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Compressive strength of masonry constructed with high strength concrete blocks

Resistência a compressão da alvenaria estrutural com blocos de concreto de alta resistência

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Abstract

Although the use of high strength concrete blocks for the construction of tall buildings is becoming common in Brazil, their mechanical properties and behavior are not fully understood. The literature shows a gap in experimental studies with the use of high strength concrete blocks, i.e., those with compressive strength greater than 16 MPa.

The work presented herein was conducted in order to study the behavior of high strength structural masonry. Therefore, the compressive strength and modulus of elasticity of concrete block walls tested under axial load were assessed. The specimens included grouted and ungrouted walls and walls with a mid-height bond beam; ungrouted walls were constructed with face-shell and full mortar bedding. The walls were built and tested in the laboratory of CESP and in the Structures Laboratory of the UNESP Civil Engineering Department in Ilha Solteira (NEPAE). Concrete blocks with nominal compressive strength of 16 (B1), 24 (B2) and 30 (B3) MPa were used. Ungrouted masonry walls had a height of 220 cm and a width of 120 cm while grouted masonry walls had a height of 220 cm and a width of 80 cm. Traditional Portland cement, sand and lime mortar was used. The testing program included 36 blocks, 18 prisms, 9 ungrouted walls (6 with face-shell mortar bedding and 3 with full mortar bedding), 9 grouted masonry walls, and 12 ungrouted walls with a bond beam at mid-height.

The experimental results were used to determine the compressive strength ratio between masonry units, prisms and masonry walls. The analyses included assessing the cracking pattern, the mode of failure and the stress-strain curve of the masonry walls.

Tests results indicate that the prism-to-unit strength ratio varies according to the block strength; that face-shell mortar bedding is suitable for high strength concrete masonry; and that 20% resistance decrease for face-shell mortar bedding when compared with full mortar bedding is a conservative consideration. The results also show that using a bond beam at the mid-height of the wall does not lead to a compressive strength decreased but it changes the failure mode and the shape of the stress-strain curve. In addition, the results show that estimating E = 800 fp is conservative for ungrouted masonry walls but reasonably accurate for grouted masonry walls and that there is no reason to limit the value of E to a maximum value of 16 GPa. Furthermore, the results show that, for design purposes, a wall-to-prism strength ratio value of 0.7 may be used for high strength concrete masonry.

Keywords: structural masonry, concrete block, high strenght, compressive strenght, high-rise buildings.

Resumo

Ainda que o uso de blocos de concreto de alta resistência para a construção de edifícios altos esteja se tornando comum no brasil, as características e alguns aspectos do comportamento não são totalmente conhecidos. A literatura mostra uma lacuna em estudos experimentais com a utilização de blocos de concreto de alta resistências, acima de 16 MPa.

O trabalho aqui apresentado foi realizado com o objetivo de estudar o comportamento da alvenaria estrutural de alta resistência. Para tanto foram estudadas a resistência a compressão e o modulo de elasticidade em paredes de blocos de concreto ensaiadas sob carregamento axial, divididas em paredes ocas, paredes grauteadas, paredes com cinta grauteada a meia altura e paredes com assentamento parcial e total. As paredes foram construídas e ensaiados no laboratório da CESP e no laboratório de Estruturas do Dep. De Engenharia Civil da UNPESP de Ilha Solteira (NEPAE). Foram utilizados blocos de concreto com valores nominais de resistência à compressão de 16 (B1), 24 (B2) e 30 (B3) MPa. As paredes ocas foram construídas com altura de 220 cm e largura de 120 cm, enquanto as paredes grauteadas foram construídas com altura de 220 cm e largura de 80 cm, utilizando argamassa tradicional de cimento, areia e cal. Foram ensaiados 36 blocos, 18 prismas, 9 paredes ocas (6 com argamassa apenas na lateral dos blocos e 3 com argamassa sobre toda a face desses), 12 paredes grauteadas, e 12 paredes ocas onde foi introduzida uma canaleta grauteada a meia altura.

A análise dos resultados experimentais possibilitou verificar a relação entre a resistência a compressão das unidades de alvenaria, dos prismas e das paredes de alvenaria. Foi também analisada a fissuração, modo de ruptura e curva tensão – deformação das alvenarias ensaiadas.

Através dos resultados dos ensaios verificou-se que o valor da relação de resistência prisma/bloco varia conforme a resistência do bloco; que o procedimento executivo com argamassa apenas na lateral é adequado para blocos de concreto de alta resistência, sendo conservadora a consideração de diminuição de resistência de 20% quando comparada com casos com argamassa disposta sobre toda a face dos blocos; que o uso de cinta grauteada à meia altura das paredes não levou a diminuição da resistência a compressão, mas alterou a forma de ruptura e a forma da curva tensão-deformação; que os módulos de elasticidade medidos nas paredes ensaidas foram sempre maiores que 800 fp para paredes ocas e entre 688 e 848 fp para paredes grauteadas, não sendo verificado a necessidade de limitar E ao valor máximo de 16 GPa. Por último, foi verificado que o valor da relação de resistência parede/prisma igual a 0,7 pode ser adotado para blocos de concreto de alta resistência.

Palavras-chave: alvenaria estrutural, bloco de concreto, alta resistência, resistência a compressão, edifícios altos.

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1. Introduction

Structural design in masonry requires a clear understanding of the behavior of the mortar and unit assembled to resist different load conditions. The combination of blocks and mortar of different characteristics, changes significantly the behavior of structural masonry elements. In structures with these elements, the compressive strength of the masonry is the controlling mechanical property for the project. The compressive strength of masonry depends on several factors such as: mortar strength; unit strength; relative ratio between the mortar and unit strength; relationship between the height of the unit and the smaller horizontal dimension of the unit; orientation of the unit in relation to the direction of the load application; and the thickness of the mortar joint. The large number of factors, individually and combined, indicates, therefore, the complexity of making a precise evaluation of the masonry strength.
According to Parsekian et al. [1], masonry compressive strength depends on large-scale on the block type and to a lesser extent on the workforce, and yet to a lesser extent on the type of mortar.
Hendry et al. [2] discuss more in deep the important factors affecting compressive strength of masonry. These factors are presented in Table 1.
Curtin et al. [3] simplify the analyses of masonry. by indicating that the characteristic resistance of masonry to compressive loads depends on the characteristic strength of the unit; on the specified mortar if the masonry is mortared; on the units' shape; on the thickness of the mortar joints; and on the workmanship.
According to ABNT NBR 15961-1 [4], the characteristic compressive strength of the masonry, f_k. should be determined based on testing of walls (ABNT NBR 8949[5]) or be estimated as 70% of the characteristic strength of the masonry prism, f_{pk}, or 85% of the characteristic strength of small walls, f_{ppk}. These values are determined using gross area.
The EUROCODE 6 [10] gives two equations to determine the masonry compressive strength: one for regular 10-mm mortar joint and one for 3-mm or less thin mortar joint. These formulations consider the compressive strength of the block, the average compressive strength of the mortar, the mortar joint thickness and a factor k that depends on the type of block and mortar:

$f_k = kf_b^{0.7} f_m^{0.3}$ - Equation 1 - masonry with 10-mm mortar joint;

$f_k = kf_b^{0.85}$ - Equation 2 - masonry with 3 mm or less thin mortar joint. with k = 0.5 for hollow concrete block.

Masonry has a brittle behavior, is non-homogeneous and anisotropic, and is composed of two materials with very different mechanical properties: the more rigid block and the relatively deformable mortar; as needed. grout and reinforcement are added. Masonry has very low tensile strength because the different materials are distributed at regular intervals, and the connection between them is weak. Therefore. unreinforced masonry, which is built extensively, should be expected to mainly resist compression loads.
Masonry under compression experience three main modes of rupture. depending on the relationship between the compressive strength of the mortar and that of the block:

a) When the mortar is very weak compared to the block. the masonry capacity is limited by the strength of the mortar. which usually fails by crushing;
b) When the mortar has a moderate strength, the masonry capacity is determined by a combination of the compression and tension strength of the block, which usually fails by lateral tension;
c) When the mortar is stronger than the block, masonry capacity is limited by the block compressive strength.

The more the masonry gets closer to failure mode "c". the more brittle and explosive the failure is. The preferred failure mode is more "b". which is a middle ground allowing the masonry to reach an adequate compressive strength without experiencing an extremely brittle failure. in addition to reducing the potential for crushing if the mortar and cracking at the mortar joint. For typical masonry. Parsekian et al. [1] recommend that the resistance of mortar is specified between 70 to 150% of the strength of the block (gross area).
During compressive testing of masonry walls constructed with high-strength blocks and with mortar with moderate, aproximatelly strength between 70 and 150% of that of the block, the mortar has a greater tendency to expand laterally in relation to blocks, because the later has a higher stiffness. However, the mortar is confined laterally on the block-mortar interface; therefore. shear stresses develop on the interface block-mortar. Thus, the mortar in the mortar joint is under a triaxial state of stress while the blocks experience bilateral tension. This stress state leads to vertical cracks in the blocks and eventual rupture of the wall (McNary and Abrams [11]; Atkinson and Noland [12]; Drysdale et al. [13]).
On Table 2 is summarized some experimental results for masonry compressive strength. Unless otherwise specified, the data in this table refer to strength in the gross area, 100x50mm cylindrical mortar specimens, two-block prisms, and prisms and walls not grouted. Cavalheiro and Gomes [14] summarized and analyzed

Table 1
Variables that affect masonry strength

Block properties	Mortar properties	Masonry
Strength	Strength	Connection
Type and geometry	Mix design	Load direction
Height/thickness ratio	Water/cement ratio	Concentrated stress
Absorption	Water retentivity	–
–	Elastic properties compared to block elastic properties	–

multiple tests of blocks, prisms and walls of concrete blocks (largest strength = 10 MPa), with strength ratios given in Table 3.
Other researchers, (ROMAGNA [19]; MAURÍCIO [20]) have conducted compressive strength tests, but with low and moderate strength blocks.

