BEHAVIOR OF CONCRETE BLOCK MASONRY BEAMS AND WALLS

Jamshid Zohrehheydariha
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BEHAVIOR OF CONCRETE BLOCK MASONRY BEAMS AND WALLS

By

Jamshid Zohrehheydariha

A Thesis
Submitted to the Faculty of Graduate Studies
through the Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for
the Degree of Doctor of Philosophy
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Windsor, Ontario, Canada

2017

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BEHAVIOR OF CONCRETE BLOCK MASONRY BEAMS AND WALLS

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DECLARATION OF CO-AUTHORSHIP AND PREVIOUS PUBLICATION

I. Co-Authorship Declaration

I hereby declare that this dissertation incorporates material that is the result of joint research undertaken in collaboration with Con-Tact Masonry Ltd and Canada Masonry Design Centre (CMDC) under the supervision of Dr. S. Das. Dr. Bennett Banting made substantial contribution to conception, design and interpretation of data as well as revising the article. The collaboration is covered in each of Chapters 2 through 5 of this dissertation. In all cases, the key ideas, primary contributions, experimental designs, data analysis and interpretation, were performed by the author, and the contribution of co-authors was primarily through advice and assistance with experimental testing.

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I certify that, with the above qualification, this dissertation, and the research to which it refers, is the product of my own work.
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<tr>
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This dissertation presents the experimental results of laboratory testing conducted on full-scale masonry beams that are constructed in stack pattern and running bond using different block size and grout strength. The first study compares effect of grout strength and block size on the structural behaviour of masonry beams. It was found that the grout strength has the largest effect on the maximum load or moment carrying capacity of a masonry beam. It was also found that The effect of the block unit size on the load carrying capacity of masonry beam is negligible. The second study was on the effect of bonding pattern on structural performance of masonry beams. It was found that there were no considerable differences in structural performances and failure modes obtained from these two beam construction. The last study is one on the experimental and field performance of PP band retrofitted URM wall. It was found that retrofitting of URM wall using PP band enhances the ductility capacity and energy absorption capacity almost 3 and 2 times, respectively, in comparison to unreinforced masonry wall. The non-contact optical metrology called digital image correlation (DIC) successfully implemented throughout the study to extracts full-field deformation and mechanical properties of various construction material. Hence, this study concluded that the stack pattern construction can be used as the way the running bond construction are currently used.
I dedicate this thesis to my family for keeping my dreams alive.

♦♦♦

God could not be everywhere, and therefore he made mothers.

~ Rudyard Kipling
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CHAPTER 1

GENERAL INTRODUCTION

Masonry construction utilizes different materials for developing masonry structures; such components include reinforcing steel, grout, mortar, and masonry units. The masonry structure strength depends upon the interactions of the aforementioned components. Construction of masonry prisms is undertaken using grout, mortar, and concrete masonry units (CMUs). The specific compressive strength for masonry is denoted as $f'm$, which an engineer specifies; and it is utilized throughout the design procedures of masonry. This kind of strength features lower and upper bounds that the Building Code Requirements and Specification for Masonry Structures (MSJC, 2008 [1]). This chapter reviews previous works on masonry prisms, beams, and walls.

Masonry construction activities are physically demanding and have high risks of work-based injuries. This is caused by undertaking heavy physical tasks that include grouting, laying bricks/blocks, handling bricks/blocks, dismantling and erecting scaffolds [2,3]. For example, a standardized Concrete Masonry Unit with dimensions of $8'' \times 8'' \times 16''$ weighs between 28 and 35 lb, and masons typically lays between 150 and 250 CMUs each working day. Within brick masonry works, a mason lays an average 1000 bricks every day [4]. Masons are required to bend, twist and lift while laying blocks/bricks. Laying such a quantity of blocks/bricks every day could trigger considerable physical load, leading to musculoskeletal disorders (MSD) among masons. According to a recent report from the Bureau of Labor Statistics (BLS, 2009, [5]), masonry construction is considered a high – risk specialty trade with non-fatal incident rates of 191.5% for every 10,000 equivalent full-time employees as well as 2,640 recorded injuries. It was discovered that masonry
possessed the second–ranked incidence rate for all trades of construction over injuries with gone working days because of overexertion arising from lifting [4]. Additionally, they reported that associated costs from medical care for masons appeared highest for all construction works.

As cited earlier, masonry work features a considerable physical demand [6,7]. It was discovered that most demanding activities were one-handed repetitive brick lifting as well as two-handed block lifting [8]. For the laborers (mason assistants) pulling/ pushing wheelbarrows, carrying materials, and manual lifting for over four hours every day emerged as the most physically demanding tasks.

According to BLS (2009) [5], in 2008, being struck by the objects accounted for the many masonry work injuries with a figure of 27%, falls came in second with 21% and overexertion was ranked third with 12% for all masonry–related injuries. The most common musculoskeletal disorder (MSD) among masons was low back pain (LBP) followed by neck, knee, and wrist/hand injuries [9,10].

Additionally, masonry workers come into contact with harmful substances including Silica dust, which can trigger severe respiratory illness and in some cases fatality [11]. Different studies have suggested some controls aimed at reducing the MSD risks within masonry tasks. Such controls comprise of using equipments for lifting blocks of more than 40 lb, pumping mortar to platforms, block/brick stacks to about 50cm (1.7 feet), and adjustable scaffolds for keeping working height in between 60 and 90 cm [2,4,12]. Other researches have suggested engineering controls such as using lightweight blocks in reducing masonry workers’ ergonomic loads [6, 8,13].
According to BLS (2009) [5], masonry construction claiming 23 fatalities alongside a 0.012 fatality per 100 equivalent workers on full-time basis is not considered as high fatality risk trade. Falls from elevated points with 43.5% as well as contact with the objects having 30.5% of all accident cases constitute the major masonry construction fatality causes of fatalities. The aforementioned statistics strongly indicate that masonry workers’ safety should be accorded significant attention.

The statistics in table 1 below describe typical homes and housing market in 2000 [14].

Table 1.1 Typical homes and housing market in 2000

<table>
<thead>
<tr>
<th>Garage</th>
<th>2 cars (65%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bathrooms</td>
<td>1-1/2 or less (7%); 2 (40%); 2-1/2+ (53%)</td>
</tr>
<tr>
<td>Bedrooms</td>
<td>2 or less (12%); 3 (54%); 4 and more (34%)</td>
</tr>
<tr>
<td>Stories</td>
<td>Single story (48%); 2 or 1-1/2 story (49%)</td>
</tr>
<tr>
<td>Average size</td>
<td>2000 sq. ft.</td>
</tr>
<tr>
<td>Number of housing</td>
<td>1.54 million (80% single family)</td>
</tr>
<tr>
<td>Overall housing units</td>
<td>107 million (about 50% single–family)</td>
</tr>
<tr>
<td>Ownership rate</td>
<td>67%</td>
</tr>
<tr>
<td>Form of purchase</td>
<td>8% (numerous funding options)</td>
</tr>
<tr>
<td>Price of new homes</td>
<td>200,000 dollars</td>
</tr>
<tr>
<td>Median family income</td>
<td>45,000 dollars</td>
</tr>
<tr>
<td>Population</td>
<td>270 million (24% rural, 76% urban)</td>
</tr>
</tbody>
</table>

Seemingly, masonry structures constitute the popular construction style of low-rise structures in both developing and developed countries globally [15]. According to Frankie et al., such buildings account for over 75% of building populations within several countries [16]. Despite representing a huge portion of building stock, masonry structures have a remarkable history of poor performance in areas that experience earthquakes [17].
1.1 SUMMARY OF LITERATURE REVIEW

This chapter provides summary of literature review completed. The focus of this literature review is to determine the previous research work completed on (i) masonry Prism and (ii) masonry walls.

1.1.1 Compression Testing of Masonry Prisms

Drysdale and Hamid provided an understanding regarding the best testing methodology for acquiring the compressive strength of masonry [18,19]. These studies conducted tests on masonry prisms made of half block units and full block units to determine the effect of block unit size on the compressive strength of the masonry prism. The results revealed that utilizing half block in the prism specimens provides similar outcomes as obtained from full block prism specimens [19]. The study concluded that the block size in laboratory prism testing may be smaller compared to those utilized in actual construction, however, the results obtained from prisms made of reduced size units in the lab can provide accurate prediction of compressive strength.

The compressive strength of masonry is also dependent upon the component interaction in the masonry structural system. Extensive investigation of the component interaction in the prism specimen subjected to axial compression load was completed. These studies found considerable effect of individual component on the compressive strength of masonry prism specimens. It was concluded that the compressive strength of fully grouted masonry prism specimen is dependent less on mortar joints; hence, increasing the thickness of mortar joint twice reduces the compressive strength by 3% for grouted prisms and 16% for hollow prisms [19]. Hence, the study found that the strength of mortar does not have a much effect
on the compressive strength of masonry [19]. Type of mortar should not have a large influence for on the compressive strength of grouted prism. However, Fahmy and Ghoneim found that for both grouted and ungrouted prisms, the strength increases if the strength of the mortar increases [20]. Hence, the study completed by Fahmy and Ghoneim contradicts the finding of Drysdale and Hamid [19,20].

Drysdale and Hamid observed that there was no proportional contribution of the strength of the grout to the strength of the prism specimen [19]. It was concluded that increasing grout strength slightly increases the compressive strength of the prism.

Fahmy and Ghoneim found that the strength of prism increases when the strength of the block unit increases [20].

Drysdale and Hamid studied the effect of height-to-thickness ratio of prism specimen on its compressive strength. The study found that two-course high prism results in a poor correlation with the behaviour of masonry wall [19].

Maurenbrecher studied the effect of the loading rate and the study found that the slower loading rate yields only small drop in strength. Average strength is utilized in many prism test investigations encountered and within lab testing for constructed masonry to verify the strength. Testing of at least ten replicates to generate accurate outcomes for achieving characteristic strength [21].

1.1.2 Relevant Requirements for Grout and Masonry

The compressive strength of masonry based on the prism test data should either be at least 1,500 psi (10.3 MPa), however, it should not exceed 1,500 psi (27.6 MPa) (MSJC, 2008). The prism testing technique that has been used to verify $f'_m$, ASTM C476 (2010) reveals
that masonry grout should have a minimal compressive strength of 2000 psi (13.8 MPa) at four weeks [22]. TMS (TMS, 2016) Indicates that the specific compressive strength of grout \( f'_g \) should be at least the same as the masonry compressive strength, while not surpassing 5000 psi (34.5 MPa) [23]. Curing ages where for masonry systems and grouts are not specified in the masonry code, Hence, 4-week (28-day) strength references may be used. Masonry grout is determined by other aspects than merely compressive strength. Sections 3.1.1.5 and 3.1.1.6 in ASTM C476 (2010) restrict slag and fly ash use by referring to ASTM C595/C595M (2010) that limits the content of maximum pozzolan to 40% by the blended cement mass and the overall pozzolan content as well as granulated blast furnace slag to not more than 70% by blended cement mass [22,24].

1.1.3 The Effect of Loading Direction on the Compressive Strength of Masonry Prisms

Little information exists with regard to concrete masonry compressive strength if compressed parallel to the bed faces, which is represented by \( f_{mp}' \) in the paper. The Canadian Standard, CSA 304 (2014) is not specific regarding any particular testing procedure for obtaining such value [25]. However, it has suggested a technique for determining the strength, \( f'_m \) based on the specific compressive strength perpendicularly towards the bed faces, which is represented by \( f'_m \). In the current standard, the \( f_{mp}' \) is measured from the \( f'_m \) value by multiplying with a reduction factor, \( \chi \), as illustrated in equation 1.

\[
      f_{mp}' = \chi \cdot f'_m \quad (1)
\]
Where, \( \chi = 0.5 \) for the compressive force is applied normal to the head face and if grout is not continuous horizontally within the compression zone. A few past studies were undertaken by other investigators to examine the properties of concrete masonry that is loaded parallel towards the bed face [26,29]. Different kinds of testing setups were utilized including use of single or double block prism samples [26,27], others examined beam specimens [28,29]. The findings from the studies differed from the two forms of tests. Additionally, the procedures were different. The findings from the study that was restricted to prism samples made from the unit shapes and types showed that compressive strength or hollow prisms when loaded perpendicularly towards the bed face displayed a considerable value compared to when loaded parallel towards the bed faces. This finding appears to replicate current findings. Secondly, the study found that the compressive strength of grouted prisms loaded parallel towards the bed face is greater compared to that loaded perpendicularly towards the bed faces regardless of the interruption level within the grout area. This contravenes recommendations within the existing Canadian standard [25]. Extensive investigation might need to be initiated even though the observation can be likened to recommendations that other investigators have made. More study may need to be done [28].

1.1.4 Masonry Beams

Before the Portland cement manufacture, mortar was mainly a mixture containing crushed brick/stone, water, and lime. In the 19th century, a mixture comprising water, sand, and cement emerged as the strongest mortar. In the current decade, engineered cementitious composite (ECC) has been substituting the cement mortar because of its greater ductility,
shear resistance, carrying capacity of tensile load, and higher strength. Although the properties of materials and use of ECC for retrofitting structures and constructing bridges are established, several investigators have examined the use of ECC and properties of materials for latter parts. Li has studied the material characteristics of ECC alongside its use for repairing structures and asserted that shear reinforcement within concrete may be replaced effectively by ECC [30].

Kyriakides et al. investigated the flexural strength for masonry beams that are retrofitted using ECC stratum alongside brick-mortar interface opening coupled with ECC stratum failure below mortar joints. Kyriakides et al. have utilized ECC within different civil engineering applications [31]. The application of FRP for structural strengthening has elicited significant attention because of chemically inert characteristics, non-corrodible, and greater tensile strength. There is extensive literature regarding the application of FRP sheets for external strengthening of supported concrete structure members.

Yuan et al. investigated the flexural property of ECC beams, concrete composite beams, and ECC, and found that Fiber reinforced polymers (FRP)-reinforced ECC beams exhibit significant flexural properties with regard to damage tolerance, ductility, shear resistance, and load-carrying capacity than FRP–reinforced concrete beams [32].

Barros and Fortes, Tang et al., Ambrisi and Focacci investigated the flexural strengthening of concrete beams reinforced using fiber-reinforced cementitious matrix (FRCM). The investigators found that near-surface mounted (NSM) approach of strengthening concrete beams led to an increase in flexural strength [33-35].

