of loading of wall no. 9 (HCB-150-E9)

Table 9 Variation of strain for eccentrically loaded walls

Sl. no	Wall type; thickness (mm)	Eccentricity	Stress at a load of 200 kN	Strain along face A	Strain along face B
1	Hollow concrete blocks; 200	0.1	1.186 MPa	0.000263	0.00019
2	Hollow concrete blocks; 150	0.1 t	1.62 MPa	0.001	0.000065

than the 200-mm-thick hollow concrete block wall. This may also be attributed to the shorter length (consequently greater stiffness) of the web. Figures 5, 6, 7, 8 and 9, show the lateral deflections of the wall at various stages of loading. It was observed that the wall shows practically no rotation at the base while there is a considerable rotation at the top of the walls. This means that the walls loaded axially have both lateral and rotational restraints at the base and only lateral restraints at the top which is achieved by roller support on both sides of the walls. For axial loading, the wall fails essentially by developing vertical splitting cracks and crushing of blocks in places. For eccentrically as indicated in Figs. 11, 12 and 13, loading the damage is confined to the top portion of the loading probably because the wall is restrained both against lateral displacement and rotation.

6 Comparison of reduction factors

Depending on the characteristics of the masonry materials, boundary conditions of the wall and the type of loading adopted, that is axial or eccentric loading, there is a considerable amount of reduction in the load carrying capacity of a masonry wall. This reduction is accounted in the design requirement of masonry by all the standards

[1, 10–13] by using a suitable term which takes into consideration all the above-mentioned factors. In the Indian standards [1], this factor is expressed as stress reduction factor (SRF) which is defined as the ratio of the wall strength to the masonry prism strength. The British code (BS:5628) [10] and the Eurocode (EN-1996-1-1) [11] address the reduction in the load carrying capacity of wall through a term capacity reduction factor (Φ_m). The Masonry Standards Joint Committee [35], however, uses the term as the strength reduction factor, while the Canadian code CSA 304.1 [12] uses the term force reduction factor.

As per the Indian code, in order to obtain the permissible compressive strength of masonry, the basic compressive stress is multiplied by three factors, namely area reduction factor, shape modification factor and stress reduction factor. The code has specified a series of stress reduction factors for various values of slenderness ratio (SR 6-27) and eccentricity ratios $e/t=0$ to $e/t=0.33$.

Table 10 presents the reduction factors including SRF computed from the experimental investigation on all nine walls for both axial and eccentric loading cases. The wall strength indicated is the ultimate strength of the walls obtained experimentally. The prism strength tested under uniaxial compression is the corrected compressive strength as tabulated in Table 3. The SRF is thus computed by taking the ratio of the wall strength and

Table 10 Comparison of reduction factors as prescribed by various codes

Wall type; thickness in mm/notation	SR	Eccentricity		SRF IS code [1]	Capacity reduction factors (Φ_m)			SRF from experi- ments
		e/t	e_{mk}/t		BS code [10]	Eurocode [11] 700 f_k	AS-3700 [13]	
CCB; 200-A1	11.28	0	0	0.86	0.94	0.88	0.84	0.96
HCB; 200-A2	11.33	0	0	0.86	0.94	0.88	0.84	0.71
HCB; 200-A3	11.30	0	0	0.86	0.94	0.88	0.84	0.64
HCB; 150-A4	15.18	0	0	0.75	0.86	0.78	0.72	0.87
HCB; 150-A5	15.18	0	0	0.75	0.86	0.78	0.72	0.95
HCB; 200-E6	11.36	0.10	0.05	0.82	0.87	0.77	0.72	0.48
HCB; 200-E7	11.02	0.10	0.05	0.81	0.85	0.78	0.73	0.73
HCB; 150-E8	14.96	0.10	0.05	0.70	0.80	0.68	0.62	0.80
HCB; 150-E9	14.84	0.10	0.05	0.70	0.80	0.68	0.62	0.69

SR slenderness ratio

the corrected prism strength experimentally and is presented as SRF from experiments in the table. Further, these SRFs were compared with the factors as that given in the IS code which appears in Table 9 of the IS:1905-1987. The values tabulated in Table 10 are the value that was obtained after interpolating the values given in IS code for the corresponding values of SR and e/t combinations. Similarly, respective reduction factors (capacity/strength/force) as prescribed by the other standards are reported and compared with the IS code as shown in Table 10.