Fortes [21] conducted an experimental testing program with concrete blocks from, 4 to 35 MPa, and determined the block and prism behavior. Figure 1 shows the results of the prism/block strength ratio for several strengths. As shown, the prism/block strength ratio decreases with increasing block strengths.

Table 2
Brazilian experimental research results on masonry strength (MPa)

		f_b	f_a	f_p	f_{gk}	f_{par}	Note
ALY [15]	Hollow concrete blocks	7.90	–	6.40	–	5.10	–
		10.60	–	9.30	–	6.40	–
		13.30	–	9.80	–	8.30	–
		7.90	–	6.40	–	8.10	Fully grouted
		10.60	–	9.30	–	9.90	
		13.30	–	9.80	–	11.10	
		7.90	–	6.40	–	10.00	Fully grouted 0.2% of reinforcement
		10.60	–	9.30	–	10.80	
		7.90	–	6.40	–	9.20	Fully grouted 0.3% of reinforcement
		10.60	–	9.30	–	12.30	

		f_b	f_a	f_p	f_{gk}	f_{par}	Note
Medeiros [16]	Hollow concrete blocks	8.40	4.55	–	–	4.28	–
		8.40	5.89	–	–	4.64	–
		10.80	4.55	–	–	4.83	–
		10.80	5.89	–	–	5.26	–
		14.90	4.55	–	–	4.97	–
		14.90	5.89	–	–	6.52	–

		f_b	f_a	f_p	f_{gk}	f_{par}	Note
Mendes [17]	Hollow clay blocks	22.89	6.47	–	–	–	3-block high prism; full bed mortar joints; fully grouted when grout strength is presented
		22.89	6.68	12.28	49.87	–	
		22.89	7.00	14.66	28.32	–	
		22.89	6.52	17.94	13.94	–	
		22.89	19.86	12.56	–	–	
		22.89	19.09	20.81	49.57	–	
		22.89	19.78	19.53	25.08	–	
		22.89	21.15	21.33	12.37	–	

		f_b	f_a	f_p	f_{gk}	f_{par}	Note
Mohamad [18]	Hollow concrete blocks	10.70	19.90	10.56	–	–	3-block high prism; full bed mortar joints
		10.70	8.63	8.60	–	–	
		10.70	4.20	8.17	–	–	
		10.70	2.28	7.54	–	–	
		15.70	19.90	11.70	–	–	
		15.70	8.63	10.80	–	–	
		15.70	4.20	8.84	–	–	

		f_b	f_a	f_p	f_{gk}	f_{par}	Note
Silva [25]	Hollow concrete blocks	11.8	5	10.1	–	8.56	3-block high prism; full bed mortar joints
		22	5	14.4	–	8.16	

Table 3
Concrete block results (f_{bk} up to 10 MPa)

66 walls	Strength ratio			
	f_{pa}/f_a	f_{pa}/f_{ppa}	f_{pa}/f_p	f_p/f_b
Average	0.51	1.00	0.69	0.80
Standard deviation	0.08	0.12	0.13	0.07
Coefficient of variation	0.16	0.12	0.19	0.09
Source: Cavalheiro & Gomes [14]				

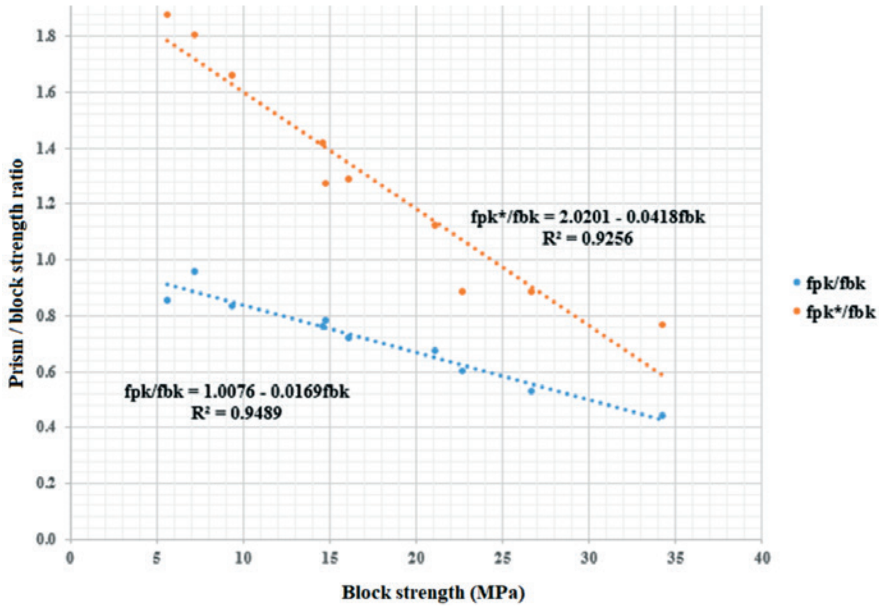


Figure 1
Prism / block strength ratio

Figure 2
Wall before testing

The literature shows that a prism/block strength ratio equal to 0.80 can be assumed for 4-MPa blocks, but that ratio diminishes to 0.5 or less for 30-MPa blocks, these strengths being determined using gross area. In the case of the wall testing, conducted with blocks up to 14 MPa, values above 0.7 for the wall/prism strength ratio are reported. There are no reports of wall testing with block with strengths higher than 14 MPa, except for one case with tests of walls built with 22-MPa block and with a weak mortar, below the recommended strength. Considering the large amount of tall buildings currently being built in Brazil and the that literature showed a lack of research on high-strength concrete blocks. over 14 MPa. the research herein presented is fully justified.

1.1 Justification

The objective of the study presented herein is to assess parameters and characteristics of masonry walls built with high-strength concrete blocks (of 18 to 34 MPa when considering the gross area). The use of these blocks is not common anywhere else in the world. and today Brazil is one of the few countries where there are

Table 4
Grouted walls with mid-height beam results

Nome	Mid-beam	Full or faceshell mortar	Hollow or grouted	f_{bk}	f_a	f_{gk}	# specimens
B1-O-AT-CT	Yes	Full	Hollow	B1	A1	G1	3
B1-O-AP	No	Face	Hollow	B1	A1	–	3
B1-G-AT-CT	Yes	Full	Grouted	B1	A1	G1	3
B2-O-AT-CT	Yes	Full	Hollow	B2	A2	G2	3
B2-O-AP	No	Face	Hollow	B2	A2	–	3
B2-G-AT-CT	Yes	Full	Grouted	B2	A2	G2	3
B3-O-AT-CT	Yes	Full	Hollow	B3	A3	–	3
B3-O-AT	No	Full	Hollow	B3	A3	–	3
B3-G-AP	No	Face	Hollow	B3	A3	–	3
B3-G-AT-CT	Yes	Full	Grouted	B3	A3	G3	3

current applications with these materials, herein that are called high-strength concrete blocks. Even though their use is common in Brazil, the characteristics and some aspects of their behavior are not fully known. Several buildings are constructed with high-strength concrete blocks, but the parameters for assessing their structural walls properties are based on strength ratios obtained from blocks with significant lower strength. Results of tests that allow to correlate the strength of concrete block, prism and wall for the high strength values reported herein are, as far as we know, unique. The research reported herein expands the knowledge and state-of-the-art.

2. Materials and experimental program

Thirty masonry walls. shown in Figure 2, including hollow and grouted walls, walls with or without an intermediate grouted bond beam, and walls fully and face-shell mortar bedding were tested under compression load.
The following elements were tested:

- Axial compression tests of masonry walls with 16, 24 and 30-MPa nominal capacity blocks:
  - Hollow, with an intermediate grouted bond beam at half wall height (bond beam blocks had strength of 6MPa);
  - Hollow, with face-shell mortar bedding;
  - Hollow, with full mortar bedding;
  - Grouted, with an intermediate grouted bond beam at half wall height;
- Axial compression tests of prisms and concrete masonry blocks of 18, 24 and 34 MPa.
- Axial compression tests of mortar and grout specimens.

Table 4 summarizes the characteristics and number of wall tests performed.

2.1 Mortar

Three mortar mixes were used to build the walls. The compressive strength of the mortars was determined according to the resistance of the blocks. Mortar mix proportions were those used by Fortes [21], and, by weight, they were equal to 1:0.42:4.25; 1:0.21:3.40 and 1:0.21:2.98, as shown in Table 5. Figure 3 shows the molding and the mortar specimens.
Portland cement type CP II-Z-32, hydrated lime and sand, were used and their average densities were determined during this research. For every mortar mix proportion, six specimens were molded and tested to control the production process.