Hajihashemi et al. studied the characteristics of concrete beams that are strengthened using pre-stressed Carbon fiber reinforced polymers (CFRP) strips through NSM methods and
found that pre-stressed strengthened beams possessed 15% greater ultimate load-carrying ability than the non-pre-stressed sample [36].

Tomlinson and Fam undertook the shear and flexural performance for strengthened concrete beams having basalt fiber Reinforced Polymers (BFRP) strips and bars. Tomlinson and Fam found that beams containing BFRP strips slumped in shear whereas ultimate flexural ability rose between 55% and 58% for beams lacking stirrups [37].

A dearth exists in literature with respect to masonry beam strengthening. Kiss and Kollar (2002) and Moon et al. (2007) have investigated the flexural reaction of FRP strip strengthened masonry beams. The impact of the FRP on the ductility and failure load of masonry beams was studied. The investigators discovered that the beams’ ductility and flexural capacity increases as the FRP is strengthened [38,39]. Galal and Enginsal examined the flexural property of concrete masonry beams that has been strengthened using Glass fiber reinforced polymers (GFRP) rebars and came to the conclusion that stiffness and flexural capacity of strengthened improved significantly [40].

1.1.5 Masonry Wall

Unreinforced masonry (URM) buildings suffered severe damage and collapse during past earthquakes as compared to steel and reinforced concrete buildings [41-44]. Starting from hair line crack to total collapse is the extent of failure of these URM buildings existed in seismic active parts across the globe. Various retrofitting techniques are used to improve the seismic performance of unreinforced masonry [45]. However, composite materials like fiber reinforced polymers (FRP) is the most preferred one due to its high strength-to-weight ratio and corrosion resistance.
Mustafa Taghdi et al. provided a description of the retrofitting mechanisms put in place for strengthening the partially reinforced walls and the un-reinforced walls [46]. The retrofitted walls containing steel strip system made of vertical and diagonal strips were attached using through-thick bolts. Anchor bolts and stiff steel angles were utilized for connecting the steel strips with the top loading beam and the foundation. All the walls were subjected to testing under integrated constant gravitational load and incrementally escalating in-plane lateral deformation reversals. Additionally, the lightly reinforced concrete walls were repaired with vertical strips and subjected to another test. The tests revealed that complete steel strip system was efficient and considerably escalating the lightly reinforced concrete walls, the partially reinforced masonry wall and low–rise unreinforced ductility and their in-plane strength. The capacity exhibited by un-reinforced masonry walls in resisting lateral load is undermined by the bed joint mortar and masonry unit strengths. Shear failures may be overcome with heavy horizontal reinforcements and lighter vertical reinforcements, thus enhancing flexural behaviors.

Varying the joint or bond pattern of concrete masonry walls may form an extensive variety of attractive and interesting appearances through standard units and sculptured –faces, alongside other architectural units. Since concrete masonry is utilized frequently for finishing wall surfaces, the application of bond patterns instead of conventional running bonds has increased steadily for non-load bearing and load-bearing walls. Building code permissible design stresses, minimum breadth, and lateral support requirements are mainly based on structural testing as well as studies investigating all panels placed in running bond construction. When another bond pattern is utilized, it is imperative to consider the effect it would have on the block wall’s flexural and compressive strength. Certain building codes
cater for changes in bond pattern by requiring horizontal reinforcement to be utilized, for instance, while laying walls within stack bond.

1.1.6 Stack Pattern and Running Bond Construction

With the exception of constructing running bond, stack bond is the most common and extensively utilized bond pattern in concrete masonry units. There is a similarity for running and stack bond construction with regard to compressive strength. In stack bond masonry heavy concentrated loads are carried downwards to the supports by specific vertical tiers or masonry column under loads, with negligible distribution towards adjacent masonry. Stability would be in jeopardy if permissible stresses are unsurpassed, but the application of supported bond beams would assist in spreading out the concentrated loads. Additionally, the application of grouted cells or pilasters will be efficient in enhancing the resistance towards concentrated loads. The stack bond wall flexural strength that spans horizontally may be increased using joint reinforcement or bond beams. Notably, it can be inferred that well-reinforced stack bond masonry may be designed to the same strength as that for running bond.

1.1.7 Code Requirements

Building Code Requirements for Masonry Structures (2013) features criteria of walls laid within stack bond [1]. Although stack bond specifically implies masonry constructed in a manner that there is vertical alignment of head joints within successive courses are offset horizontally less than a quarter of the unit length. All stack bond masonry should have a minimum horizontal reinforcement area equivalent to 0.00028 times the wall gross vertical
cross-sectional area. This specification could be fulfilled with bond beams that are spaced less than 1219 mm (48 in.) at the center or through joint reinforcement. The veneer of anchored masonry should have horizontal reinforcement of joints, of about 1 wire size, of at least one wire size 9 gauges (W 1.7) or (MW11) or bigger, spaced at maximum length of 457 mm (18 in.) on vertical centers. This equals the required reinforcement cited above for the nominal 102 mm (4 in.). When construction of stack bond is subject to hurricane velocity winds or seismic loads, considerations should be directed to other specifications and restrictions in line with engineering practice, local experience, and local codes. For instance, Building Code Requirements for Masonry Structures (2013) requires stack bond masonry within Seismic Design Category D and greater for solid units with a maximum space of 610 mm (24 inches) for reinforcement or solid grouted hollow open-ended units, full grouted hollow units having complete head joints [1]. Seismic Design Category E & F have another specification requiring horizontal reinforcement to have at least 0.0015 in terms of wall cross-sectional area for walls, which do not form part of the resisting system of lateral force. For walls, which constitute the resisting system of lateral forces, the minimum specification for horizontal reinforcement should be increased to 0.0025 times the area of the gross cross section with an optimum spacing of 406 mm; additionally, such elements should be solid grouted hollow open-ended units or two solid unit wythes.

1.1.8 Compressive Strengths

The decline in strength for vertical stack bond has a direct correlation with decreasing net block compression area. Within the vertical positions, the interior and end webs are very oriented with regard to stress direction that they lack any contribution to the wall
strength save for ties involving the face shells. If blocks are placed horizontally, the middle and end webs lie parallel to stress direction, thus strengthening the wall [1].

1.1.9 Vertical Span Flexural Strength

Where there is vertical spanning of walls between lateral support, failure triggered by transverse loading takes place because of bond failure involving mortar and block [47]. Construction of horizontal stack bond appeared stronger within vertical span flexure, and walls constructed using diagonally laid units also appeared stronger since additional mortar bond area was included within the vertical span flexure, and walls built with “saw-tooth” line across the width of the wall [47,48].

1.1.10 Horizontal Span Flexural Strength

For unreinforced concrete masonry placed in running bond and horizontally spanning between the lateral supports’ flexural resistance is dependent on the block design and strength. Under escalating lateral loads, the units would rupture under tension than fail based on mortar bond. Because of this, walls are twice generally strong within flexure if spanned horizontally. However, this is not applicable to stack bond laid walls that have almost similar strength within the two directions. Since there is no relevant research undertaken to evaluate the structural competency of full-scale stack pattern masonry beams or walls, CSA S304, (2014) has suggested that masonry beams be built using running bond construction [25]. Since CSA S304 (2014) does not provide any design guidelines for implementation of stack pattern construction in load-bearing structures, its use in Canada is restricted to decorative and non-load bearing structural uses only [25].
1.1.11 Alternative Materials

The key towards any housing structure is the selection of proper technologies for specific regions. For instance, while structural reliability for reinforced concrete has been accepted widely, generating and applying such conventional materials might require skilled workforce, alongside educated supervision, and the production costs to development sites is prohibitive.

Various structural materials, which form a portion of the conventional material costs, have been studied in the past. For instance, different materials are investigated as replacement of cement for mortar they include furnace slag, gypsum fly ash, lime, rice ash, and rice husk [49]. Earthen blocks are utilized extensively across the globe; however, are subject to low strength, shrinkage, and poor durability. It is suggested that the stability of earthen blocks to escalate the strength be undertaken using cement or utilize fibers that include barley straw in reducing shrinkage and reinforcing masonry [49].

Many earlier studies completed to determine the prevention or reduction in the development of tensile and shear cracks in masonry and concrete beams. Some of the methods used to reduce excessive cracking in beams is the use of fiber reinforced polymers (FRP), use of skin reinforcement, use of intermediate steel [50-52]. Some studies even used bamboo as the reinforcement [49].

Concrete has emerged as strong in terms of compressive strength; however, it has limited tensile strength and steel has higher tensile strength but limited compressive strength. Additionally, the traditional steel–reinforced concrete construction is used widely across the world. Thus, a scarcity and higher production costs triggered by depleted natural resource for materials [53]. Such impediments have compelled significant development
within the construction setting for formation of modern concrete technologies including application of waste as well as local materials in concrete production and for improvement of properties. Consequently, investigations by researchers have found sustainable materials that might replace or substitute natural materials. Recycled concrete, steel slag, and waste tyres have been proposed replacements for natural rough aggregates [54-56]; crushed granite fines for fine aggregates, sheet glass powder, waste paper, saw dust, pulverised plastics, and laterite [57,58]; bagasse ash, fly ash, wood ash, and rice husk ash for cement [59-61]. Notably, traditional reinforcements are being substituted with local fiber material in the making of concrete. Shende and Pande investigated the physical characteristics of steel fiber reinforced concrete and found that flexural strength, split tensile strength, and compressive strength increases as the fiber content increase for percentages involving considered fiber [62]. Nataraja examined the steel fiber reinforced concrete’s (SFRC) splitting tensile strength with a 100 mm cube specimen. The splitting tensile, compression, and flexural tests revealed that the SFRC’s splitting tensile strength was 0.67 times that of flexural strength, and 0.09 times that of compressive strength. Seemingly, fibre-reinforced concrete could be utilized for improving the concrete structural members including floors, columns, and deep beams with regard to ductility, toughness, and crack-reduction [63]. Likewise, Lee found that large quantities of longer fibers will yield reliable mechanical performance over concrete when distributed uniformly [64].

Nevertheless, there might be an issue arising from uniform distribution and workability with rising fiber length and volume. Additionally, investigators have examined bamboo as the ideal substitution for steel within reinforced concrete. According to Chu, one major study regarding the application of bamboo within a cement matrix occurred in 1914 [65].
Indeed, bamboo might be re-harvested after three years, thus replenishing its availability. Chauhan et al. examined some mechanical and physical characteristics of bamboo at various ages and culm height [66]. Considering the engineering properties as reinforcement within concrete, only scarce research has been undertaken with regard to this. Moroz et al. investigated how shear walls of bamboo reinforced masonry performed, it was found that inclusion of vertical bamboo reinforcements offered additional shear strength, while also providing a ductile failure than unreinforced masonry [49]. In a similar study, Wu and Zongjin found that bamboo might be used to reinforce cementitious composites because of its superior characteristics that include harmless towards the environment in service, easy availability, low cost, greater tensile strength, and high strength towards weight ratio [67]. In contrast, Ghavami used bamboo to reinforce the production of light concrete beams and the study found that bamboo offers an improvement in the outcomes [68]. Adom-Asamoah and Afrifa in their investigation of shear strength for bamboo reinforcement concrete found that concrete beams supported with split bamboo culms along the horizontal axes, developed significantly greater loading capacities compared to unreinforced concrete beams with similar sections [69]. Additionally, Satjapan examined the ductility and compressive strength of short columns of concrete supported using bamboo. It was found that 1.6 % steel reinforcement within a cross-section column might be substituted with 3.2% of treated reinforced bamboo to attain similar ductility, strength, and behavior [70]. However, many existing literatures focused on the normal application of bamboo and steel fibers within concrete; however, none has taken into consideration their application in producing high-strength concrete. Hence, to determine the difference within physical characteristics of bamboo and steel fibre-reinforced concretes, the study discovered a gap
in the evaluation of bamboo fiber and steel on the compressive, splitting tensile and flexural strengths for high-strength concrete.

In summary, the literature review reveals that there are lack of experimental results on the masonry stack pattern construction. In addition, the limited use of high strength grout and different block size. Also, the most of the current retrofitting techniques proposed for such structures (masonry walls) are very expensive while one of them consisting of retrofitting using polypropylene (PP) band seems to be economic.

1.2 OBJECTIVES

The primary objective, as well the motivation for this dissertation, is that there is already an inherent conservatism within the masonry structures design standard produced by the Canadian Standards Association CSA S304 (2014) that is not an accurate reflection of the behavior of the engineered masonry structural buildings constructed in stack pattern [25]. The approach taken by this author will be through the analytical and experimental program to determine the structural behavior of the stack pattern and running bond masonry construction in masonry prisms, beams and, walls. The other objective of this research is to examining the efficacy of the retrofitting technique using PP band on unreinforced masonry walls.

1.3 METHODOLOGY

The test dates and test methods presented in this thesis are part of a larger research program. The objective of this part of the study (phase 1) is to determine if beams built using stack
pattern construction demonstrate any reduction in capacity and deformation. The first phase of this project was to construct 20 prism specimens and 12 beam specimens to be tested. The second phase of this project involves testing the 35 prisms and 12 beams built with varying grout strength, block size, and construction pattern. All specimens were constructed in Mississauga, and tested at the University of Windsor.

The construction of the prisms and beams was completed at room temperature at the Canadian Masonry Design Centre (CMDC) in Mississauga, ON. In phase one, a total of 12 beam specimens and 20 prisms were built. All beams spanned 4.8 meters long, and ranged from 2, 3 and 4 courses high. The prisms were built in sets of 5, consisting of 4 different types. The types were as follows; stack pattern with vertical head joint, running bond with vertical head joint, stack pattern with vertical bed joint, running bond with vertical bed joint. In phase two, a total of two unreinforced masonry (URM) wall specimens and 12 beam specimens were built. URM wall specimen dimension was 1600 mm (length) × 2200 mm (height) × 200 mm (thickness). All beams spanned 4.8 meters long and 3 courses high. In addition, 15 prism specimens were made and tested to determine the material properties and compressive strength of the masonry used in making the beam specimens. The prisms were built in sets of 5, consisting of 7 different types.