In Eurocode [11], at the ultimate limit state, the design value of the vertical load applied to a masonry wall shall be less than or equal to design value of the vertical resistance of the wall. The design value of the vertical resistance is calculated by the product of design compressive strength of the masonry, wall thickness and the capacity reduction factors (Φ). The capacity reduction factor can be calculated at the top or bottom of the wall as appropriate, allowing for the effects of slenderness and eccentricity of loading, for any modulus of elasticity E and characteristic compressive strength of unreinforced masonry f_k , as mentioned in the clause 6.1.2 of the code. The annexure G of the Eurocode gives an illustrative procedure of determination of reduction factors. Although the capacity reduction factor can be obtained for various values of modulus of elasticity, in Table 10, the capacity reduction factor is computed for a $700 f_k$, where f_k is the characteristic compressive strength of masonry with 700 as the modulus of elasticity which is closer to quality of blocks in India. A similar philosophy to determine the reduction factor is adopted in Australian standards (AS-3700), where Table 7.3 of the Australian code gives the reduction factor in order to achieve lateral stability. The values are also tabulated in Table 10. Similarly, the capacity reduction factor as prescribed by the British code [10] is also computed and tabulated in Table 10;

however, the British code is now obsolete and is not being used and is superseded by the Eurocode.

It may be noted from Table 10 that the stress reduction factor obtained experimentally for cellular concrete block wall appears to be higher than the values specified by the various codes. It can be seen from the results of CCB walls that the codal values are on a conservative side. However, more number of specimens should be tested for CCB walls to ascertain the same. A similar trend was also observed for 150-mm hollow concrete block walls when an average value is taken for both walls. This argument also holds good with the results presented for 150-mm HCB walls by Mangala Keshava [22, 23] and brick masonry walls presented by Gumaste [21] and Somanath [36], whereas 200-mm hollow concrete block walls indicate much lower values of experimental stress reduction factors as compared to codal recommendations. Hence, there is a need to revisit the values of the stress reduction factor by testing more number of hollow concrete block walls. Also, it is suggested to present the stress reduction factor values separately for brick work and block work masonry. It may be noted from the table that the experimental stress reduction factor for 200-mm HCB walls shows lower values of SRF specified by various codes. However, for 150-mm HCB walls the Indian code and British code values seem to be marginally comparable.

7 Summary and conclusion

This paper highlights the experimental results carried out on nine full-scale hollow concrete block walls. The first wall was with cellular concrete blocks, followed by four axially loaded walls of two different thickness (150 mm and 200 mm thick) and finally four walls of two different thicknesses and with an eccentricity of 0.1. From the

experimental investigations carried out, the following conclusions may be drawn.

- The axially loaded masonry walls can take higher loads as compared to the eccentrically loaded masonry specimens which are as expected. However, for the same thickness, the cellular block masonry exhibited marginally higher load-carrying capacity than hollow concrete block masonry.
- The test results confirm the better performance of hollow concrete block masonry as compared to brick masonry wall specimens tested by previous researchers. This may be due to the bigger size of the masonry units as compared to brick masonry and fewer mortar joints.
- It is very interesting to note that 150-mm-thick hollow concrete masonry walls have higher masonry strength as compared to that of 200-mm-thick masonry. It may be due to a smaller opening in a 150-mm-thick block, making it stiffer. A similar observation was seen in the work of Thamboo et al. [24] where it is observed that the presence of end web shells increases the compressive strength and stiffness of the masonry.
- The SRF values for 200-mm hollow concrete block walls indicate much lower values of experimental stress reduction factors as compared to code recommendations for both axial and eccentrically loaded cases. A separate provision may be provided in IS 1905-1987 for stress reduction factors for brick work and block work.
- The applied eccentric load varies from a maximum at the top of the wall, that is, at the point of loading and is possibly zero at the bottom. Taking the combined effect of eccentricity of loading and slenderness, critical stress is at section 0.6 H above the base as proposed in the draft commentary on structural use of masonry [37]. In the present investigation, the first cracks were observed above the mid-height or approximately 0.7 to 0.8 times the height of wall specimens. This seems to agree with the draft code commentary. A similar conclusion was seen in the work of Mangala Keshava [23].
- The investigations suggest that stress reduction factors indicated in the Indian code seem to overestimate for eccentrically loaded hollow concrete blocks and marginally underestimate in axially loaded 150-mm hollow concrete blocks. However, more numbers of samples may be required to test to ascertain the same.

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Compliance with ethical standards

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

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