2.2 Grout

For the walls and to fill the intermediate bond beam. the admixture "dry d1 c compact", tested by Fortes [21], was used in the grout mix to reduce grout shrinkage and minimize the separation at the block and grout interface. This admixture is a powdered inorganic product, free from chlorides and other harmful components. It is a

Table 5
Mortar characterization

Name	Mix proportion by volume	Mix proportion by weight	w/c	Nominal strength (MPa)
A_1	1:1.0:5.0	1:0.42:4.25	1.05	12
A_2	1:0.5:4.0	1:0.21:3.40	0.86	18
A_3	1:0.5:3.5	1:0.21:2.98	0.77	24

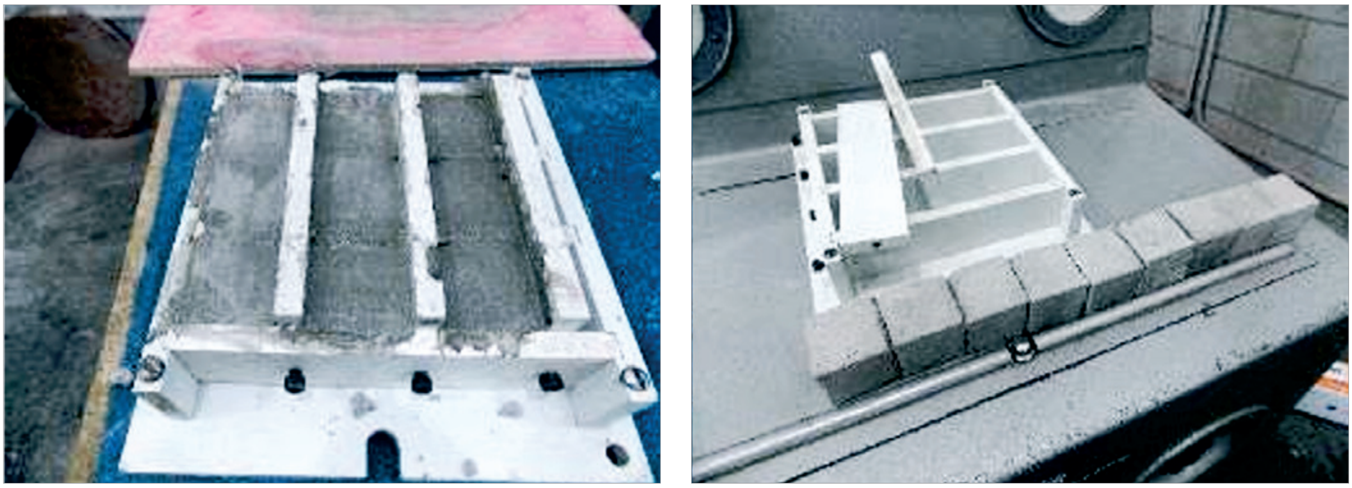


Figure 3
Prismatic mortar molding

Table 6
Grout characterization

Name	Mix proportion by volume	Mix proportion by weight	w/c	Nominal strength (MPa)
G_1	1:0.1 :0.009 :1.6:1.8	1:0.06:0.01:1.60:1.80	0.68	25
G_2	1:0.1:0.009:1.4:1.6	1:0.06:0.01:1.98:2.02	0.58	30
G_3	1:0.1:0.009:1.0:1.3	1:0.06:0.01:1.42:1.64	0.45	40

heat-treated calcium oxide admixture with a selected specific granulometry and with expansive property. The expansive property induces decrease of porosity and reduction of permeability, an increase in compressive strength, and greater adherence between elements.
Three distinct grout strengths were used according to the block type, with mix proportions being those presented in Fortes [21]. The mix proportion, by mass, were 1:0.06:0.01:1.60:1.80; 1:0.06:0.01:1.98:2.02; and 1:0.06:0.01:1.42:1.64 (cement: lime: admixture: sand: gravel); with nominal compression strengths, equal to 25 (G_1). 30 (G_2) and 40 (G_3) MPa. Table 6 gives the denominations for each mix and summarizes, the mass and volume of the mix and the expected strength. Figure 4 shows the grout specimens and walls grouting procedure.

Figure 4
Grout specimens and grouted mid-height bond beam



Figure 5
Standard concrete block, cut half-block; bond-beam block

2.3 Block

High-strength concrete blocks from the same supplier, complying with NBR 6136 [22] standard, of uniform geometry, and of nominal strengths of 16. 24 and 30 MPa. called respectively of B_1. B_2 and B_3 were used. For each block type, twelve blocks were tested to determine their compressive strength. The samples did not have cracks, deformities, or irregularities at the edges. Bond beam blocks, provided by the same manufacturer, but with nominal strength of 6 MPa, were used in the construction of the intermediary bond beam regardless of the type of blocks used on the walls. Whole blocks were cut to be used as half blocks. Figure 5 shows the whole block, the block cut in half, and the bond-beam block.

2.4 Construction of the prisms and walls

Hollow and grouted prisms were built as specified by the Brazilian standard ABNT NBR 15961-2 [23]. The blocks, mortar and grout were combined for assembling prisms. Each prism was built with two stacked blocks and one mortar joint. Figure 6 shows one hollow and one grouted prism. The construction of the prisms closely followed the ABNT NBR 15961-2[23] requirements, and the same mason built all specimens.

The prisms were grouted after approximately 24 hours of their construction. Before grouting, mortar protrusions and droppings were removed from the prisms cells and each prism was wet. The grout was poured in two layers and consolidated with 12 rodding per layer as per ABNT NBR 15961-2[23]. After completing the grouting, the grout surface was leveled and smoothed with a trowel and immediately covered with an impermeable plastic film.

The construction of the walls followed the same procedure. Figure 7 to Figure 13 show the construction procedure sequence.

2.5 Blocks, mortar and grout

For each block strength, twelve blocks were tested as per ABNT NBR 12118 [6]. Block testing was done the same day of walls testing. The average compression strength of mortars was specified as 70% of the block compressive strength as peer ABNT NBR 15961-2 [23]. Mix proportions were the same as those reported in Fortes [21]. Six grout specimens and six mortar specimens were molded and tested for each group of walls. Grout testing followed ABNT NBR 5739 [24] specifications. The top and bottom surfaces of the specimens were ground prior to testing at 28-day age.

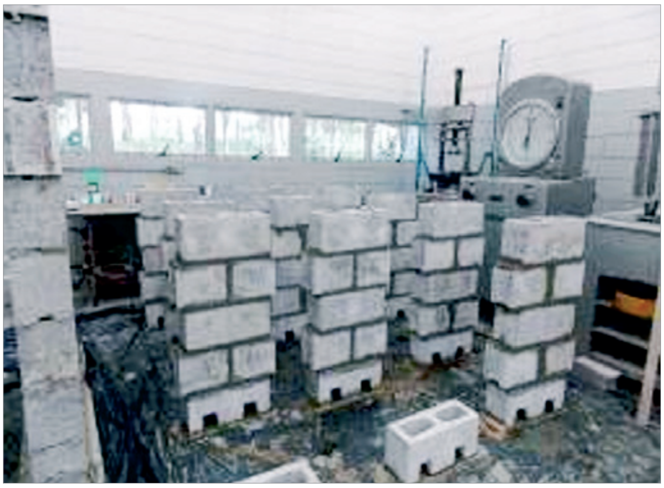



Figure 6
Hollow and grouted prism

Figure 7
Corner gauge to aid wall construction

Figure 8
Wall construction sequence

Figure 9
Hollow wall construction

Figure 10
Mid-height bond beam laying

2.6 Prism

Following the specifications of ABNT NBR 15961-2 [23], 18 hollow prisms and 18 grouted prisms were constructed and tested. The prisms were capped with cement paste as per ABNT NBR 15961-2[23]. Tests were conducted using a 2000-kN testing machine and the prisms were loaded at 0.05 ± 0.01 MPa per second.

2.7 Wall configuration

The wall testing was divided into three groups according to the blocks strength. In the first group, twelve walls were built with 30-MPa nominal strength blocks (B_3): three hollow walls with full bed mortar joints, three hollow walls with face-shell mortar joints, three walls with full bed mortar joints with a mid-height bond beam, and three walls fully grouted and with a mid-height bond beam.
In the second group, nine walls were built with 24-MPa nominal strength blocks (B_2): three hollow walls with full bed mortar joints and with a mid-height bond beam, three hollow walls with face-shell mortar joints, and three walls fully grouted with a mid-height bond beam.
In the third group, nine walls were built with 16-MPa nominal strength blocks (B_1): three hollow walls with full bed mortar joints with a mid-height bond beam, three hollow walls with face-shell mortar joints, and three walls fully grouted with a mid-height bond beam.
The walls were unreinforced except that a horizontal steel 10-mm rebar was used in the mid-height bond beams. The ungrouted walls were 220-cm high and 120-cm wide while the grouted walls were 220-cm high and 80-cm wide. due to load capacity of the testing machine. Regular cement-lime-sand mortar was used in the walls and prisms construction. The mortar was designed to have a 28-day compressive strength equal to 70% of the strength of the units, considering the gross area.
The grout used was designed to have a compressive strength at 28 days equal to 40 (G_3), 30 (G_2), and 25 (G_1) MPa, respectively

Figure 11
Wall with mid-height bond beam

Figure 12
Full bed mortar joint wall construction

Figure 13
Face-shell bed mortar joint wall construction

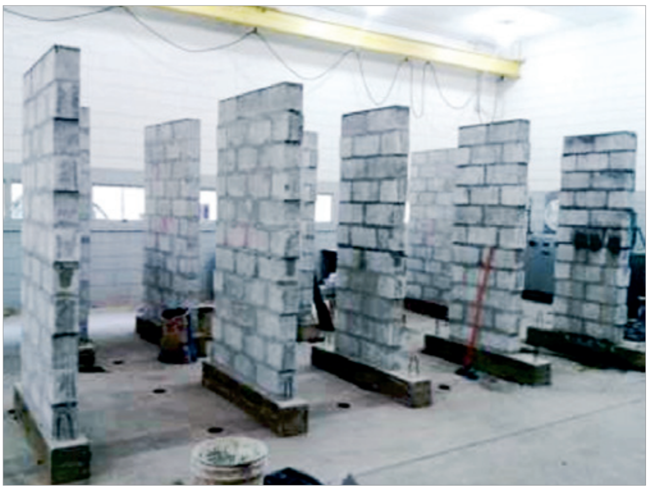


Figure 14
Walls before testing

Figure 15
Walls capped with mortar

for the walls built with 30-MPa (B_3), 24-MPa (B_2), and 16-MPa(B_1) units. To reduce grout shrinkage and possible debounding at the grout and block shell interface, a shrinkage compensating admixture was added to the grout mix. For each wall group, twelve concrete blocks, six hollow prisms, and six grouted prisms were tested at the same time of the walls testing.