1.3.1 Prism Test

To obtain a uniformly distributed load across the entire cross-sectional area of the prism specimens, the contact surfaces needed to be capped with a quick set mortar, and steel plates. The capping compound used for the prisms and beams was Euco-Speed Red Line produced by EUCO. This product was purchase from Target Building Materials located in
Windsor ON. Before the prism can be tested they must be capped using the capping compound mentioned earlier, drilled to allow for mounting of LVDT’s, and centered under the 3000 kN capacity loading jack. Two LVDT’s of 5 mm stroke were placed on each side of the prisms. They were mounted using Tapcon screws and aluminum angles. The top angle had a threaded bar and a nut on either side to hold it in place. The bottom screw had a hold the size of the LDT’s diameter with a slot to allow for tightening (Figure 1a). The gauge length of the LVDTs was 635 mm, covering roughly 3 blocks and two mortar joints. Strain was determined by calculating change in length divided by original length. The prisms were tested in a random order ensuring any testing errors were not isolated to the same prism type.

Once prisms were centred under the loading jack, the spherical head was lowered into the slot on the swivel head to lock everything in place. The entire setup from top to bottom consists of the spherical head of the load cell jack pressed into the swivel head which is sitting on to the top loading plate. Under this is the capped prism sitting on the bottom loading plate all sitting on the strong floor (Figure 1b). Load was applied slowly and deflection and load data were acquired using DAQ (LabView) and DIC (VIC 2D). Once half the expected capacity was reached, the load was released and the four LVDT’s were removed. Loading continued until rupture of the specimen. Once the specimen failed, the plates were recovered and the prism specimens were discarded.
1.3.2 Beam Test

All beams were tested using the same test setup. The beams were simply supported spanning 4.5 meters in unbraced length. A roller support was simulated using two plates and round rod. The pin support was simulated using two plates and square stock imbedded in the plates. On top of the beams, the same supports were used and a 700mm long steel loading beam was placed on the top to simulate a four-point bending load. A swivel plate was then mounted on the loading beam to obtain uniform contact between the head of the jack and the loading beam (Figure 2). Two-point loading was used to increase the span in which maximum moment occurred.
Load was gradually applied to the beams using a hydraulic jack controlled by a pump. The pump had a valve in which we used to fine tune the rate of loading. At predecided deflections, loading was paused and cracks were marked on the north face of the beam. This was done when crack growth was visible, and the condition of the beam was safe. Loading was continued until a clear failure mode was visible in the beam. Once the test was complete the beam was slowly unloaded and discarded into a large waste bin.

Finally, by using experimental results, the semi-empirical equation developed using the symbolic regression, or symbolic function identification to calculate the effect of the different parameters such as height of the beam, stirrups, grout strength, and block size on the ductility of masonry beams constructed in stack pattern and running bond.

1.3.3 Wall Test

Two masonry wall specimens were constructed and tested in displacement controlled lateral loading frame to determine the effectiveness of retrofit technique of URM wall using PP band. First wall specimen was unreinforced and the second wall specimen was
reinforced using PP band. Each wall specimen was made of 200 mm × 200 mm × 400 mm concrete blocks. The boundary conditions of these wall specimens were fixed at the base and free at the top. The PP band was laid in both horizontal and vertical directions and the average grid of the PP band was 200 mm × 200 mm. These bands were tightened using mechanical equipment. Each PP band continued on both faces of the wall specimens and hence, these PP bands reinforced the wall specimen as well as some level of confinement to the wall specimen. (The PP band is 12.7 mm wide and 0.67 mm thick and the breaking strength is 1.23 kN). The reinforced wall specimens were subjected to same monotonic loading using displacement controlled method.

1.4 ORGANIZATION OF DISSERTATION

This dissertation consists of six chapters. The first chapter provides a general introduction and the very last chapter, Chapter 6 consist of general discussions and conclusions. The second chapter discusses the combined effect of using three different masonry blocks with 150mm, 200mm and 350mm width, along with 2 types of normal and high strength grout in stack pattern and running bond masonry beams were analyzed in the full scale tests.

The third chapter provides a detailed account of results on the ductility, mode of failure, and strain distribution in the stack pattern and running bond masonry beams. Further, this chapter discusses the effect of different slenderness ratio on the strain concentration and failure of masonry beam specimens.
The forth chapter describes the efficacy of the retrofitting technique using PP band through experimental program. The monotonic load displacement behavior of URM wall and the wall retrofitted with PP band is compared.

The fifth chapter provides a detailed information regarding the Digital Image Correlation (DIC) technique and its implementation in masonry structures such as prisms, beams, and walls.

1.5 REFERENCES


CHAPTER 2

EFFECT OF GROUT STRENGTH AND BLOCK SIZE ON THE
PERFORMANCE OF MASONRY BEAM

2.1 INTRODUCTION

While masonry is one among the oldest building materials, complexity involved with the behavior of masonry structure is still not well understood. Various components of masonry such as block unit, grout, mortar, and reinforcement act together as a composite material. Many of these components have anisotropic properties resulting masonry construction to exhibit non-isotropic properties. Several studies were conducted in the past to determine the effect of these components on overall behavior of the masonry structures. Various alternative materials were also investigated as the replacement of cement in the mortar. The alternative materials used in these studies included furnace slag, gypsum fly ash, lime, rice ash, and rice husk [1].

Limited information with regard to the effect of block unit size and grout strength of masonry is available. Drysdale and Hamid, based on their research, recommended the best experimental technique for determining the compressive strength of masonry [2]. In this study, masonry prism specimens made of half-blocks and as well as full-blocks were tested to determine the effect of the size of the block unit on the behavior of masonry. The study concluded that half-block prisms provides similar outcomes to that of full-block prisms. Fahmy and Ghoneim found that for both grouted and ungrouted prisms, 40% increase in the strength of mortar led to an average increase in the strength of prism by only 12% [3]. The effect of mortar is more significant if failure occurs due to splitting of masonry units.
Drysdale and Hamid observed that there was no proportional contribution of grout strength to the strength of the masonry prism and the increase in grout strength resulted marginal increases in the strength of masonry prism [2]. Fahmy and Ghoneim reported that the strength of prism increases when the strength of the block increases [3]. For ungrouted prisms, 50% increase in block strength resulted in an average increase of about 15% in prism strength. However, for the grouted prism specimens, 50% increase in the block strength resulted in only 8% increase in the prism strength.

Edwin et al. investigated the effect of proportion of grout (cement-to-sand ratio) on the physical properties such as modulus of elasticity and compressive strength of grout and grouted concrete masonry [4]. Shin et al. implemented ultrasound technique during curing of grout to increase physical properties of cement grout such as the uniaxial compressive strength [5]. The increase in the properties were determined by undertaking tests on cylinders of grout specimens. Xue and Mao developed modified cement mortar by adding polyvinyl-butylar and methylcellulose to cement mortar mix [6]. Then the study undertook tests to determine the bond strength of the modified mortar and compared that with the regular cement mortar. The study found that bond strength of the modified mortar was about 65% higher than the regular cement mortar.

Previous studies implemented the use of fiber reinforced polymers (FRP) and its effective bond length and behaviour to reduce excessive cracking [7-9]. As an alternative, lightweight masonry mortar [10] and lightweight masonry block [11] was introduced.

Current Canadian standard, CSA S304 [12] does not allow stack pattern (SP) construction in masonry beams. American masonry code, TMS [13] also provides restrictions on SP masonry beams. The limitation is due to the belief that the stack pattern (SP) masonry beam
is weak since it is susceptible to the development and faster growth of flexural cracks through the head joints, which are continuous and not interrupted by block units in alternate courses. In running bond (RB) beams, the head joints are not continuous since the block units in the adjacent course (Figure 1) interrupt them. However, no studies are reported in the literature where effect of SP construction on masonry beam was studied.

![Running bond and stack pattern constructions](image)

**Fig. 2.1.** Running bond and stack pattern constructions

Hence, the literature review reveals that only one study was undertaken to determine the effect of grout strength on masonry prisms. The same study also determined the effect of block unit size on masonry prism strength. However, literature review did not find any previous studies on the effect of grout strength and block size on the behavior of masonry beams. Further, no previous researches studied the effect of construction pattern on the performance of masonry beam. Hence, the current study was carefully designed and executed to determine the structural performance of masonry beams with two different block sizes and two different grout strengths. In addition, performance of stack pattern masonry beams was compared that of with similar running bond masonry beams. This paper discusses the test matrix, instrumentation, test procedure, and data obtained from the full-scale tests conducted under the scope of this study.
2.2 EXPERIMENTAL PROGRAM

This research work was completed using six full-scale reinforced masonry (RM) beam specimens. Twenty-five grouted prism specimens were also tested. Further, material tests on block units, mortar, grout, and steel rebar were completed in accordance with relevant standards to determine their properties [14-16]. The values are reported in Table 1. The prism specimens were four-course high and fully grouted. The $f_m$ calculated as per annex C and D ($f_m = f_{av} - 1.64S$).

The test data obtained from the prism specimens were used to determine specified compressive strength ($f'_m$) and modulus of elasticity ($E_m$) of masonry in accordance with Canadian standard [12] as shown in Table 2. The load data was acquired through a loadcell attached to the loading actuator and the deflection of the prism specimens was measured using digital image correlation (DIC) technique. The prism specimens are named such that they indicate their attributes. The first letter, “N” and “P” refers to the loading direction: “N” for normal to the bed joint and “P” for parallel to the bed joint. Second letter explains the grout strength. The letter “N” is for normal strength grout and “H” is for the high strength grout.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Beam test-day values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Failure load (kN)</td>
</tr>
<tr>
<td>20 cm block</td>
<td>544.3</td>
</tr>
<tr>
<td>30 cm block</td>
<td>754.4</td>
</tr>
<tr>
<td>Mortar</td>
<td>-</td>
</tr>
<tr>
<td>Normal strength grout</td>
<td>-</td>
</tr>
<tr>
<td>High strength grout</td>
<td>-</td>
</tr>
<tr>
<td>Reinforcement ($f'_y$) – 10M</td>
<td>-</td>
</tr>
<tr>
<td>Reinforcement ($f'_y$) – 20M</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2.1. Material properties
The test matrix for prism specimens is shown in Table 3. As can be found in this table, test parameters chosen in this study are: (i) block unit size, (ii) grout strength, and (iii) construction pattern. These specimens were made of two different block unit sizes and these are 20 cm and 30 cm units. Actual dimensions of these units are: 390 mm x 190 mm x 190 mm and 390 mm x 290 mm x 190 mm as can be seen in Figure 2. Grout of two different strengths were used and these are: normal strength grout which had average compressive strength of 22.5 MPa and high strength grout with average compressive strength of 67 MPa (Table 1). Effect of two construction patterns namely, running bond (RB) construction and stack pattern (SP) construction were also studied (Figure 1).

The naming of the beam specimens is done to identify the main attributes (parameters) of the beam specimens. The first character in the name of the beam refers to construction pattern (R for RB and S for SP). The next number indicates the width of the block unit (20 cm or 30 cm). The last character is related to the grout strength: “N” for normal strength grout and “H” for high strength grout. Hence, beam specimen S30N was constructed in stack pattern using 30 cm block units and normal strength grout. The bottom course of all beams consisted of lintel blocks to facilitate placement of main flexural rebars (Figure 3b).
Table 2.3. Test matrix of beam specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Beam specimens</th>
<th>Construction pattern</th>
<th>Unit size</th>
<th>Grout strength</th>
<th>Stirrup (Total)</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Name</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom</td>
</tr>
<tr>
<td>1</td>
<td>R20H</td>
<td>RB</td>
<td>20 cm</td>
<td>High</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
<tr>
<td>2</td>
<td>R20N</td>
<td>RB</td>
<td>20 cm</td>
<td>Normal</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
<tr>
<td>3</td>
<td>S20H</td>
<td>SP</td>
<td>20 cm</td>
<td>High</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
<tr>
<td>4</td>
<td>S20N</td>
<td>SP</td>
<td>20 cm</td>
<td>Normal</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
<tr>
<td>5</td>
<td>R30N</td>
<td>RB</td>
<td>30 cm</td>
<td>Normal</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
<tr>
<td>6</td>
<td>S30N</td>
<td>SP</td>
<td>30 cm</td>
<td>Normal</td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
</tr>
</tbody>
</table>

All the beam specimens in Table 3 were 3-course high (590 mm). All beam specimens were 4.8 m long and they all had a span length of 4.2 m (Figure 4). The stretcher block units used in constructing these beam specimens had reduced web height and it was done to increase the continuity in the grout in the horizontal direction (Figure 3a). All beam specimens had same flexural and shear reinforcements as can be seen in Table 3. The beams were designed to ensure ductile failure in flexure. These beam specimens were built in one phase and all of them were cured in room temperature.
The schematics of the test setup is shown in Figure 4. Pin-roller boundary condition was used to simulate a simply supported boundary condition. A steel spreader beam with pin-roller boundary condition was mounted at the top surface of the beam specimen to facilitate the application of a four-point bending load. The spreader beam was used to produce a constant maximum moment zone of 700 mm at the mid-span (Figure 4). A universal loading actuator was used to apply monotonically increasing quasi-static load at the mid-span of the beam specimen.

As shown in Figure 4, a total of four 100 mm (4 in.) stroke Linear Variable Displacement Transducers (LVDTs) were used for acquiring the displacement data. LVDTs 1, 2, and 3 were used to measure vertical deflections under the bottom of the beam specimen at one-quarter lengths. LVDT 4 was attached to the loading actuator and hence, this LVDT measured deflection at the mid-span on the top surface of the beam. Five loadcell were used for acquiring the load data. Loadcell 5 was attached to the loading actuator and hence, this loadcell was used to obtain the load applied to each beam specimen. Loadcell 1 and 2 were used under the roller and pin supports of the beam specimen to ensure that the applied load is distributed equally. Similarly, loadcell 3 and 4 were used to ensure equal load
distributions onto the top surface of the beam specimen. The behaviors of the steel rebars were monitored with strain gauges, which were installed before the beam specimens were built. Two strain gauges were installed on the flexural steel rebars and four strain gauges on the stirrups (Figure 5).

![Fig. 2.4. Test setup for beam specimens](image)

![Fig. 2.5. Rebar cage and strain gauge locations](image)

The load on the beam specimens was applied using displacement control method. Loading was paused several times and cracks were marked. This was done at increments of about every 5 mm of deflection or when significant crack growth was observed. Loading was continued until a clear failure in the beam was observed. The test data collected during
each beam test included load-deflection behaviour, strain distribution on reinforcements, crack growth, and crack width. Test data was acquired using a computerized data acquisition system.

In this study, DIC technique was implemented to obtain crack growth and crack pattern. Digital Image Correlation (DIC) is a non-contact technique to measure the stain contour. This can be implemented for a variety of tests from simple tensile tests on a material specimen to more complicated tests with rotation and deformation. Measuring and recording crack width information manually when a specimen is heavily loaded is unsafe. Hence, the DIC was useful in collecting and storing all the crack width data, which could not be collected manually.

2.3 TEST RESULTS AND DISCUSSION

2.3.1 Effect of block size

Four beam specimens, R20N, R30N, S20N, and S30N in Table 3 were used to determine the effect of block size on both running bond (RB) and stack pattern (SP) beam construction. These beam specimens were constructed with block units of two different widths: 20 cm and 30 cm. In order to determine the effect of the block size in two different construction patterns, the load-deflection behavior of specimens built with running bond construction, R20N and R30N are compared (Figure 6a). As well the load-deflection behaviors of stack pattern construction, S20N and S30N are compared (Figure 6b). As depicted in Figure 6a, R20N and R30N specimens showed maximum load capacities of 150 kN and 230 kN, respectively. Hence, the difference in their maximum load carrying capacities is about 35%. For the stack pattern beam specimens (S20N and S30N), the
maximum load capacities were found to be about 164 kN and 228 kN, respectively. Thus, the difference in strengths between these two beam specimens is about 28%.

As can be found from Table 1, average failure load values obtained from 20 cm and 30 cm block units are 544.3 kN and 754.4 kN, respectively. Hence, the difference in the average failure loads between these two block units is about 28%. Hence, the study found that the size of masonry block units has no or minimal effect on the load carrying capacities of masonry beams, other than the fact that larger unit provides larger load or moment carrying capacity.

DIC technique was used to acquire crack growth data. Figure 7 shows an example on how crack width data was obtained. This figure shows the horizontal displacement contour of beam R30N just before failure has occurred. The results for only half of the beam is shown in this figure since the test specimen was symmetric about its mid-span. The crack width data obtained from DIC displacement contour using the method was recommended by Corr et al. [17]. The horizontal displacements in this specimen were measured along the reference line shown in Figure 7a. The horizontal displacement profile on the reference
line (as shown in Figure 7b) was computed by comparing two images: the first one was obtained before application of any load (reference image) and the final image was obtained at the time just before failure has occurred. The difference in the value of $U$ (horizontal displacement) in Figure 7b is the crack width at a specific location (shown as point index in Figure 7b) on the reference line of Figure 7a. For example, the crack width at point index 20 is about 2.5 mm ($\approx 2.5 \text{ mm} - 0 \text{ mm}$). The maximum crack width recorded by DIC for this beam specimen was 6 mm at about point index 5, which was located very close to the mid-span of the beam specimen.

![Diagram](image)

(a) Horizontal displacement contour

(b) Horizontal displacement profile on reference line

**Fig. 2.7.** Flexural cracks at maximum load for R30N
For both running bond and stack pattern beam specimens, failure occurred due to crushing of concrete at the top surface and at this stage, the test was discontinued (Figure 8).

![Crushing of concrete](image)

**Fig. 2.8.** Failure of beam specimens R30N

### 2.3.2 Effect of grout strength

Load-deflection behaviors of all beam specimens built with the 20 cm block units are shown and compared in Figure 9 to determine the effect of the grout strength on both running bond and stack pattern masonry beams. Load-deflection data obtained from beam specimens built in running bond but with two different grouts (R20N and R20H) are shown and compared in Figure 9a. Figure 9b shows and compares the load-deflection behavior of similar stack pattern beam specimens (S20N and S20H).
As can be found in Figure 9a, the maximum load capacities for the running bond beam specimens, R20N and R20H, were found to be 150 kN and 228 kN, respectively. Hence, the specimen R20H which was built with high-strength grout exhibited 34% higher capacity than its counterpart running bond beam specimen built with normal strength grout, R20N and this difference is significant. It is worth noting that the difference in the strengths of two grouts is 197%. Hence, this study shows that increases in grout strength increases the strength of the running bond masonry beam, however, the increase in the strength of the beam is not proportional to the increase in grout strength.

Figure 9b shows the load-deflection behaviors obtained from beam specimens, S20N (stack pattern beam with normal strength grout) and S20H (stack pattern beam with high strength grout). As can be found from this figure, the maximum load carrying capacities for these two specimens were found to be 164 kN and 230 kN, respectively (Figure 9b). Hence, the difference in the maximum load capacities between these two beam specimens is about 29%. This difference is less than the difference in strength (34%) obtained from counterpart running bond beam specimens, R20N and R20H. Hence, the effect of grout strength on the
maximum strength of stack pattern beam is less than that for running bond beam. Further, the maximum strength of stack pattern beam increased by only 29% when the strength of grout increased by 197%. Hence, this study confirms that the increase in the strength of stack pattern beam due to increase in grout strength is not proportional.

Thus, this study concludes that despite the significant difference in the strength (197%) between the high strength and normal strength grouts, the increase in maximum load carrying capacity of the beam specimen can only be 34% and 29% for running bond and stack pattern constructions, respectively. This is due to the fact that there are other elements such as mortar, block, and steel rebar and they all share the load as a composite construction.

Failure of two running bond beam specimens (R20N and R20H) was due to formation of wide flexural cracks near mid-span. At the time of failure, the crack width measured by DIC was about 4 mm (Figure 10). For specimens constructed in stack pattern, S20N and S20H, same mode of failure was observed as well (Figure 11).
2.3.3 Effect of construction pattern

Load-deflection behaviors of all beam specimens built with 20 cm block units are shown in Figure 12. The beam specimens built with normal strength grout and beam specimens
built with high strength grout are separated in two groups as can be found in Figures 12a and 12b, respectively. Figure 12a shows that two beam specimens built in running bond and stack pattern constructions using normal strength grout (R20N and S20N) exhibited maximum load capacity of about 150 kN and 164 kN, respectively (Figure 12a). Hence, the difference is the maximum load carrying capacities is about 9%. Therefore, this study found that stack pattern beam (S20N) performed slightly better than its counterpart running bond beam (R20N) when normal strength grout was used. However, the difference in their performances is not much.

Figure 12b shows the load-deflection behaviors of stack pattern and running bond beams

![Load-deflection behaviors of stack pattern and running bond beams](image)

(a) R20N and S20N  (b) R20H and S20H

Fig. 2.12. Load-deflection behaviors of stack pattern and running bond beams

Figure 12b shows the load-deflection behaviors of stack pattern and running bond beam specimens grouted with high strength grout (R20H and S20H). These two beam specimens also showed similar behavior as their counterpart beam specimens (R20N and S20N) constructed with the normal grout strength (Figure 12a). The maximum load carrying capacity for R20H and S20H were 228 kN and 230 kN, respectively. Thus, the difference in their load carrying capacities is less than 1%. Hence, Figure 12b shows that construction
pattern (running bond versus stack pattern) has no effect on the structural performance of masonry beam when built with high strength grout.

All these four beam specimens failed due to formation of large flexural crack width and compression failure (crushing at the top) and at that stage the beam specimens became very unstable and unsafe and the tests were discontinued. Hence, this study found that the stack pattern construction has insignificant beneficial effect on the behavior of masonry beam when normal strength grout is used. The effect of construction pattern (running bond construction versus stack pattern construction) is negligible when high strength grout is used.

2.4 Ductility Index

In this study, the method suggested by Priestley and Park [18] was used for calculating the ductility index of each beam specimen. This method is schematically depicted in Figure 13. The ductility index is calculated using Equation 1 as follows.

\[
\mu = \frac{\Delta u}{\Delta y}
\]  

(1)

In this equation, \(\Delta u\) is the displacements at 80% of the maximum load \((0.8H_u)\) as shown in Figure 13. The \(\Delta y\) is the displacement at yield load and it is equal to \(H_u/S\), as shown in Figure 13. \(H_u\) is the maximum load carrying capacity and \(S\) is the slope of the pre-yielding load deflection curve (see Figure 13). As can be found in Figure 13 and Equation 1, the ductility index calculated using this method depends on the slope of the pre-yielding linear part of the load-deflection curve \((S)\) and the maximum (ultimate) load carrying capacity of
the beam specimen ($H_u$). Use of high-strength grout resulted in increase in the maximum (ultimate) load carrying capacity ($H_u$) of a beam specimen as shown in Figure 9. Hence, both $\Delta u$ and $\Delta y$ values increase; however, the increase in the value (the difference in the values) of $\Delta u$ is much higher (about two times) than the increase in the value of $\Delta y$ (Table 4). Hence, beam specimens made with high-strength grout (R20H and S20H) showed higher ductility than their counterpart beam specimens (R20N and S20N) as shown in Table 4. For the similar reason, beam specimens made of wider block units (R30N and S30N) are expected to exhibit higher ductility (see Table 4) since use of wider block units resulted in higher ultimate (maximum) load carrying capacity of the beam (Figure 13).

The ductility values for all beams specimens computed using this method are shown in Table 4. This table shows that the largest difference in ductility between a running bond beam and its counterpart stack pattern beam is 14.3% and this was obtained from beam specimens made of high strength grout, R20H and S20H. As can be found in Table 4, stack pattern masonry beam exhibited a higher ductility index as compare to the similar running bond beam. For beam specimens made of normal strength grout, the differences were found to be 10.5% and 4% for beam specimens made of 20 cm block units and 30 cm block units, respectively and the stack pattern beam showed higher ductility than the running bond beam. Hence, this study found that a stack pattern beam exhibited higher ductility compare to its counterpart running bond beam.
Table 2.4. Ductility ratio of beam specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>80% of Ultimate Load (kN)</th>
<th>$\Delta u$ (mm)</th>
<th>$\Delta y$ (mm)</th>
<th>$\mu = \frac{(\Delta u)}{(\Delta y)}$</th>
<th>Difference in $\mu$ (%)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>R30N</td>
<td>184.0</td>
<td>58.2</td>
<td>24.4</td>
<td>2.4</td>
<td>4.0</td>
</tr>
<tr>
<td>S30N</td>
<td>182.4</td>
<td>52.2</td>
<td>20.7</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>R20H</td>
<td>182.4</td>
<td>37.1</td>
<td>21.1</td>
<td>1.8</td>
<td>14.3</td>
</tr>
<tr>
<td>S20H</td>
<td>184.0</td>
<td>41.2</td>
<td>20.0</td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td>R20N</td>
<td>120.0</td>
<td>24.9</td>
<td>15.0</td>
<td>1.7</td>
<td>10.5</td>
</tr>
<tr>
<td>S20N</td>
<td>131.2</td>
<td>25.8</td>
<td>13.5</td>
<td>1.9</td>
<td></td>
</tr>
</tbody>
</table>

*The difference is between running bond and stack pattern construction

Determination of yield displacement ($\Delta y$) as proposed by Priestley and Park [18] and shown in Figure 13 uses an indirect approach and hence, error may result in the yield displacement ($\Delta y$) value obtained using this approach. Nonetheless, this approach is extremely useful when no other approaches are available for determining the accurate value of the yield displacement ($\Delta y$). In the current study, strain gauges were installed on the steel rebars and thus, strain data were obtained from all the beam specimens. Hence,
accurate values of the yield displacements using these strain data were successfully obtained in this study and reported in Table 5. The ductility indices ($\mu$) calculated using the strain-based yield displacements are reported in this table. This table also shows the ductility indices obtained using the yield displacement values using the approach proposed by Priestley and Park [18]. The difference in the ductility indices obtained using these two approaches (indirect approach proposed by Priestley and Park and strain based approach used in the current study) is reported in the last column of Table 5. As can be found from the last column of this table, yield strain-based approach used in the current study yielded higher ductility indices for all the beam specimens. Thus, this study found that the approach for determining yield displacement ($\Delta y$), recommended by Priestley and Park [18], provides a conservative estimate for the ductility index.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\Delta u$ (mm)</th>
<th>$\Delta y$ (mm)</th>
<th>$\mu$</th>
<th>Difference in $\mu$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Priestley and Park</td>
<td>Current study (Strain gauge)</td>
<td>Priestley and Park</td>
<td>Current study (Strain gauge)</td>
</tr>
<tr>
<td>R30N</td>
<td>58.2</td>
<td>24.4</td>
<td>22.0</td>
<td>2.4</td>
</tr>
<tr>
<td>S30N</td>
<td>52.2</td>
<td>20.7</td>
<td>19.5</td>
<td>2.5</td>
</tr>
<tr>
<td>R20H</td>
<td>37.1</td>
<td>21.1</td>
<td>15.4</td>
<td>1.8</td>
</tr>
<tr>
<td>R20N</td>
<td>24.9</td>
<td>20.0</td>
<td>14.0</td>
<td>1.7</td>
</tr>
<tr>
<td>S20H</td>
<td>41.2</td>
<td>15.0</td>
<td>13.8</td>
<td>2.1</td>
</tr>
<tr>
<td>S20N</td>
<td>25.8</td>
<td>13.5</td>
<td>9.8</td>
<td>1.9</td>
</tr>
</tbody>
</table>

The current study proposed a strain-based approach for determining the yield displacement ($\Delta y$) accurately and the approach uses strain data to be obtained from the test (see Table 5). Hence, the strain-based approach for determining the yield displacement is not a realistic solution for the practicing engineers since it is not feasible to undertake test on every single beam specimen to be built in a construction site. Hence, an alternative approach based on
the sensitivity analysis is also proposed and this approach can be used if strain data is not available. The sensitivity analysis was undertaken to determine the influence of various parameters such as width of block unit (W), strength of grout (S), and maximum load carrying capacity (P) on the yield displacement (Δy). As can be found in Table 3 effect of three parameters (P, S, and W) on the load-deflection behavior of masonry beams was studied. Hence, the following relationship can be written as in Equation 2.

\[ X = F(P, S, W) \] (1)

In above equation, \( X \) is the mid-span deflection of the beam. The test data shows the parameter that affects the mid-span deflection most is the grout strength (S) and sensitivity weight for the grout strength was found to be 0.0983. It is worth mentioning that the sensitivity analysis also found that the effect of the grout is almost twice than that of the block unit size, which had sensitivity weight of 0.0486 (Table 6). It also worth notice that since load (P) is an inseparable part of any experiment, it is not considered as a separate design parameter. Furthermore, the relation between load and deflection must be linear since increase in load directly affects the deflection as shown in Table 6.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Sensitivity</th>
<th>Positive Magnitude</th>
<th>Negative Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>P</td>
<td>1.0353</td>
<td>1.0353</td>
<td>0</td>
</tr>
<tr>
<td>S</td>
<td>0.0983</td>
<td>0</td>
<td>0.0983</td>
</tr>
<tr>
<td>W</td>
<td>0.0486</td>
<td>0</td>
<td>0.0486</td>
</tr>
</tbody>
</table>
The data obtained from six masonry beam specimens was analyzed for deriving the empirical relationship (Equation 3). This relationship can be used for predicting the yield displacement ($\Delta y$) of both stack pattern and running bond masonry beams. Only simple algebraic operators were used in deriving this relationship to minimize complexities. The accuracy in the prediction of the mid-span deflection is good since $R^2$ value was found to be 0.98.

$$X = \Delta y = 0.1P + 4.261 \times 10^{-4}P^2 - 6.242 \times 10^{-4}PS - 2.233 \times 10^{-4}WP^2 - 0.3384$$

(3)

In above empirical relationship, $P$ is the design ultimate load in kN, $S$ is the strength of the grout in MPa, and $W$ is the width of the masonry block unit normalized by 20 cm unit.