To differentiate the walls, a system with the following letter were used: Par-B-O-G-AT-AP-CT. The letters "Par" refers to the wall itself, the letter B refers to the type of block used, and the letters O and G refer to hollow wall or grouted wall, respectively. The letters AT and AP are related to full bed mortar joint and face-shell mortar joint, respectively. Finally, the letters CT refer to the presence of the mid-height bond beam. To help better observe crack formation and propagation, all walls were white washed. Figure 14 shows some walls ready to be tested.

2.8 Capping

The walls were capped with mortar as shown in Figure 15 as per NBR 8949 [5] specifications. The mortar mix proportioning used was 1:0.5:2.0 (cement. lime and sand) designed to obtain a mortar strength higher than that of the blocks. The blocks were capped with a hardboard as described in Fortes [21] and the prisms were capped with cement paste.

2.9 Test configuration and instrumentation

A hydraulic 500-ton capacity (5000 kN) analog compression machine was used for testing Groups 1 and 2 walls, as shown in Figure 16.

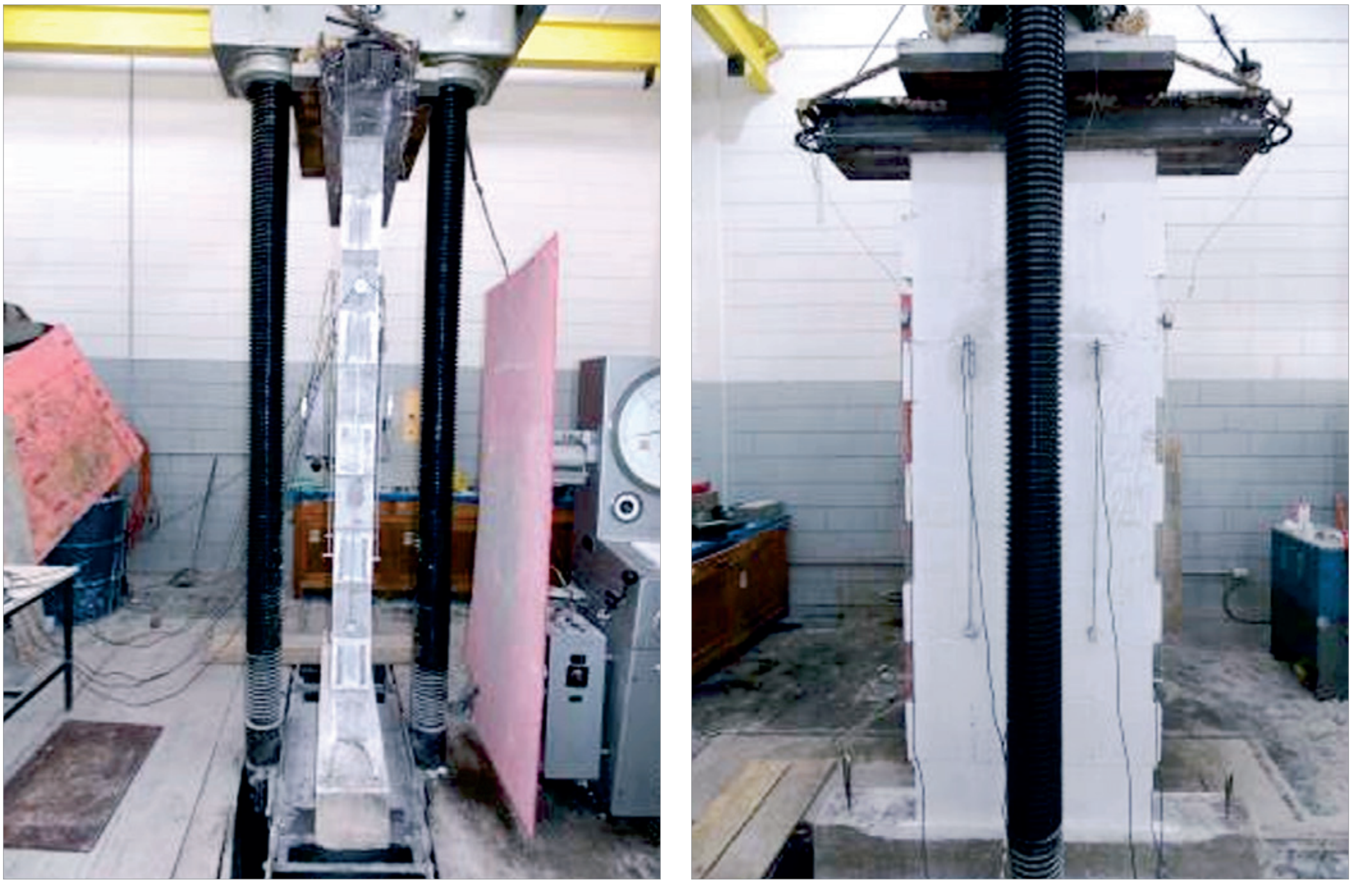


Figure 16
Walls testing. Group 1 e 2

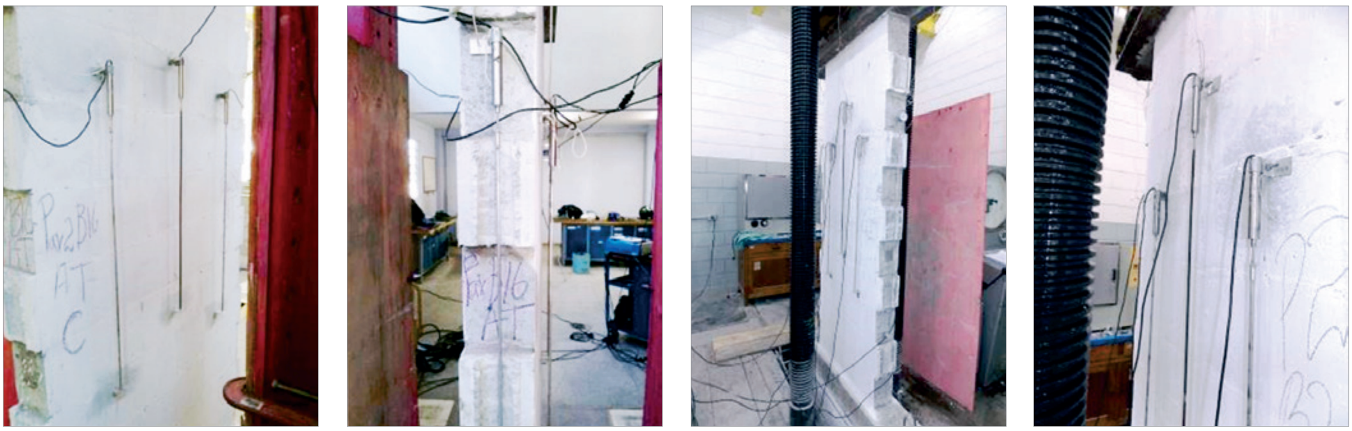
The Group 3 walls were tested using a self-reacting steel frame as shown in Figure 17. During the testing, the vertical displacements of both sides (Faces 1 and 2) of the walls were measured using 25-mm stroke displacement transducers (LVDTs) as shown in Figure 17. The precision of the LVDTs used was 0.000010mm. The vertical displacements of the sides of the wall were monitored continuously with wireless dial gauges, also shown in Figure 16.

To account for any accidental eccentricity of the loading during the test, measurements were taken on the front and on the back of the walls, as shown in Figure 18, and the mean value of the two measurements was used in the analysis. Before each test, the wall being tested was centered in test position and aligned vertically with the help of a laser level and a plumb line. The wall was then loaded and unloaded twice up to 5% of the maximum expected load to lock the wall in place and check the instrumentation readings. During each test, the vertical load was gradually applied with a rate of about 10 kN/s up to the failure.

2.10 Experimental procedure

Each test started by loading and unloading the wall twice up to

Figure 17
Walls testing. Group 3



A Group 3 **B** Group 1 and 2

Figure 18
Wall instrumentation

Table 7
Materials characterization methods

Material	Method	Test
Block	ABNT NBR 6136/2014	Specification
Block	ABNT NBR 12118/2013	Full absorption
Block	ABNT NBR 12118/2013	Compressive strength
Wall	ABNT NBR 8949/1985	Compressive strength
Wall mortar prism	ABNT NBR 15961-2/2011	Compressive strength
Grout	ABNT NBR 5738	Molding
Grout	ABNT NBR 5739	Compressive strength
Grout	ABNT NBR 7211	Specifications
Mortar	ABNT NBR 13279	Flexural and compression strength

50% of the maximum expected load. Then, the load was applied in increments equal to 10% of the expected failure load up to 50% of this value, waiting 5 minutes between each increment to allow obtaining and plotting of load vs displacement and stress vs strain curves. After this load, the axial load was gradually monotonically increased until failure of the wall. The load was read from the analog gauge at the compression machine used for testing groups 1 and 2 walls and directly from the data acquisition system for testing group 3 walls. Readings of the LVDTs were taken and recorded by a computer controlled data acquisition system. Data recording occur every second. A wall was considered to fail when vertical cracks appeared and the applied load started to decrease or when the wall exhibited large lateral deformation with a vertical load drop. The procedure adopted for each test is summarized in Table 7.

3. Results and discussion

A detailed description of the block, mortar, block, prism, and wall testing results are discussed below.

3.1 Grout

Figure 19 shows some ground grout specimens and one specimen after testing. The grout strength results according to their use in the walls and horizontal bond beams for Groups 1, 2 and 3 are presented in Table 8.

3.2 Mortar

Figure 20 shows some mortar specimens and the testing assemblage. Table 9 presents the average compressive strength results for the mortar.