### 2.5 CONCLUSIONS

The conclusions presented below are made based on the full-scale tests and sensitivity analysis completed under the scope of this study. Therefore, these conclusions may have limitations associated with the specific masonry beam specimens used in this study.

1. The grout strength has the largest effect on the maximum load or moment carrying capacity of a masonry beam. The sensitivity analysis also confirmed this. The strength of high-strength grout used in this study was about 200% higher than the strength of normal-strength grout. However, the increase in the maximum load carrying capacities of the beam specimens made with high-strength grout was only 29% to 34% higher than the maximum load carrying capacities obtained from similar beam specimens.
made of normal-strength grout. Hence, the increase in the strength of masonry beam due to increase in grout strength is not proportional.

2. 30cm block units have larger area compare to 20cm block units which decreases the depth of the neutral axis and increase the lever arm that creates higher bending capacity in beams constructed using 30cm block units. The NA depth of 30 cm wide beam (R30N) is about 25-30% smaller than the counterpart 20 cm wide beam (R20N). At the ultimate load capacities, this difference is about 28%. Thus, a masonry beam built with larger units also shows similar increase in the load carrying capacity as compare to the masonry beam made of smaller size units.

3. The study found that there is a negligible difference in load-deflection behaviors and modes of failure exhibited by stack pattern and running bond masonry beams. Both stack pattern and running bond beams failed in flexural mode and the maximum difference in the ultimate load or moment carrying capacities of these specimens was only about 9%. Hence, this study found that in general, both stack pattern and running bond beams performed same way. In general, a stack pattern masonry beam performed slightly better than its counterpart running bond beam.

4. Specimens constructed with high-strength grout exhibited more ductile behavior compare to beam specimens built with normal-strength grout. Stack pattern beams exhibited higher ductility compare to its counterpart running bond beams. Beam specimen constructed with wider (30 cm) block units provided the higher ductility index than narrower block unit (20 cm).

5. Ductility indices computed using the method recommended by Priestley and Park were found to be conservative. This is due to the fact that an approximate approach is used
for determining the yield displacement. In the current study, yield displacement was accurately determined using strain data. These data were used in developing an empirical relationship for accurate estimation of the yield displacement. Since the empirical relationship was derived from the yield load obtained using the strain data, the results are more accurate compared to the approach proposed by Priestley and Park.

2.6 ACKNOWLEDGEMENTS

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2.7 REFERENCES


CHAPTER 3
STRUCTURAL PERFORMANCE OF STACK PATTERN
CONCRETE MASONRY BEAM

3.1 INTRODUCTION

Loadbearing concrete masonry construction in North America is by default designed assuming a 50% running bond pattern of units (Figure 1a) according to the TMS 402 [1] and CSA S304 [2] design standards, unless otherwise noted. Stack pattern (or stack bond) masonry (SP) (Figure 1b) may be substituted in for running bond (RB) in some structural elements, such as out-of-plane walls or shear walls. However, the assemblies which contain SP typically have more restrictive prescriptive design requirements for their reinforcement detailing due to the reduced ability to transfer forces across the uninterrupted head joints of the units. Despite the dearth of any research in the area to support either position, TMS 402 [1] and the CSA S304 [2] define these prescriptive requirements for loadbearing SP masonry quite differently: with the former being generally more liberal in its acceptance and the latter being more restrictive. The crux of the issue that clouds the design world

![Fig.3.1. Running bond and stack pattern constructions](image)
today arises due to the fact that the masonry design component of a building is held primarily by the architect. It is the architect who may often specify SP for aesthetic purposes not realizing that there are implications for the structural engineer when these walls are loadbearing or otherwise engineered. A persisting issue in Canadian design specifically occurs when a loadbearing masonry wall is specified with SP unit coursing but contains an opening, such as a door or window, necessitating a beam to span over top. The CSA S304 [2] makes wall design possible with SP masonry, albeit more costly, however, the design of a SP reinforced masonry beam element is explicitly forbidden. Interestingly, the CSA S304 [2] permits the use of single course beams, including “high lintel” units laid in soldier course, which by their very nature do not contain any overlapping head joints and would theoretically then behave in the same manner as a multi-course stack pattern beam. However, when engineers are faced with a moderate span (e.g. double doors) or a high design load, a single course masonry beam is not likely to be sufficient. Typically, then, to maintain the architectural look of a continuous SP wall, the use of some type of steel lintel or I-section is required to carry the loads over the opening, adding significant cost and labor beyond that which normally could be designed using an all RB masonry wall system. However, since there is no research to support either the use or restriction of SP in multi-course beams, the status quo remains as reflected by the disparity in the current editions of the TMS 402 [1] and CSA S304 [2], which respectively, permit and forbid its use.

Past experimental research on the effect of unit bonding pattern on the structural behavior of masonry beams or walls is either limited or non-existent. Compared with reinforced concrete, there were a scant 112 concrete block beam tests available in the literature at the
time of the drafting changes to the shear strength equation of the 2014 edition of the CSA S304 (Sarhat, S. R., and Sherwood, T. G.) [3]. Unfortunately, the vast majority of these beams are only 1-2 courses deep and constructed in running bond and therefore, offer no real insight to how stack pattern beams may behave over multiple courses in height where long uninterrupted head joints may affect behavior.

Design standards for masonry have historically adapted results from reinforced concrete testing to help fill in the gaps where there is insufficient masonry-specific data. For instance, an equivalent rectangular stress block concept is used to determine the stress in the compression zone in reinforced concrete block masonry beams, just as adopted in reinforced concrete. Applicability of an equivalent stress block in masonry beams was supported by several previous studies (Khalaf, et al. [4]; Suter and Fenton [5]; Hamid et al. [6]). Due to the anisotropy of masonry, the development of the equivalent stress block for masonry beams in the CSA S304 [2] accounts for a reduction to the compressive strength of concrete block masonry when compression forces act normal to the head joint (as opposed to normal to the bed joint as in walls) supported by (Lee et. Al [7], Wong and Drysdale [8], Kalaf [9] and Drysdale and Hamid [10]). However, more recent research has suggested the concrete block loaded normal to the head joint may actually experience an increase in strength compared to the normal to the bed joint Ring et a. [11]. Historically, masonry beams have been constructed of stretcher units which had mortar slush filled into the small gaps created by abutting frogged ends of concrete block units which was often left uncompacted, or potentially even unfilled. It is believed that it was due to this construction method for which a reduction to the compressive strength of masonry loaded normal to the head joint was originally derived from. Whereas, the use of units with knock-
out webs or special “A” or “H” shaped blocks eliminates this issue and ensures a greater
degree of grout continuity. There clearly remains a significant gap in the knowledge
surrounding masonry beam behavior, and reinforced concrete beam testing is unable to
address these deficiencies and provide any reasonable guidance. The unique response of
masonry beams is derived from the anisotropic nature of masonry construction, therefore
necessitating new experimental testing to address the structural effects of unit bonding
pattern as it pertains to ultimate strength.

The main structural concern with SP masonry beams is the unrestricted propagation of
flexural cracks through the continuous head joints which are not interrupted by the
relatively strong face shells of units in a RB beam configuration. Crack location, crack
width, and the crack development pattern in a masonry beam during its loading is an
indication of how it will perform at serviceability and ultimate limit states of design.
Essentially, the wider the opening of cracks that develop during loading, the weaker the
beam will become. The TMS 402 [1] requires reinforcement to span the head joints of stack
pattern masonry to inhibit crack propagation and by effect improve shear bond across the
head joint. Some other methods used to reduce excessive cracking include use of fiber
reinforced polymers (FRP) (Lee et al. [12], Franco and Royer-Carfagni [13], Mazzotti and
Murgo [14]), use of skin reinforcement (Frosch [15]), use of intermediate steel (Ring et al.
[16]), and use of lightweight masonry block (Sousa et al. [17]). Some studies used bamboo
as the reinforcement (Moroz et al. [18]). Other research developed a new system of
reinforcement in concrete to resist vertical bending (Omikrine et al. [19]). Finally,
lightweight masonry mortar was introduced as an economically viable alternative (Muñoz
et al. [20]). However, there is no experimental data related to the effects of crack
propagation on SP masonry beams and their corresponding behavior under serviceability and ultimate loads.

To address the need for new experimental research on both the ultimate strength and serviceability response of SP masonry the following experimental program was carried out. This experimental program is a part of an ongoing study at the University of Windsor on the behavior and structural response of stack pattern masonry construction to harmonize the 2014 CSA S304 [2] masonry design standard with modern construction practices and experimentally observed masonry performance. The database of concrete block masonry beams available is predominantly focused on running bond or single course configurations.

To address this, beams of up to four-courses will need to be tested to reflect typical requirements for large openings such as a double doors and garage openings. The anisotropy of masonry units as well as the dimensional limitations for reinforcement placement precludes any comparison with the much more extensive database of reinforced concrete beams. Furthermore, the lack of a unified understanding of masonry flexural behavior, especially the effects of compressive forces acting normal to the head joint, demands that a direct comparison be made with running bond concrete block masonry beams to provide relevant context. Finally, in anticipation of the potential for large cracks to develop through the uninterrupted head joints of stack pattern masonry, the use of concrete block units with depressed or ‘knock-out’ webs (at least 50% the unit height) will be used throughout the beam. The use of such units are typical in modern fully-grouted and reinforced masonry construction and will permit the maximum flow and continuity of grout over the head joints. The following section describes in detail the experimental program.
3.2 EXPERIMENTAL PROGRAM

A total of 12 full-scale concrete block masonry beam specimens were tested under four-point loading with details of the beams are provided in Table 1. Each of the beams was designed with relatively large span of 4.5 m (177.16 in.) using version 2.2 of MASS™ design software which uses the 2004 edition of CSA S304 [2]. Reinforcement detailing, blocks, mortar, and grout were all specified based on typical construction practice.

Table 3.1. Test matrix for beam specimen

<table>
<thead>
<tr>
<th>Beam Type No.</th>
<th>Name</th>
<th>L/H Ratio</th>
<th>H mm (in.)</th>
<th>Construction Pattern</th>
<th>Stirrup (Total)</th>
<th>Bottom Rebar</th>
<th>Top Rebar</th>
<th>Inter Rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-course</td>
<td>1</td>
<td>RN2</td>
<td>11.5</td>
<td>390 (15.4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>RY2</td>
<td></td>
<td></td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
<td>1 - 10M</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>SN2</td>
<td></td>
<td></td>
<td>None</td>
<td>2 - 10M</td>
<td>None</td>
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<td>SY2</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-course</td>
<td>5</td>
<td>RN3</td>
<td>7.7</td>
<td>590 (23.2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>RY3</td>
<td></td>
<td></td>
<td>10M @ 200 (24)</td>
<td>2 - 10M</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>7</td>
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<td></td>
<td></td>
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<td>2 - 10M</td>
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<td>None</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>SY3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-course</td>
<td>9</td>
<td>RN4</td>
<td>5.7</td>
<td>790 (31.1)</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>10</td>
<td>RY4</td>
<td></td>
<td></td>
<td>10M @ 200 (24)</td>
<td>1 - 20M</td>
<td>1 - 10M</td>
<td>1 - 15M</td>
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<td>11</td>
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<td></td>
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<td>1 - 20M</td>
<td>1 - 10M</td>
<td>1 - 15M</td>
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<tr>
<td></td>
<td>12</td>
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<td></td>
<td></td>
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</tr>
</tbody>
</table>

All the beams were 4800 mm (189 in.) long with a span (L) of 4500 mm (177.16 in.)

Beams corresponding to two, three, and four-courses tall were specified and constructed to heights of 390 mm (15.4 in.), 590 mm (23.2 in.), and 790 mm (31.1 in.), respectively. The bonding pattern of the concrete block is denoted as either running bond (R) or stack pattern (S) in Table 1. Beams were designed to be flexural or shear governed using the MASS™ software, and hence, some beams were designed with 10M (100 mm², 0.155 in²) single-leg shear stirrups in every cell (spaced at 200mm, [8 in.]) denoted as (Y) or without any shear reinforcement, denoted as (N). Thus, each beam specimen is given a unique name to
identify its main attributes in Table 1. The first letter (R or S) indicates if the beam was built with running bond (R) or stack pattern construction (S). The second letter (Y or N) denotes whether the beam had shear reinforcement (Y) or not (N). The final character of the name is a number and this indicates how many courses the beam had. For instance, beam specimen SN3 was constructed in stack pattern construction (S), did not have any shear reinforcement (N), and it was three course high (3). Similarly, specimen RY4 was constructed in running bond (R), had shear reinforcement (Y), and it was four-course high (4).

The beams were made of regular two-cell stretcher units (Figure 2a). However, the bottom course of the beams was made of lintel block units (Figure 2b) to facilitate the placement of main flexural rebar as is common practice. The upper courses were constructed from blocks which have knock-out webs (Figure 2c), such that a mason can remove a portion of the webs with a hammer on a jobsite so that approximately 50% of the web remains to facilitate intermediate reinforcement when required and allow continuity of grout (Figure 2c). However, the main reason of using units with knock-out webs was selected specifically

![Fig.3.2. Schematics of various block units used in this study](image)

(a) Stretcher block  (b) Lintel block  (c) Knockout block
to facilitate grout continuity across the head joints throughout the beam and as such the results of this study are limited to stack pattern beams which contain stretchers with no more than 50% of their webs intact throughout the beam. Standard 20 cm (8 in.) concrete masonry units with dimensions of 390 mm (15.4 in.) long, 190 mm (7.5 in.) wide, and 190 mm (7.5 in.) tall as per CSA-A165 [22] were selected. Each beam specimen was 4.8 m (189 in.) long with an effective span of 4.5 m (177.2 in.) and a width of 190 mm (7.5 in.) as indicated in Figure 3.

A pre-bagged type S mortar and ready-mix fine grout were used in accordance with the performance specification of the Canadian standard CSA A179 [23].

All stirrups (shear reinforcement) in the beams were made of 10M rebar and placed in every cell resulting in a spacing of stirrups was 200 mm (8 in.). The main flexural reinforcement in two and three course high beams consisted of two 10M bars. Any beams with stirrups also contained an additional 10M bar in the top course to allow for anchorage of the stirrup. For the four-course high beams, flexural reinforcement consisted of one 20M rebar in the first course, one 15M rebar in the second course, and a 10M rebar in the top course.
course. The cross sectional area of each 10M, 15M, and 20M bars are 100 mm² (0.155 in²), 200 mm² (0.310 in²), and 300 mm² (0.465 in²), respectively. The beams were built in these configurations as an attempt to control their dominant failure modes: either flexure induced failure or shear induced failure based on the design requirements at the time of the CSA S304 [21]. The schematics of the test setup and strain gauge maps for a four-course high beams are shown in Figures 3 and 4, respectively.

![Strain gauge map for four-course high beam](image)

(a) Running bond beam without stirrups (RN4)

(b) Running bond beam with stirrups (RY4)

**Fig.3.4.** Strain gauge map for four-course high beam

A pin-roller boundary condition was used to simulate simply supported conditions typical for design. A 1.3 m (51.2 in.) long steel spreader beam with pin-roller boundary condition was mounted at the top of the beam specimen to facilitate application of four-point bending load (Figure 3). The spreader beam was used to produce a constant maximum moment zone of 700 mm (27.6 in.) at the mid-span. A swivel head was then mounted on the loading beam to ensure verticality of the load being applied (Figure 3). A universal loading actuator was
used to apply monotonically increasing quasi-static load at the mid-span of the beam specimen. Reinforcement details for each beam height are shown in Table 1. It is to be noted that four-course high beams (Figures 4a and 4b) had one 15M rebar as intermediate reinforcement as required by CSA S304 [2].

3.3 INSTRUMENTATION

The test data collected during each beam test included load, deflection, and strains from various reinforcement bars; as well as the pattern of crack formations, crack growth, crack spacing, and crack width. Test data was acquired using a computerized data acquisition system, except for crack related information which were collected manually and also through the digital image correlation (DIC) technique. All beams were tested using the same test setup (Figure 3).

Load data were acquired through the load cell of the loading actuator. For deflection measurements, a total of five 100 mm (4 in.) stroke LVDTs (Linear Variable Displacement Transducers) and two 25 mm (1 in.) stroke LVDTs were used. The 25 mm (1 in.) stroke LVDTs were used to measure longitudinal and out-of-plane displacements. LVDTs 1, 2, and 3 had 100 mm (4 in.) stroke and they were used to measure vertical deflections at third lengths underneath the beam specimen (Figure 3). LVDTs 4 and 5 were attached to the loading actuator and hence, these two LVDTs measured deflection at the top surface of the beam. The behavior of the steel reinforcing bars was monitored with strain gauges which were installed before the beam specimen was built. Two strain gauges were installed on the flexural steel reinforcement bars, and two strain gauges on the stirrups (Figures 4). Two
cameras were used to cover the entire length of masonry beam specimens for digital image correlation (DIC) implementation.

### 3.4 TEST PROCEDURE

The load was applied using the displacement control method. Loading was paused several times and cracks were marked and crack related information was recorded. This was done at increments of about every 5 mm (0.2 in.) of deflection or when significant crack growth was observed. Loading was continued until a clear failure mode in the beam was observed.

Material tests on mortar, grout, concrete block units and rebar were undertaken in accordance with relevant standards CSA A165 [22], CSA-A179 [23], and ASTM C109 [24], respectively. The prism specimens were tested under monotonically increasing compressive load until failure occurred (CSA S304, [2]).

### 3.5 TEST RESULTS

#### 3.5.1 Material Properties

The summary of material properties obtained from mortar, grout, and concrete block unit tests are shown in Table 2. The yield strength of 10M, 15M, and 20M steel bars were obtained at 450 MPa (62.3 ksi), 465 MPa (67.4 ksi) and 495 MPa (71.8 ksi), respectively and modulus of elasticity for all the rebars was about 202 GPa (29x10^6 psi). The specified compressive strength of the fully grouted knock-out stretcher unit (f’m) obtained from stack pattern and running bond prisms were 9.7 MPa (1407 psi) and 10.7 MPa (1552 psi) with coefficient of variation of 11.1% and 10.2%, respectively. All the prisms used in this study
were 190 mm (7.5 in.) wide, 390 mm (15.4 in.) long and 790 mm (31.1 in) (four-course) high and these prisms were fully grouted.

### Table 3.2. Material properties

<table>
<thead>
<tr>
<th>Materials</th>
<th>Beam test-day values</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength, MPa (psi)</td>
<td>C.O.V. %</td>
</tr>
<tr>
<td>Stretcher Units</td>
<td>18.2 (2635)</td>
<td>2.2</td>
</tr>
<tr>
<td>Type S Mortar</td>
<td>16.0 (2320)</td>
<td>8.1</td>
</tr>
<tr>
<td>Fine Grout</td>
<td>22.5 (3268)</td>
<td>7.7</td>
</tr>
<tr>
<td>Reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10M</td>
<td>Yield Stress = 450 (62.3 ksi)</td>
<td>1.3</td>
</tr>
<tr>
<td>15M</td>
<td>Yield Stress = 465 (67.4 ksi)</td>
<td>1.8</td>
</tr>
<tr>
<td>20M</td>
<td>Yield Stress = 495 (71.8 ksi)</td>
<td>1.5</td>
</tr>
</tbody>
</table>

### 3.5.2 Load–Deflection Behavior and Failure Modes

A summary of the beam test results is found in Table 3, the following sub-sections describe the behavior in each series based on qualitative and quantitative observations.

### Table 3.3. Summary of the beam results

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Name</th>
<th>Yield Load (kN)</th>
<th>Difference (%)</th>
<th>Ultimate Load (kN)</th>
<th>Difference (%)</th>
<th>Failure Mode</th>
</tr>
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<td>2-course</td>
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<td>23</td>
<td>4.3</td>
<td>27</td>
<td>11</td>
<td>Flexural</td>
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<tr>
<td></td>
<td>SN2</td>
<td>24</td>
<td>30</td>
<td>30</td>
<td>1</td>
<td>Flexural</td>
</tr>
<tr>
<td></td>
<td>RY2</td>
<td>24</td>
<td>30</td>
<td>4</td>
<td>1</td>
<td>Flexural</td>
</tr>
<tr>
<td></td>
<td>SY2</td>
<td>25</td>
<td>29.7</td>
<td>4</td>
<td>1</td>
<td>Flexural</td>
</tr>
<tr>
<td>3-course</td>
<td>RN3</td>
<td>31.5</td>
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<td>50</td>
<td>2</td>
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<td></td>
<td>SN3</td>
<td>29</td>
<td>51</td>
<td>50</td>
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<td></td>
<td>RY3</td>
<td>30</td>
<td>52.9</td>
<td>7</td>
<td>10.6</td>
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<td>SY3</td>
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<tr>
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<td>158</td>
<td>9</td>
<td>3.2</td>
<td>Flexural</td>
</tr>
</tbody>
</table>
3.5.2.1 Four-course high beams

The four-course high beams had the largest height among all the beam specimens tested and hence, these beams had the lowest slenderness ratio (L/H in Table 1) of 5.7. Hence, the four-course high beams were expected to exhibit the highest load carrying capacity and the smallest vertical deflections. Load-deflection behaviors for all four-course high beam specimens are shown in Figure 5.

![Load-deflection curves for four-course beams](image)

**Fig.3.5.** Load-deflection curves for four-course beams

Failure in both specimens without shear reinforcement (specimens RN4 and SN4) occurred due to the formation of large shear cracks (Figure 6) which resulted in sudden drop in the load carrying capacity (Figure 5). Hence, the test at this stage was discontinued. The maximum (ultimate) load values obtained from these two specimens are 140.6 kN (31.61 kip) and 137.5 kN (30.91 kip), respectively. Hence, the difference in their ultimate load carrying capacity is about 2.2%.
Strain data obtained from top (compression) and bottom (tension) reinforcement in specimen RN4 are shown in Figure 7a. It can be found that for specimen RN4, the bottom rebar (tension) yielded at about 128 kN (28.77 kip) when the strain value in the compression rebar was only -0.06%. The maximum strain in the bottom rebar (tension) was 0.68%. The maximum strain that the top rebar (compression) was -0.09% which is less than the yield stress of steel rebar (~0.2%) and far less than the crushing strain of masonry which is considered to be 0.3% by CSA S304 [2]. Hence, the strain data confirms that this beam did not experience any crushing in the masonry when shear failure occurred. The
crack pattern and crack width data obtained from this beam specimen indicate that the beam experienced considerable flexural deformation before it failed in shear. The maximum flexural crack width was about 12 mm (0.47 in.). Similar behavior was also observed in the stack pattern beam specimen without shear reinforcement, SN4. The bottom rebar of this specimen yielded at the load of 123 kN (27.65 kip) which is only 4% less than its running bond counterpart, RN4 (128 kN [28.77 kip]). The maximum strain value in the top (compression) rebar was only -0.07% which is far less than the crushing strain of masonry. However, the bottom (tension) rebar experienced about 0.72% maximum strain which is much higher than its yield strain (Figure 7b). Hence, the strain data obtained from the beam specimen, SN4 also confirms that this beam did not experience masonry crushing when shear failure occurred and the beam exhibited considerable flexural deformation when shear failure occurred.

For both four-course high masonry beam specimens with shear reinforcement (RY4 and SY4), failure occurred due to flexural induced wide crack formation. Both these specimens experienced small crushing in the compression zone while the tests for both specimens were discontinued (Figures 8a and 8b).

(a) Beam RY4  
(b) Beam SY4

**Fig.3.8.** Failure mode for beams RY4 and SY4
At that time, the maximum flexural crack width was large and it was about 12 mm (0.47 in.). The maximum load carrying capacities of these two beam specimens were 149.7 kN (33.65 kip) and 158 kN (35.52 kip), respectively and hence, the beam specimen, SY4 exhibited 3.2% increase in the ultimate load capacity as compare to the ultimate load capacity of RY4.

The bottom rebar of specimen RY4 yielded at about 100 kN (22.48 kip) load (Figure 9a). At this load, the strain gauge readings in the compression rebar and stirrup were -0.05% and 0.03%, respectively. The maximum strain (at the time when failure occurred) in the compression zone rebar was recorded at -0.25% which is slightly larger than the yield strain of steel rebar (Figure 9a). The steel rebar in the compression zone had 75 mm (2.95 in.) cover and hence, strain value in the top surface of the masonry was calculated at -0.29% which is close to the masonry crushing strain of 0.3% (CSA S304, [2]). Thus, the strain data obtained from specimen RY4 indicates that this beam in the compression face may have crushed. The maximum strain in the stirrup (shear reinforcement) was found to be

![Strain data obtained for RY4 and SY4](image)

**Fig.3.9.** Strain data obtained for RY4 and SY4
0.16% indicating that stirrups were elastically loaded and no shear failure occurred in this beam specimen.

The bottom rebar of specimen SY4 yielded at about 109 kN (24.50 kip) load (Figure 9b) which is 9% higher than its running bond counterpart. At this load, the strain gauge readings in the compression rebar and stirrup were -0.05% and 0.01%, respectively (Figure 9b).

Maximum strain values in tension, compression, and shear stirrups for specimen SY4 were 2%, -0.5%, and 0.28%, respectively (Figure 9b). It is worth mentioning that the strain gauge on the bottom (tension) rebar failed when the load was about 140 kN (31.47 kip) and mid-span displacement was about 21 mm (0.83 in.) (Figure 5). Hence, the maximum strain of 2% found from this strain gauge does not correspond to the strain value at the time when the test was discontinued (158 kN [35.52 kip] load and 56.1 mm [2.20 in.] mid-span deflection). Nonetheless, the maximum strain in the compression rebar obtained at the end of the test (0.28%) does indicate that the top surface of the beam had a crushing failure.

The maximum strain recorded from the bottom (tension) rebar (2%) was high even at the mid-span deflection of only 21 mm (0.83 in.). Hence, the tension rebar strain data obtained from this specimen confirms that the primary reason of failure for this beam was flexure.

The strain value in stirrups was 0.05% which is less than the yield strain of steel reinforcement.

Specimen SN4 showed about 2.2% less ultimate strength than its counterpart running bond beam specimen, RN4 (Figure 5). In contrast, specimen SY4 exhibited about 3.2% gain in its ultimate strength as compare to specimen RY4. These differences are likely attributed due to the nature of inherent variability in the masonry materials given how relatively small they were. Maximum mid-span deflections obtained from specimens RN4 and SN4 at the
time when the loading was discontinued were 28.5 mm (1.12 in.) and 27.5 mm (1.08 in.), respectively and hence, the difference in deformability between these two specimens is about 3.7%. Maximum deflections for beam specimens RY4 and SY4 were recorded at 55.6 mm (2.19 in.) and 56.1 mm (2.20 in.), respectively. Thus, the difference in deformability between these two beams (RY4 and SY4) specimens is about 1.0%. These differences in deformability (3.7% and 1.0%) and as well as the difference in the maximum load carrying capacity (2.2% and 3.2%) between running bond and stack pattern construction are negligible if the range of acceptable variability for masonry material is considered.