Figure 19
Grout specimens and testing

Table 8
Grout compressive strength

Name	Average compressive strength (MPa)	Standard deviation (MPa)	Coefficient of variation (%)
G_1	31.3	2.35	7.5
G_2	34.9	3.92	11.2
G_3	42.4	2.68	6.3

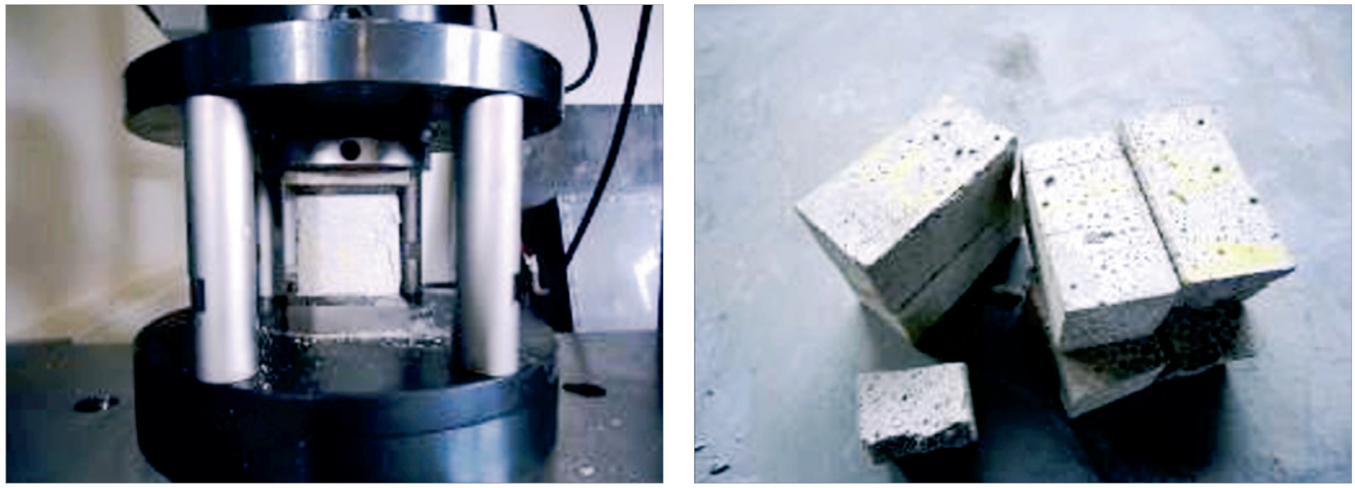


Figure 20
Mortar specimens and testing

Table 9
Mortar compressive strength

Name	Average compressive strength (MPa)	Standard deviation (MPa)	Coefficient of variation (%)
A_1	13.4	0.69	5.3
A_2	21.8	0.65	3.0
A_3	26.9	0.80	3.0

3.3 Block

Figure 21 shows a block being tested and the block typical mode of failure. Table 10 presents the average compressive strength and the coefficients of variation results.

3.4 Prism

Figure 22 shows some hollow and grouted prisms, respectively after testing. Probably due to the high strength blocks, hollow prisms failure was not as the usual failure reported in the literature. The hollow prisms failed due to the development of vertical cracks along the loading direction followed by separation of the lateral walls of the blocks or by crushing of the mortar joint followed by separation of the blocks faceshell.
Results on the hollow and grouted prisms compressive strength are presented in Table 11 and Table 12, respectively. The tables also provide the standard deviations and coefficients of variation.
The results presented in Table 13, Table 14, and Table 15 are f_{pa}, Δ_u, $\Delta_{u50\%}$, ε_u, and $\varepsilon_{50\%}$, representing the wall average compressive strength, the wall average maximum vertical shortening, the average vertical shortening at 50% of the maximum stress, the ultimate strain and the strain at 50% of the maximum stress, respectively. The strength results, the stress-strain curves for masonry in compression, the stress-strain relationship at the ultimate load, the effect of the mid-height grouted bond beam,

Figure 21
Block specimens and testing

Figure 22
Hollow and grouted prism testing

Table 10
Block compressive strength

Name	Average compressive strength (MPa)	Standard deviation (MPa)	Coefficient of variation (%)
B_1	18.7	1.1	5.9
B_2	27.3	3.0	11.0
B_3	34.5	0.4	1.0

Table 11
Hollow prism compressive strength

Name	Two-block hollow prism assemblage		
	B_1 - A_1	B_2 - A_2	B_3 - A_3
Average compressive strength (MPa)	10.0	13.3	16.9
Standard deviation (MPa)	0.90	1.26	1.45
Coefficient of variation (%)	8.96	9.46	8.6

Table 12
Grouted prism compressive strength

Result	Two-block hollow prism assemblage		
	B_1 - A_1 - G_1	B_2 – A_2 – G_2	B_3 – A_3 – G_3
Average compressive strength (MPa)	22.3	29.0	32.2
Standard deviation (MPa)	1.1	2.8	1.2
Coefficient of variation (%)	4.75	9.6	3.7

the mortar laying type, the grouting effect on the wall compression capacity, and the failure mode are discussed in the subsequent sections.

3.5 Group 1-Walls with 30-MPa high-strength concrete blocks (B_3)

The results of the tests for Group 1 walls are presented in table 13.

For the hollow walls with full bed mortar joints, the average failure stress was equal to 11.43 MPa. For hollow walls with face-shell mortar joints, the average failure stress was equal to 11.82 MPa. This result is unexpected, the walls with full mortar bed joints were expected to have greater strength than those constructed with face-shell mortar joints. The difference in the average wall strength is only 3.3%. Applying a t-test on the results, a p-value equal to 22.5% was determined, indicating that, statistically, the averages

Table 13
Walls with block B_3 results

Wall name	f_{pa} (MPa)	$\Delta_{u50\%}$ (mm)	$\varepsilon_{u50\%}$ (mm/mm)
Par-B3-O-AT			
Average	11.4	0.49	0.0006
Coefficient of variation	4.45	28.90	27.98
Par-B3-O-AP			
Average	11.8	0.48	0.0006
Coefficient of variation	4.89	17.69	19.16
Par-B3-O-AT-CT			
Average	11.1	0.46	0.0005
Coefficient of variation	3.88	7.93	5.09
Par-B3-G-AT-CT			
Average	19.0	0.49	0.0006
Coefficient of variation	14.03	24.44	19.58

Table 14
Walls with block B_2 results

Wall name	f_{pa} (MPa)	Δ_u (mm/mm)	ε_u (mm/mm)
Par-B2-O-AP			
Average	10.7	0.78	0.0008
Coefficient of variation	0.9	15.68	19.60
Par-B2-O-AT-CT			
Average	11.8	0.97	0.0012
Coefficient of variation	3.3	22.22	19.64
Par-B2-G-AT-CT			
Average	19.8	1.20	0.0013
Coefficient of variation	10.7	18.65	16.31

Table 15
Walls with block B_1 results

Wall name	f_{pa} (MPa)	Δ_u (mm/mm)	ε_u (mm/mm)
Par-B1-O-AT-CT			
Average	7.8	0.72	0.0008
Coefficient of variation	6.19	23.45	19.60
Par-B1-O-AP			
Average	7.8	0.98	0.0011
Coefficient of variation	8.35	21.75	23.47
Par-B1-G-AT-CT			
Average	11.5	1.06	0.0012
Coefficient of variation	20.22	24.88	27.39

are equal to a significance level of 95%. The average shortenings, $\Delta_{u50\%}$, of all walls of Group 1 varies from 0.46 to 0.49 mm. Clearly, the shortening of the walls is indifferent to the variables considered. The average strains, $\Delta_{u50\%}$, are also similar, with values of 0.0006, 0.0006, 0.0005, and 0.0006, respectively for the full mortar bed joint hollow walls, face-shell mortar joint hollow walls, full mortar bed joint hollow walls with a mid-height bond beam, and grouted walls with a mid-height bond beam.

The Brazilian concrete block structural masonry standard, NBR 15961-2 [23], specify that the compressive strength of walls constructed with face-shell mortar joints should be considered as 20% lower than that of a wall with full bed mortar joint. The results presented here do not corroborate the reduction indicated by the standard. The geometry of the blocks may explain the similar results obtained herein. Due to the running bond pattern used, the position and thickness of the block web do not align vertically with that of the block web placed on its top. Therefore, laying blocks with full bed mortar joints may have been inefficient because there is no vertical alignment of the webs.

The average compressive strengths were 19.0 and 11.1 MPa for the grouted and ungrouted walls (both with a mid-height bond beam), respectively, representing an increase in strength for the grouted walls of approximately 66%.

3.6 Group 2-Walls with high-strength concrete blocks of 24MPa (B_2)

Group 2 test results are shown in table 14

For hollow walls with full bed mortar joints and with a mid-height bond beam, the average strength was 11.77 MPa while the hollow walls with face-shell mortar joints without a mid-height bond beam, the average strength was 10.69 MPa. The capacity of the hollow walls with full bed mortar joints with a mid-height beam is approximately 10% higher than that of the walls with face-shell mortar joints without a mid-height bond beam. The t-test indicates a p-value of 2.7%. indicating that there is difference between the average strengths. The existence of the mid-height bond beam may have contributed to better stress distribution the wall critical region and contributed to the strength increase for the full bed mortar joint walls. However, the walls in Group 3 did not experienced the same kind of results, which invalidates this assumption. The wall shortenings at the failure loads, Δ_u, were, respectively 0.97 0.78 and 1.20 mm for the hollow walls with full bed mortar joints with a mid-height bond beam, for the hollow walls with face-shell mortar joints, and for the grouted walls with a mid-height bond beam. There is therefore a variability in the walls shortening. The average strains at ultimate loads, ε_{u}, were 0.0012, 0.0013, and 0.0008. mm/mm, respectively.