Both stack pattern and running bond beam specimens failed same way. The N-group (specimens without stirrups) beam specimens (SN4 and RN4) failed in shear whereas, beam specimens (SY4 and RY4) in Y-group (specimens with stirrups) failed in flexure. Hence, it is evident that both flexural and shear capacities obtained from running bond and stack pattern construction for four-course high beams are similar. Thus, it can be concluded that for four-course high masonry beams, construction pattern has no effect on the strength, deformability, and failure mode.

3.5.2.2 Three-course high beams

Three-course high beam specimens had higher slenderness ratio (L/H) than the four-course high beam specimens as the height of three-course high beams is 25% shorter than the four-course high beams. Hence, the three-course high beams exhibited reduced load carrying capacity as compare to four-course beams. Figure 10 shows the load-deflection behavior of all the three-course high beam specimens. This figure shows that three-course high
beams exhibited larger deformability as compared to counterpart four-course high beams. Hence, the failure in three-course high beam specimens occurred at much larger deflections than similar four-course high beams (Figures 5 and 10).

Both three-course high beam specimens without shear reinforcement (RN3 and SN3 in Table 3) failed due to flexure and hence, at this point tests were discontinued (Figure 11).

Notable shear cracks developed in the N-group beam specimens, however, these cracks did not lead to the failure. These two specimens (RN3 and SN3) exhibited maximum load capacity of about 50 kN (11.26 kips) and 51 kN (11.46 kips), respectively. The difference is only 2%. Hence, this study found that there is no considerable difference in the maximum load carrying capacity obtained from three-course high stack pattern and running bond.
beams without shear reinforcement (RN3 and SN3). The bottom rebar strain data obtained from these two specimens (RN3 and SN3) shows that the difference in the yield load values between these two beams is about 8%.

The maximum strain obtained from the bottom rebar of beam specimens RN3 and SN3 were 1.9% and 2.1%, respectively. Hence, the strain data confirms that these two beams failed in ductile manner. The maximum mid-span deflections experienced by specimens RN3 and SN3 were 40 mm (1.57 in.) and 42 mm (1.65 in.), respectively. Hence, the difference in the deformability is only about 5%. Thus, this study shows that the deformability of these two beams were similar.

Specimens RY3 and SY3, which had shear reinforcement, did not experience shear cracks that can be considered alarming or significant. Any development of diagonal cracks in these two beam specimens was short and superficial and the shear cracks did not grow as loading progressed. Failure of these two beam specimens (RY3 and SY3) was due to formation of wide flexural cracks. The maximum (ultimate) load capacities when the tests were discontinued were found to be 52.9 kN (11.89 kip) and 59.2 kN (13.30 kip) for RY3 and SY3, respectively. Hence, the specimen SY3 exhibited 10.6% higher capacity compare to its counterpart running bond beam, RY3 and this may appear as a considerable difference.

However, it may be premature to claim that three-course high stack pattern beam is stronger than its counterpart three-course high beam with running bond construction. The maximum mid-span deflections obtained from these two specimens are 58.4 mm (2.23 in.) and 60.6 mm (2.38 in.), respectively. Hence, the deformability of these two beams are comparable.

Figure 12 shows the crack pattern after the beam specimen RY3 was completely unloaded. Crack pattern in specimen SY3 after unloading was similar.
These tests were stopped when the width of the widest crack was in between 10 mm and 12 mm (0.4 in. and 0.47 in.). At this stage, the test setup became very unstable and the roller supports looked unsafe.

Figure 13 shows that the specimens RY3 and SY3 yielded when the load was about 30 kN (6.74 kip) and 24 kN (5.4 kip), respectively. At this time, the strain in the stirrups (shear reinforcement) was almost zero. In fact, strain in the stirrup at the time when the test was stopped was only 0.01%. Thus, the strain data indicates that for these two beam specimens (RY3 and SY3), shear mechanism had little effect on the failure (Figure 13).

(a) Strain data obtained from RY3
(b) Strain data obtained from SY3

The maximum strain values at the top rebar (compression steel) during the tests, were found to be around -0.04% for both beam specimens RY3 and SY3. Thus, the strain data indicate
that the crushing in the masonry did not occur in these two beam specimens. It should be
noted that the shear strength provisions found in the 2004 edition of the CSA S304 (CSA
S304, [21]) for which this test program was based on have been changed for the 2014
edition (CSA S304, [2]). The fact that beams which theoretically would have failed in shear
based on the 2004 design equations further supports the changes that were made to update
the 2014 edition of the standard.
Hence, this study found that the deformability, failure mode, load carrying capacity
obtained from three-course high stack pattern beams and their counterpart running bond
beams are similar and this was observed for both N-group and Y-group beam specimens.
Hence, it can be concluded that both running bond and stack pattern three-course high
masonry beams perform with negligible measurable difference.

3.5.2.3 Two-course high beams
The load-deflection relationships for all two-course high beams are shown in Figure 14.
Two-course high beam specimens had highest slenderness ratio (L/H in Table 1) and
therefore, these beams experienced largest mid-span deflections (Table 1).

![Fig.3.14. Load-deflection curves for two course beams](image-url)
The maximum deflections experienced by all four beam specimens were in between 75 mm and 80 mm (3 in. and 3.2 in.). The maximum load recorded from these tests occurred usually at the maximum deflection and the maximum load value ranged from 27 kN to 30 kN (6.07 kip to 6.75 kip). Figure 14 shows that all two-course high masonry beams (RY2, SY2, RN2, and SN2) exhibited almost same maximum load carrying capacity as well as maximum deformability. All these four beam specimens failed due to flexure induced crushing in the compression zone of the masonry irrespective of the fact that beam specimens RN2 and SN2 did not have any shear reinforcement (Figure 15). This happened because the slenderness ratio for these beams was very high (11.5). At the time of crushing failure, the maximum flexural crack widths were in the range of 12 to 15 mm or 0.47 to 0.59 in.

![Image](image.png)

Fig.3.15. Flexural crushing failure in specimen RN2

The failure due to crushing in masonry is also evident from the strain data obtained from these beam specimens. For beam specimens RN2 and SN2, there were no shear reinforcement and hence, strain data for stirrup is not available. However, the maximum strain obtained from the bottom (tension) reinforcement was 2%. For beam specimens with shear reinforcement (RY2 and SY2), there was a considerable difference between the shear
and flexural strain values. For beam specimen RY2, at the time of failure and when the tests were discontinued, the maximum strain value in tension rebar was 4% and the strain in the stirrup was only 0.03%. These strains for specimen SY2 were 3% and 0.025%, respectively (Figure 16).

![Comparison of Strain Values](image)

(a) Compression and shear reinforcement

(b) Tension reinforcement

**Fig.3.16.** Strain value for specimen SY2

The maximum strain values in beams without shear reinforcement (RN2 and SN2) are comparable with the maximum strain values obtained from specimens with shear reinforcement (RY2 and SY2). Physical evident also confirmed that these two beam specimens (RN2 and SN2) failed same way as other two beam specimens (RY2 and SY2) failed.

This study, therefore, found that both stack pattern and running bond two-course high beams exhibited similar deformability at failure. This study also found that the specimen SN2 showed an increase of 11% in ultimate strength if compared with its counterpart beam specimen with running bond construction, RN2. Further, the stack pattern two-course high beam specimen with shear reinforcement (SY2) exhibited about 1% increase in the ultimate
strength over its counterpart running bond beam specimen (RY2). Though the difference in load carrying capacity in N-group beam is higher (11%) than Y-group beams (1%), it may be premature to conclude that two-course high stack pattern masonry beam performs better than similar running bond beam. Such differences can be due to the nature of inherent variability in masonry. Nonetheless, based on the current study, it can be concluded that the two-course high stack pattern masonry beam is not inferior to its equivalent masonry beam with running bond construction.

3.5.3 DIC and Crack Data

DIC technique was employed for all the beam specimens to obtain the crack widths and crack patterns at various load and deflection levels. Cameras used for collecting the images were removed at past-yield load and much before reaching the failure load to avoid any possible damages to the cameras. Only the test data obtained from running bond beams are presented in the paper since the crack patterns and crack widths between a running bond beam and its counterpart stack pattern beam were similar as shown in Figure 17 for the two-course high beams (RN2 and SN2).

![DIC images](image)

(a) Flexural cracks for RN2 (mid-span)  
(b) Flexural cracks for SN2 (mid-span)

**Fig.3.17.** Crack formation at yield load for SN2 and RN2 beam
Crack formation at the yield load (rebar strain of 0.2%) for RN2, RN3, and RN4 beam specimens are shown in Figure 18a, 19a, and 20a, respectively. Failure of RN2 and RN3 beam specimens were due to formation of large flexural cracks, followed by crushing of the concrete at the top of the beam. Hence, at the yield load, flexural cracks were dominant. Hence, DIC data for the mid-span of the beam specimens RN2 and RN3 are presented (Figures 18 and 19).

![Mid-span of Reference Line](image)

(a) Horizontal Strain (e_{xx}) contour

(b) Horizontal displacement profile on reference line

**Fig.3.18.** Crack formation at yield load for RN2 (mid-

However, failure of specimen RN4 was due to formation of large shear cracks and hence, the DIC data for left-half of the entire beam span is presented in Figure 20. Figure 20a shows that at yield load, the shear cracks in beam specimen RN4 began to form.
The horizontal displacements at various points along the reference line (which is drawn at the level of flexural rebar) of a beam specimen was measured using DIC images. These values of horizontal displacement are shown in the vertical axis in Figure 18b, 19b and, 20b for RN2, RN3, and RN4 specimens, respectively. These horizontal displacements were determined from the DIC data and the horizontal displacements were then used to determine the crack width at a particular location on the reference line of a beam specimen. The maximum flexural crack widths at the yield load for specimens RN2, RN3, and RN4 were found to be 0.7 mm (0.03 in.), 0.75 mm (0.03 in.), and 0.8 mm (0.03 in.), respectively. The maximum flexural crack widths at service loads were found to be 0.6 mm (0.02 in.), 0.65 mm (0.02), and 0.7 mm (0.02), respectively. In this study, service load is considered
as 60% of the yield load. For both beam types (running bond and stack pattern), the crack widths at service load are well within the acceptable limit of serviceability requirement of current Canadian standard, CSA S304 [2]. Hence, this study found that the crack width and crack patterns among all these three beams are similar. Further, the crack patterns and crack widths between the running bond beam and a similar stack pattern beam were similar.

![Image of crack formation and displacement profiles](image)

a): Horizontal Strain (εxx) contour

(b) Horizontal displacement profile on reference line

**Fig.3.20.** Crack formation at yield load for RN4
At the service load, vertical cracking in the mortar joints for the stack pattern beams were more evident than the cracks in the running bond beams. However, these cracks could only be identified after the tests were completed and when DIC data were analyzed. Cracks at the service load became longer and denser (closely spaced) as the slenderness of the beam increased. Further analysis revealed that the presence of stirrups has increased the spacing and decreased the height of the flexural cracks development at the service load.

Crack patterns at the ultimate load were similar for both construction patterns. Cracks in the vertical mortar joints (head joints) developed quickly in both beams due to debonding of the mortar from the concrete block at the bottom most course. Since the vertical mortar joint in the running bond beams are interrupted by the blocks at every other course, these beam specimens (R-group beams) experienced shorter initial cracks than the stack pattern beam specimens (S-group beams). As the load increased, cracks penetrated into the grout and consequently, the crack patterns (spacing, height, and width) for the two beam types gradually converged.

### 3.5.4 Comparison to S304 Design Equations

Beam specimens were detailed according to the design equations provided by the CSA S304 [2]. The Canadian design standard relies on an equivalent stress block for masonry subjected to bending stresses in manner modeled off that adopted for reinforced concrete design in accordance with CSA A23.3 [25]. Historic testing of masonry beams and assemblages suggested that when compressive forces were orientated normal to the head joint (as in a beam) a reduction to the compressive strength capacity of the masonry was
warranted, currently defined as the modification factor $\chi$ in the CSA S304 [2] which is multiplied by the compressive strength $f' m$. The reduction factor $\chi$ is equal to 0.5 when the compressive forces are normal to the head joint and the grout is not horizontally continuous within the compression zone (i.e. it is interrupted by webs of the units). The factor $\chi$ is increased to 0.7 when grout is horizontally continuous within the compression zone, such as when units with a knock out web are used. The rationale for the factor $\chi$ at the time of its introduction was likely due to the fact that mortar compaction in head joints is typically less than that of bed joints and the common practice for beam construction was to use stretchers with full unit height webs and slush fill the small space between units with mortar since it was difficult for grout to penetrate. The use of this reduction factor in modern concrete masonry beam construction is questionable when units with reduced webs are used. In an effort to further explore the merits of the $\chi$-factor in beam design, the moment resistance from the flexurally failing beams tested in this study are compared to the CSA S304 [2] code predicted strengths in Table 4.

<table>
<thead>
<tr>
<th>Beam Type</th>
<th>Observed</th>
<th>Theoretical (CSA S304)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Course</td>
<td>27.7</td>
<td>15.6</td>
<td>24.4</td>
</tr>
<tr>
<td>3-Course</td>
<td>50.6</td>
<td>29.2</td>
<td>42.4</td>
</tr>
<tr>
<td>4-Course</td>
<td>146.2</td>
<td>69.6</td>
<td>129.9</td>
</tr>
</tbody>
</table>

In the first column of the table, the series of beam is listed as being either the 2, 3, or 4-course high beams. The next column is the observed moment resistance for each series of beams are based on the average of all beams that had an observed flexural failure mode.
The flexural strength of the beams based on the CSA S304 [2] is then provided. To match the strength that a designer would arrive at default values of masonry strength ($f_m' = 10.0\text{MPa, [1.45 ksi]}$), yield strength of the reinforcement ($f_y = 400\text{MPa, [58 ksi]}$), material reduction factors ($\phi_m = 0.6, \phi_s = 0.85$), and the recommended $\chi$-factor (determined to be 0.5 for each beam) were all used. The S304 approach predicted strengths (see column 3 of Table 4) that were 43.6%, 42.2%, and 52.3% below the overserved capacities for the 2, 3, and 4-course beams, respectively. In the final column, the CSA S304 design equation is used adopting the actual average material properties of the beam presented previously and assigning $\phi_m$, $\phi_s$ and $\chi$ equal to 1.0. This approach more closely predicted the actual observed moment resistance, with strengths still below that observed by 11.8%, 16.2%, and 11.1% for the 2, 3, and 4-course beams, respectively. Based on these observations, it can be concluded that a more accurate means to estimate the actual flexural strength of these masonry beams would be to assume $\chi = 1.0$ and that additional study is warranted regarding the applicability of the $\chi$-factor to other types of beam construction.