When comparing the hollow full bed mortar joint walls with the grouted wall, both with a mid-height bond beam, the average strength of 11.77 MPa obtained for the hollow walls case is increased to 19.79 MPa for the fully grouted walls. The average compressive capacity of the grouted wall is approximately 68% higher than that of the hollow walls.

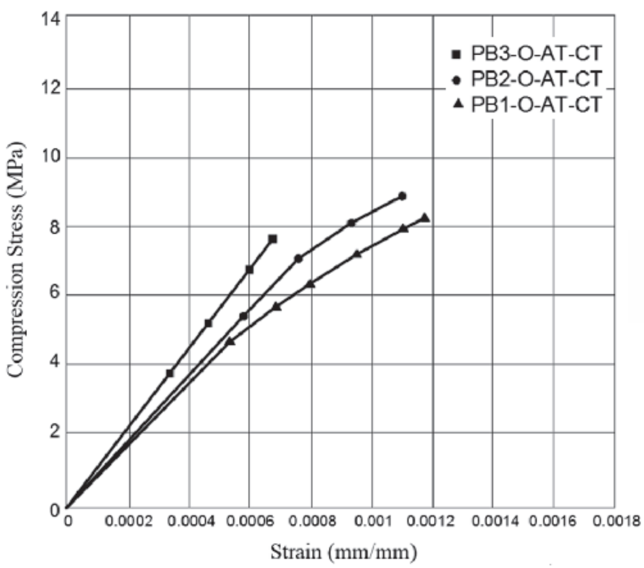


Figure 23
Walls PB-O-AT-CT stress vs strain results

3.7 Group 3-Walls with high-strength concrete blocks of 16MPa (B_1)

Group 2 test results are presented in table 15.

The average compressive strength was 7.8 MPa for both hollow face-shell mortar joint walls without a mid-height bond beam and the hollow full -bed mortar joint walls with a mid-height bond beam. Therefore, there was no significant difference between the type of bedding and the presence or not of a mid-height bond beam on the wall strength.

The wall vertical shortening, Δ_u, was respectively 1.06, 0.98, and 0.72 for hollow walls with full bed mortar joints and with a mid-height bond beam, for hollow walls with face-shell mortar joints, and for the grouted walls with a mid-height bond beam. As happened with the walls of the Group 2, some difference in the vertical shortening of walls is observed. The average ultimate shortening, ε_u, were 0.0008, 0.0011, and 0.0012, respectively.

When comparing the hollow full bed mortar joint walls with the grouted wall, both with a mid-height bond beam, the average strength of 7.78 MPa obtained for the hollow walls case is increased 11.53 MPa for fully grouted walls. The average compressive capacity of the grouted wall is approximately 48% higher than that of the hollow walls.

3.8 Stress-strain relationship

To assess the effect of the mid-height bond beam, of the grouting or non-grouting, and of full or face-shell bed mortar joints on the stress-strain relationship of high-strength masonry in compression, the stress-strain curves were plotted. The average ultimate strain for the walls of groups 2 and 3 were calculated using the vertical displacements within a calibrated measurement length on the walls faces. The ultimate strain for walls of groups 2 and 3 and the 50% of ultimate load strain for walls the Group 1 are presented in Table 13, Table 14, and Table 15, respectively. The average ultimate strain for walls of groups 2 and 3 ranged from 0.001 to

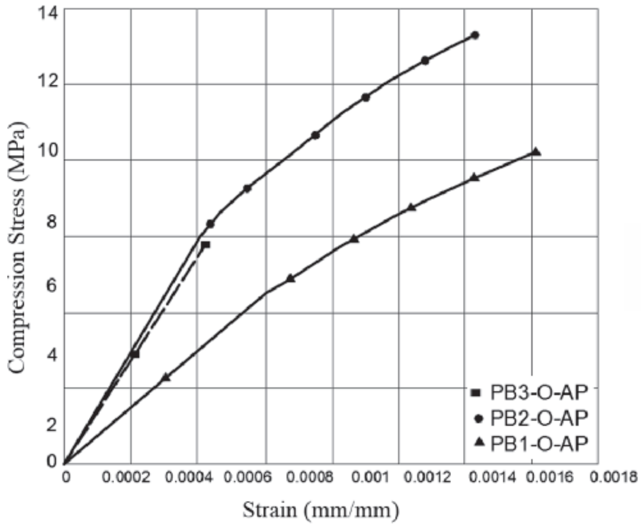


Figure 24
Walls PB-O-AP stress vs strain results

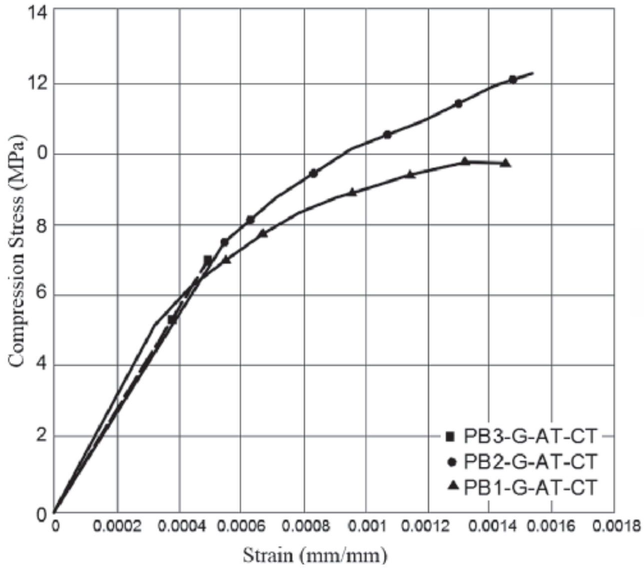


Figure 25
Walls PB- G-AT-CT stress vs strain results

0.0015. For the walls of Group 1, the strains were measured up to 50% of the failure load and ranged from 0.0005 to 0.0006.
The stress-strain curves for the walls with the same characteristics of each group are presented in Figure 23, Figure 24, and Figure 25. The compression stresses were calculated considering the wall gross area. For the walls PB3-O-AT-CT, PB3-O-AP, and PB3-G-AT-CT, the curves are up to 50% of failure load only while for the remaining walls the curves were obtained up to failure. The results presented in Figure 23, Figure 24, and Figure 25 show a brittle behavior for all walls tested.

For the full bed mortar joint hollow walls with a mid-height bond beam, the stress-strain curve shown in Figure 23 remained approximately linear up to 75% of failure load. For the face-shell bed mortar joint hollow walls without a mid-height bond beam, the stress-strain curve shown in Figure 24 remained approximately linear up to 70% of failure load. In the case of the grouted walls with and without a mid-height bond beam, the stress-strain curve shown in Figure 25 remained approximately linear up to 60% of

Table 16
Wall elastic modulus results

Wall name	Grupo 1 Par - B_3	Group 2 Par - B_2	Group 3 Par - B_1
Par-O-AT			
E (average) (MPa)	20631	–	–
E_a (800 f_p) (MPa)	13520	–	–
Coefficient of variation	26.05%	–	–
Par-O-AP			
E (average) (MPa)	20283	19658	14327
E_A (800 f_p) (MPa)	13520	10640	8000
Coefficient of variation	8.31%	8.07%	4.76%
Par-O-AT-CT			
E (average) (MPa)	23438	20473	15781
E_a (800 f_p) (MPa)	13520	10640	8000
Coefficient of variation	13.96%	6.71%	4.68%
Par-G-AT-CT			
E (average) (MPa)	24219	23927	15394
E_a (800 f_p) (MPa)	25760	23200	17840
Coefficient of variation	17.14%	34.17%	-12.97%

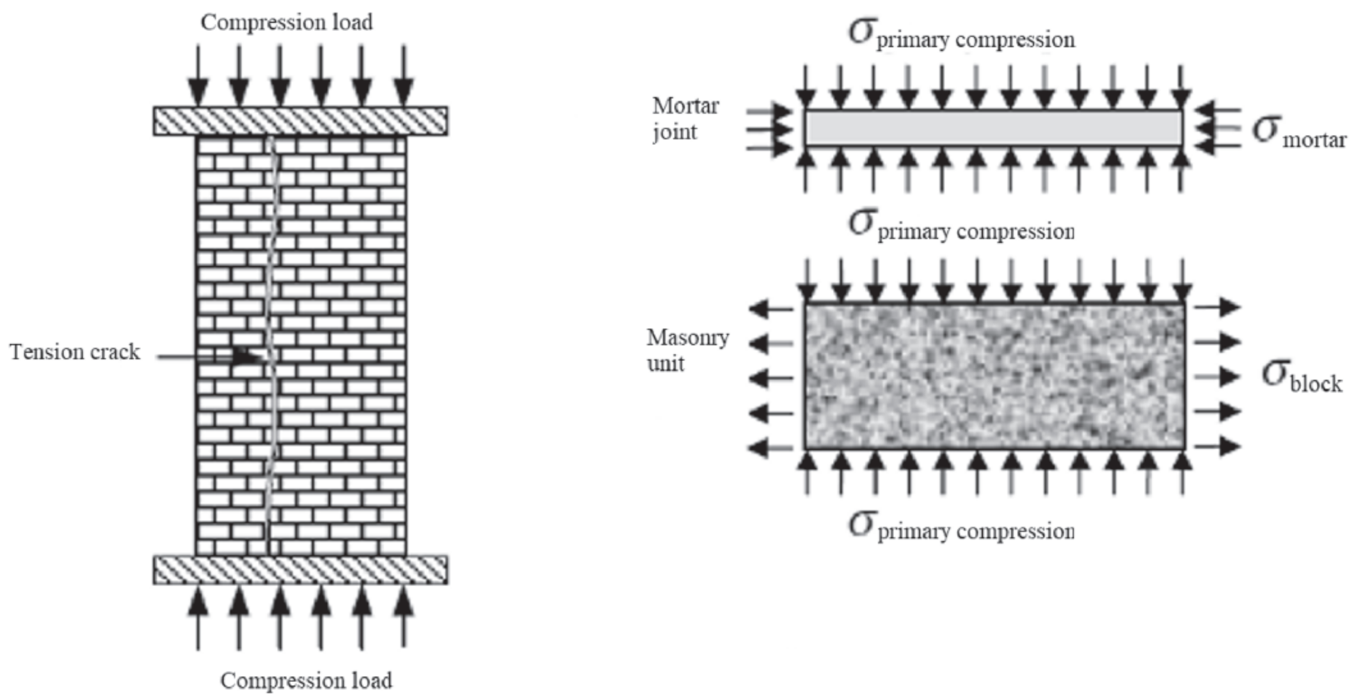


Figure 26
Wall typical failure mode (Hendry et al 2004)

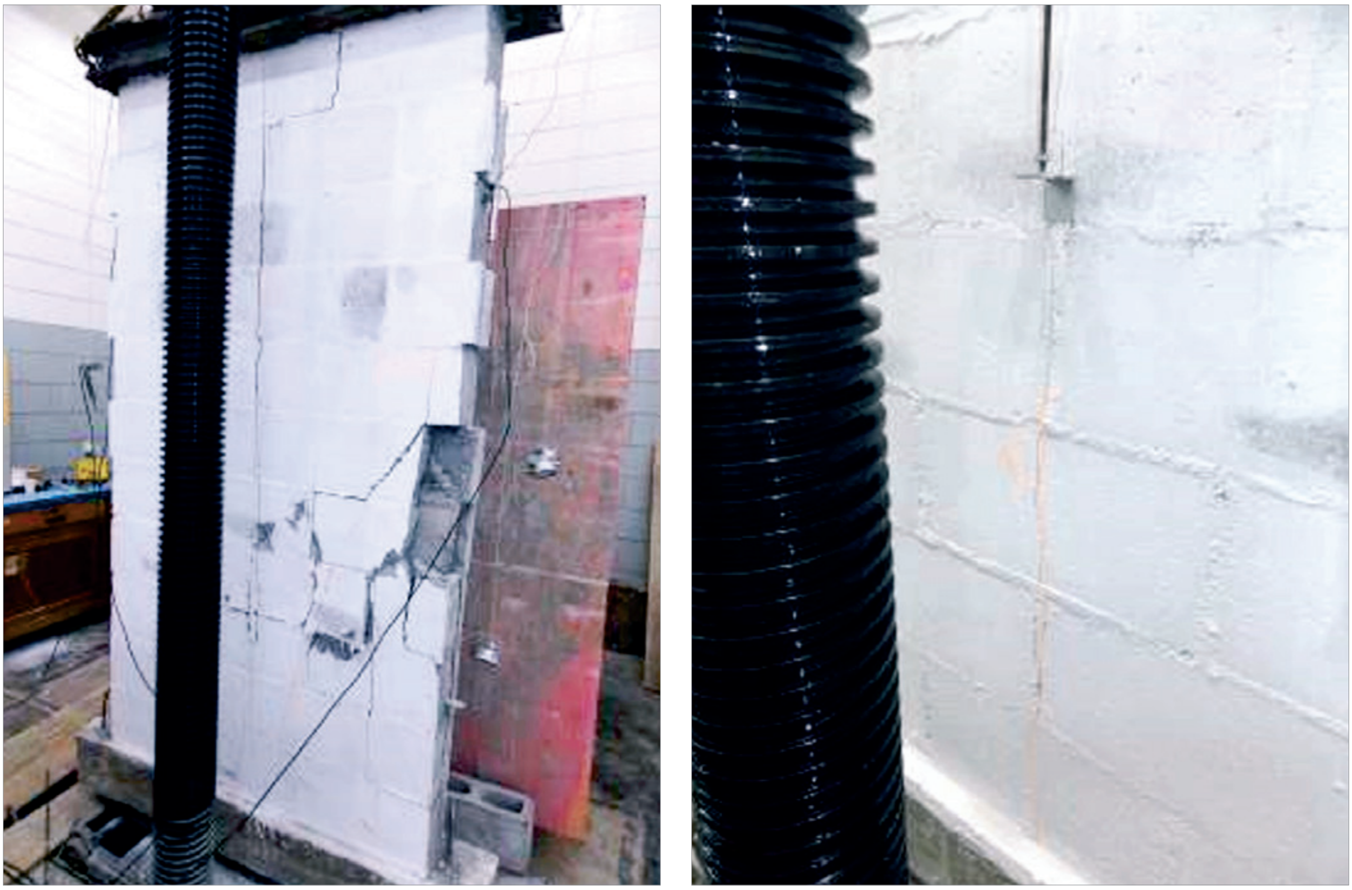


Figure 27
Wall with mid-height bond beam failure

failure load. Results indicate that each variable has a small effect on the strain-strain curve shape.
A comparison of stress-strain curve between the hollow or grouted walls indicates that, overall, the shape of the stress-strain curve is similar, with that of the grouted wall being slightly more nonlinear.

3.9 Modulus of elasticity of masonry

The modulus of elasticity was considering as the secant line of the strain-strain curve between 5 and 30% of the failure stress. The modulus of elasticity can also be calculated as a function of the compressive strength of masonry, and these estimates are usually presented in standard codes. The ABNT NBR 15961-1[4] specifies that the modulus of elasticity of concrete block masonry can be estimated as 800 f_{pk}.
Table 16 shows the average results of the modulus of elasticity obtained for the walls using the linear part of the stress-strain curves. On average, the results of the secant modulus of Group 1 walls ranged from 20.2 GPa for the hollow walls to 24.2 GPa for the grouted walls. The results of the secant modulus of elasticity of Group 2 walls ranged from 19.6 GPa for the hollow walls to 23.9 GPa for the grouted walls. Group 3 walls results ranged from 14.3 GPa for the hollow walls to 15.4 GPa for the grouted walls.
Considering the standard ABNT NBR 15961-2 [23], which specifies the modulus of elasticity of masonry equal to 800 f_{pk}, limited to 16 GPa, the calculated values are far superior for all hollow walls results, being considerably larger than 16 GPa, between 50% to almost 100% higher. In the case for the grouted walls, the calculated values are close to those estimated with E = 800f_p, with differences between -14% and +6%; that is, between 688 f_p the 848 f_p.

3.10 Failure mode

Failure mode of masonry in compression is usually caused by a tension crack that propagates through the blocks and mortar in the direction of the applied force, as shown in Figure 26. This crack is caused by secondary tension stress resulting from the deformation of block and confined mortar in the masonry joints (Hendry et al. [2]). The tensile stresses that induce the cracks are developed in the mortar-block interfaces due to restricted deformation of the mortar. In most cases, the masonry strength is considerably smaller than the block strength, which, however, is considerably higher than that of the mortar. The apparent increase in the mortar strength is due to the biaxial or tri-axial stress state imposed to the mortar when it is loaded in conjunction with the blocks.
The observed failure mode in typical in compression and was started by vertical cracks in the longitudinal and transverse faces of the walls as shown in Figure 27 and Figure 31. The verticals cracks

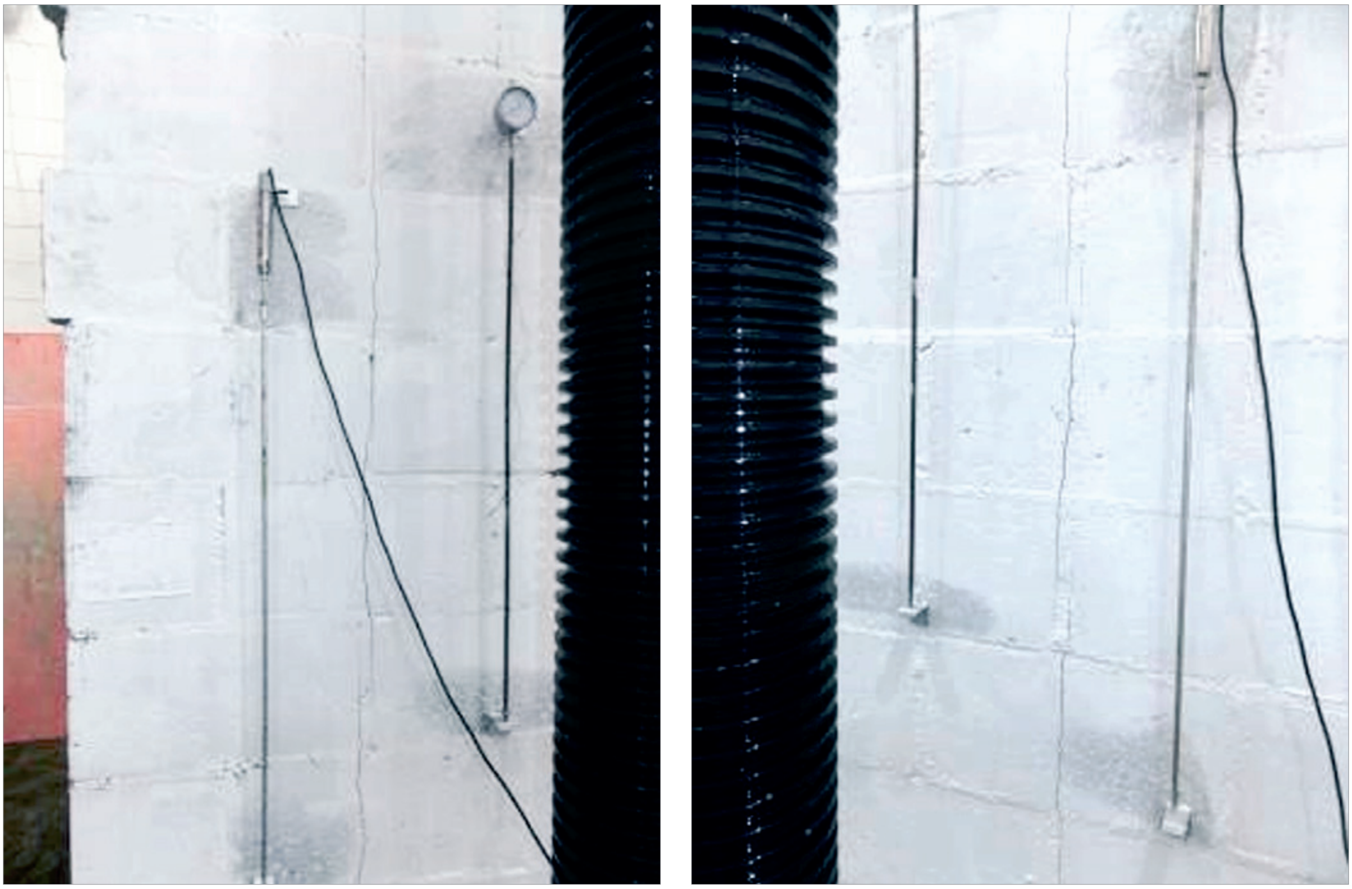


Figure 28
Hollow wall failure

began because of tensile cracking, with some evidence of crushing of mortar, as shown in Figure 29. In some cases, as shown in Figure 30, the development of cracks happened right in the middle of the walls and sometimes a little away from the center.

For the grouted walls with a mid-height bond beam, the failure mode was characterized by vertical separation cracks of the blocks webs, as shown in the Figure 31. For Groups 1 and 2 walls, the first crack was observed at about 60% of the ultimate load. For Group 3 walls, cracking started at approximately 75% of the failure load. Grouted walls with a mid-height bond beam presented an atypical type of rupture when compared to the wall without the mid-bond beam, as shown in Figure 31. The failure load, However, was similar to that of walls without the mid height bond beam.

3.11 Wall/prism/block strength ratio

The Brazilian standard allows the strength of the wall, f_k, to be equal to 70% of the strength of the prism, f_{pk}. Full-scale walls typically have lower strength than that of the prism and the prism strength is inferior to that of the block strength due to the slenderness effect and possibility of non-uniform load distribution. Fortes [21] noted in his research with several block strengths a decrease in the prism strength as the block strength increases and calculated values of efficiency ranging from 0.8 to 0.5 for 6.0 to 34.0 MPa blocks. Table 17 summarizes the compressive strength of blocks, mortar, prisms, and grouted and hollow walls. The strength efficiencies, here defined as the ratio between the compressive strength of the walls to the compressive strength of the prisms, are also presented.

For Group 1 walls, the ratio of the compressive strength of the hollow walls with full bed mortar joints to the compressive strength of hollow prisms (also with full bed mortar joints) is 0.7. When the mid height bond beam is included in the wall, the compressive strength ratio remains equal to 0.7. The ratio between the average compressive strength of grouted walls with a mid-height bond beam to the grouted prism strength, however, is equal to 0.6.

For the walls of Group 2, the ratio of compressive strength of hollow full bed mortar joint walls with a mid-height bond beam and the hollow prisms is 0.9 while that for the hollow walls with face-shell mortar joints and without a mid-height bond beam is 0.8. For the grouted walls with a mid-height bond beam, the ratio to the grouted prism is 0.7.

For the walls of the 3 Group, the ratio of the compressive strength of face-shell mortar joint hollow walls to the compressive strength of hollow prisms is 0.8. The same ratio value was obtained for the full bed mortar joint hollow walls built with the mid height bond beam. In the case of the grouted wall built with the mid height bond beam, the strength ratio to the grouted prism is only 0.5.

Therefore, the results indicate that it is safe to use the conventional wall to prism strength ratio equal to 0.7 for hollow walls even for high-strength blocks. In the case of grouted walls, a wall to prism strength ratio equal to 0.5 is recommended.

An Anova on the results for the walls built with the same B_3 block type was conducted. The test aimed to verify if there is significant difference in the compressive strength when considering:

- Hollow walls built with concrete blocks with full bed mortar joints and with a mid-height bond beam: PB3-O-AT-CT;
- Hollow walls built with concrete blocks with full bed mortar joints and without a mid-height bond beam: PB3-O-AT;

Figure 29
Wall vertical cracking and failure

Figure 30
Wall vertical cracking and failure Wall vertical cracking Group 1 e 2

- Hollow walls built with concrete blocks with face-shell mortar joints and without a mid-height bond beam: PB3-O-AP.

Three specimens were tested into axial compressive loading for each type of test. Based on the results of the ANOVA test, varying the type of mortar bedding and the presence of a mid-height bond beam, it was determined that F_0 = < 2.34 and $F_{\alpha, \upsilon1, \upsilon2}$ = 5.14. So, there is no difference in the average result among PB3-O-AT-CT, PB3-O-AT, and PB3-O-AP.

For the walls built with blocks B_2 and B_1. t-test was applied to determine whether there is a difference in the wall compressive strength average result at a level of significance of 5% and considering the use or not of the mid height bond beam and the type of mortar bedding. The comparison was conducted between:

- Hollow walls built with concrete blocks with full bed mortar joints and with a mid-height bond beam: PB1-O-AT-CT;
- Hollow walls built with concrete blocks with full bed mortar joint and without a mid-height bond beam: PB1-O-AT.

Three specimens were tested to the axial compressive strength for each type of walls. Based on the t-test results, it was determined that $t_{4; 5\%}$ = 2.132 > t = 0.049. Therefore, there is no significant difference between the average compression strength between PB1-O-AT-CT and PB1-O-AT.

The same analysis, t-test, was performed for the walls built with blocks B2. Based on the t-test results it was determined that $t_{4; 5\%}$ = 2.132 < t = 6.67. So, there is also no difference between the mean average compressive strength of PB2-the-AT-CT and PB2-the-AP. Therefore, the type of mortar bedding and the presence or not of a mid-height bond beam did not affected the wall strength.

Table 17
Testing strength and strength ratio summary

f_b (MPa)	f_p (MPa)		Wall	Hollow wall		Grouted wall		Grouted/ hollow
	Hollow	Grouted		f (MPa)	f/f_p	f (MPa)	f/f_p	
18.7	10	22.3	B1-O/G-AT-CT	7.8	0.8	11.53	0.5	1.5
			B1-O-AP	7.8	0.8	-	-	-
27.3	13.3	29	B2-O/G-AT-CT	10.7	0.8	19.79	0.7	1.9
			B2-O-AP	11.8	0.9	-	-	-
34.5	16.9	32.2	B3-O/G-AT-CT	11.4	0.7	18.98	0.6	1.7
			B3-O-AT	11.8	0.7	-	-	-
			B3-G-AP	11.1	0.7	-	-	-

Figure 31
Grouted wall failure

4. Conclusions

This paper presented a comprehensive experimental program with the objective of assessing the compressive behavior of masonry with high-strength concrete block in ungrouted and grouted walls. Nominal resistance of blocks was 16, 24, and 30 MPa. The grout was produced with shrinkage compensating admixture. Walls compressive strength was evaluated considering the compressive strength of blocks, the mortar bedding type, and the use of not of a mid-height grouted bond beam.

The main conclusions of the research presented here are:

- There was no statistically significant difference between the average compressive strength for the hollow walls with full bed or face-shell bed mortar joints.
- There was no statistically significant difference between the average compressive strength for the hollow walls with full bed or face-shell bed mortar joints with and without a grouted mid-height bond beam;
- All the hollow walls presented wall to prism strength ratio of 0.7;
- All the grouted walls showed an increase in compressive strength of at least 50% compared to hollow walls;
- The stress-strain graphs show brittle fracture of all walls with strain at failure between 0.10 and 0.15% (axial compression);
- The results of the modulus of elasticity were all greater than 800 f_p for hollow walls and between 688 and 848 f_p for grouted walls. Several results were above the limit of 16 GPa specified in ABNT NBR 15961-1.

From these observations, for masonry with blocks of strength greater than 16 MPa the following is applicable:

- The value of the wall to prism strength ratio equal to 0.7 can be adopted for high-strength concrete blocks for non-grouted hollow walls;
- For fully grouted walls the designer can consider an increase of 50% in wall strength when compared to the ungrouted wall strength;
- Considering the block geometry used, the use of mortar over the block web shell is not efficient. The recommendation is to consider mortar only on the face shell, both in the design as in the construction. Other block geometries, with vertical alignment of block webs, can produce different results than those reported herein;
- The placement of a mid-height bond block beam in the walls leads to no decreased in compressive strength;
- The value of the modulus of elasticity specified in the Brazilian standards equal to 800 f_{pk} can be benchmarked to ungrouted hollow walls but a lower value was verified for grouted walls. This is a preliminary result that needs further verification is recommended to adopt the value of E = 600 to 650 f_{pk} for

high-strength concrete blocks masonry (greater than 16 MPa);
- On the other hand, the limit value for E = 16 GPa. has not been verified in the results presented herein. Therefore, it is recommended that this limit be eliminated from the Brazilian standards.

5. Acknowledgement

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7. Notation

- f_b. f_{bk} = average and characteristic compression strength of the block (MPa);
- f_p. f_{pk} = average and characteristic compression strength of the prism (MPa);
- f_m. f_{bm} = average and characteristic compression strength of the mortar (MPa);
- f_k = characteristic compression strength of masonry (MPa);
- f_{ppk} = characteristic compression strength of small wall (MPa).