3.6 CONCLUSIONS

The conclusions presented below are made based on the results obtained from the current study and hence, these conclusions may be limited to the scope of this study. It is important to know that in this study all the beams were made of stretcher units with knock-out or otherwise reduced webs such as to permit continuous grout across the head joints equal to at least 50% of the unit height.

1) Structural performance of reinforced masonry beams made with stack pattern unit coursing did not deviate much from that observed and measured from its counterpart made
of 50% running bond construction. This conclusion is valid even when the slenderness ratio is changed and thus, when the dominant mode of failure changes between shear and flexure.

2) The load-deflection behavior, load carrying capacities, and failure modes of both running bond and stack pattern beams were found to be similar. As well, the crack patterns and crack growths of these two beam construction methods were similar enough to warrant the use of stack pattern masonry beams for loadbearing applications so long as the webs of units are reduced to sufficiently permit 50% grout continuity.

3) The DIC technique employed was found to be an effective and accurate method for monitoring and determining the crack pattern and crack width at the various load and displacement levels on masonry beam specimens. The DIC data showed that the masonry beams of both construction patterns satisfied the serviceability requirement for the crack width.

4) The theoretical strength of the beams was most accurately predicted assuming no reduction to the compressive strength of the masonry due to loading normal to the head joint, $\chi = 1.0$. Although preliminary, the results here support the notion that the $\chi$-factor should not be applied to the modern masonry construction where the webs are reduced by at least 50% of the unit height and the grout is adequately consolidated to ensure continuity. Hence, this study concludes that the stack pattern reinforced masonry beams can be used as the loadbearing structural elements same way as the running bond reinforced masonry beams are currently used when units with reduced web heights of at least 50% are used throughout the beam.
3.7 ACKNOWLEDGMENTS

This work was completed with the financial assistance from the Natural Sciences and Engineering Research Council (NSERC) and Canada Masonry Design Centre (CMDC). The authors sincerely thank Con-Tact Masonry Ltd. located in Oldcastle, ON for their support and help. Special thanks to Kyle Gerard, many graduate students, Lucian Pop, and Matthew St. Louis for their help in the lab work.

3.8 REFERENCES


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CHAPTER 4

PERFORMANCE OF PP BAND REFROFITTED MASONRY WALL

4.1 INTRODUCTION

Unreinforced masonry (URM) buildings suffered severe damages and collapses during past earthquakes as compared to steel and reinforced concrete buildings [1-5]. Starting from hairline cracks to a total collapse is the extent of failure of these URM buildings existed in seismic active zones across the globe [6]. Low construction cost without requiring much technicality and pleasant aesthetics have compelled the people with relatively low income to choose the URM buildings as their most preferred habitats. The brittle failure of these buildings against lateral loadings (such as earthquakes) leads to a wide range of human casualties and great extent of economic loss. Further, many structures of historical importance which are made of URM constructions also need to be preserved against earthquakes or wind loads. Thus, the technical community are compelled to think over this serious issue, which in turn will not only be a solution for the safe habitat for common people, but also help in preserving historical buildings and other important structures.

Various retrofitting techniques [6] are used to improve the seismic performance of unreinforced masonry. However, composite materials like fiber reinforced polymers (FRP) are often preferred due to its high strength-to-weight ratio and corrosion resistance. On the other hand, higher cost involvement, unique technical expertise required, and non-availability of the composite materials lead to the development of easily available low-cost strengthening and retrofitting techniques those do not require much technical rigor. Use of PP band (Polypropylene) for retrofitting and strengthening of various structures has a
promising potential due to its high tensile strength, waterproofing property, high deformability, low-cost, easy availability, and ability to resist ultra violet rays when coated with mortar [22-23].

Literature review on low-cost strengthening of URM structure found that several researches were carried out using PP band [7-21]. Most of these researches concentrated on improving the seismic resistance of URM structures by increasing the strength. However, study on monotonic load-displacement behavior of URM as well as PP band retrofitted masonry is nearly nonexistent. On the other hand, all the previous researches were carried out with scaled specimens. Hence, the current research was designed and undertaken to determine the seismic performance of URM wall retrofitted with PP band. The study presented in this paper addressed monotonic load-displacement behavior of URM wall specimens with and without being retrofitted with PP band. This was accomplished through full-scale tests on URM wall specimens. The tests were undertaken in the structural engineering laboratory of the University of Windsor, Canada. This paper presents outcomes and evaluates the performance of URM masonry structure retrofitted using PP band.

4.2 EXPERIMENTAL METHOD AND RESULTS
This study was completed using two full-scale tests on masonry wall specimens. The test setup and test specimen are shown in Fig. 1. Two specimens were tested under monotonically increasing displacement controlled load until a failure occurred. The objective was to determine the effectiveness of retrofit technique of URM wall using PP band. Hence, specimen 1 was unreinforced masonry (URM) wall or control specimen and
the specimen 2 was retrofitted using external PP band. The PP band used in this study was 12.7 mm wide and 0.67 mm thick and the breaking strength is 1.23 kN. The boundary condition for both wall specimens were fixed at the base and free at the top. Each wall was 2000 mm (10 courses) high, 1600 mm (four block lengths) long, and 200 mm (one block width) thick. Both wall specimens were cured in room (laboratory) condition. Hollow concrete blocks used in these wall specimens have compressive strength of 18.6 MPa and Type S mortar (CSA S304, 2014) used in these wall specimens has compressive strength of 17.6 MPa.

![Test set-up](image)

**Fig. 4.1.** Test set-up

The load-displacement behavior for both wall specimens are shown in Fig. 2. Such load-displacement behaviors for masonry is extremely rare in the literature, though these curves are important for understanding the structural behavior, failure mechanism, and ductility in seismic action. In Fig. 2, the curve indicated by A, B, C, D, E, and F and solid line is for
the specimen 1 (URM wall specimen) whereas, the curve shown by A’, B’, C’, D’, E’, and F’ with broken line is for specimen 2 (retrofitted specimen). The load data were collected from the load cell attached to the loading jack. The displacement data were collected from the wall at four different heights through four linear voltage displacement transducers (shown as LVDT 1 through LVDT 4 in Fig. 1). LVDT 1 and LVDT 4 were located at 300 mm and 1500 mm above the top of the foundation, respectively. The displacement reported in Fig. 2 was acquired through LVDT 4 which was located at 1500 mm above the top of the foundation. Digital image correlation (DIC) technique was used to monitor crack initiation, crack growth, and strain distributions.

4.2.1 Specimen 1

The test began with the loading at 12:57:48 hours. After two minutes and 50 seconds (at 13:00:38 when applied load was 3.5 kN and displacement was 0.3 mm as shown in Fig. 2), the DIC data showed that there were no strain concentrations in either direction (e_{xx} and e_{yy}) anywhere in the wall specimen and hence, DIC did not detect any cracks in the wall specimen.
However, just after another 10 seconds, that is at 13:01:05 hours, and at load of 3.7 kN and displacement of 0.4 mm (at Point A in Fig. 2), fine horizontal crack was detected by the DIC though this crack was not noticeable by visual inspection. At this stage, the strain value in that area was found out to be in the range of 0.2% and 0.25%.

With further loading, crack opened up very fast and the wall specimen reduced its stiffness (line AB in Fig. 2) and this resulted in increase in the displacement at a faster rate. The DIC data found that the maximum crack width at points A and B are 0.152 mm and 4.2 mm, respectively. The displacement values at these two points measured by LVDT 4 are 0.4 mm and 1.15 mm, respectively. Nonetheless, the load carrying capacity increased from 3.70 kN at point A to 5.75 kN at point B.

The loading continued and the load carrying capacity suddenly dropped by 0.97 kN (point C in Fig. 2) while the displacement increased to 1.77 mm. This happened because of the formation of the vertical cracks near the toe of the wall (Fig. 3). At this stage, wall specimen was not able to maintain the load capacity. This is because at point C, the wall began to
over-turn about the toe and hence, the load carrying capacity slowly reduced until point D. Soon after (point E) the toe of the wall crushed and the specimen became unstable.

Hence, the wall specimen was unloaded and the test was discontinued. This wall specimen, after unloading, separated into two parts, above and below the horizontal crack line and the top part fell on the strong floor of the structural testing lab. Thus, the wall specimen lost its structural integrity (Fig. 4a).

4.2.2 Specimen 2

The load-displacement behavior of specimen 2 is also shown in Fig. 2. Unlike specimen 1, this specimen did not have any change in the stiffness between points A’ and B’ (see line A’-B’). The DIC data showed no sign of strain concentrations. Hence, it is concluded that the PP band retrofit delayed the initiation of the horizontal crack. The DIC data revealed that the first crack occurred at Point B’ and at the load of 5.9 kN and displacement of 0.64 mm. The crack continued to grow in length and width and this has resulted in small drop
of 0.4 kN (see point C’). However, this drop (0.4 kN) is significantly less than the drop that the specimen 1 experienced (0.97 kN). Hence, it is obvious that the PP band started sharing the load at this stage.

With further loading beyond point C’, the load capacity began to increase until point D’ when the load capacity reached 7.0 kN and the displacement was recorded at 12.6 mm. Both the maximum load and the maximum displacement values are much higher than the specimen 1 showing that the PP band not only increased the ductility of the wall, but also resulted in increase in the load carrying capacity. It should be noted that specimen 1 reduced its load carrying capacity beyond point C and completely failed at point D when the displacement was 5.74 mm and load capacity was 4.2 kN.

As the loading continued beyond point D’, the toe of the wall crushed as well and the load capacity dropped to 6.4 kN when the displacement reached 12.9 mm. At this stage, the test was discontinued and the specimen was unloaded. Unlike specimen 1, this wall specimen
maintained the structural integrity after complete unloading (see Fig. 4b). Further, the horizontal crack in specimen 2 occurred in the first course and just above the foundation. However, the crack occurred in specimen 2 in between 3rd and 4th courses.

4.3 SEISMIC PERFORMANCE

Load-displacement behavior of both wall specimens are shown and compared in Fig. 2. The figure shows that the unreinforced masonry wall (specimen 1) exhibited a maximum load of about 5.75 kN and a maximum displacement of 5.74 mm and the specimen separated in two parts along the bed joint between the 3rd and 4th courses. However, the retrofitted wall specimen (specimen 2) exhibited maximum load carrying capacity of 7 kN and the test was discontinued when the displacement reached 12.9 mm. Thus, the test data shows that the ductility capacities for the URM and retrofitted wall specimens are about 2.4 and 7, respectively. Ductility capacity is calculated by taking the ratio of maximum displacement and displacement where yielding like behavior (point B for specimen 1 and point B’ for specimen 2) took place. Hence, the increase in ductility capacity in the retrofitted wall specimen was almost three times if compared to the URM wall specimen. Fig. 2 also shows that the energy absorption capacity is also increased due to the retrofit by PP band. The energy absorption capacity of the retrofitted wall specimen (specimen 2) is more than two times than that of the unreinforced wall specimen (specimen 1). The energy absorption capacity is the area bound by the load-displacement curve. Hence, it can be inferred that severe earthquake may be survived by using masonry with PP band retrofit through absorption of more energy in the post-cracking (inelastic) range. Even if complete
survival may not be possible, at least the large energy absorbing capacity may delay the collapse time allowing the users to safely evacuate.

Further, a closer look at the load-displacement curves (Fig. 2) clearly shows that during post-cracking (inelastic) displacement the unreinforced wall (specimen 1) exhibited gradual reduction in its strength, while the PP band retrofitted wall specimen (specimen 2) maintained a positive gradient. In fact, the gradual strength deterioration exhibited by unreinforced wall (specimen 1) may be due to continuous damage and crack propagation. Finally, this wall specimen could not maintain its structural integrity and separated into two parts. On the other hand, PP band provided a better structural integrity to the entire wall specimen (specimen 2) since this specimen maintained its structural integrity even after completion of the test.

4.4 DIGITAL IMAGE CORRELATION

The digital image correlation (DIC) technique was used on both wall specimens to further monitor the crack initiation, crack growth, failure mode, and lateral displacement during loading. DIC data for the specimen 1 is discussed in this paper. It is worth noting that two digital cameras were used to cover both top and bottom half of the specimen. However, the crack occurred on the forth course (bottom half) of the wall specimen hence, DIC data for the bottom part of the wall specimen is presented and discussed in this paper. As shown in Fig. 5, horizontal crack initiated at area with higher strains located on the fourth course in wall specimen (Point A in Fig. 2). At this stage, lateral displacement (U) of the wall specimen is zero (Fig. 6) and no crack can be observed by open eyes.
With further loading, crack opened up and the load dropped (Point C in Fig. 2). The horizontal displacement contour of wall specimen shown in Fig. 7 at point C. As can be seen in this figure, displacement underneath the crack location is almost zero (0.07 mm). However, lateral displacement above the crack line is about 1.0 mm at the height where LVDT 2 was placed. At this point, the crack opening was about 4.2 mm (Fig. 7).
Load continued to drop as crack width increased further. At the end of the test as shown in Fig. 8, the displacement at the height of LVDT 2 was about 1.6 mm. At this stage, the width of the crack was about 8.5 mm.

Fig. 4.7. Crack opening on the wall specimen 1 at point C

Fig. 4.8. Crack opening on the wall specimen 1 at point D
4.5 SUMMARY AND CONCLUSIONS

Previous studies [7-21] have shown that masonry structure when reinforced with PP band has exhibited greater resistance against earthquake. However, no experimental quantification of such improvement has been documented in the literature. Hence, the current study was completed.

The study compares the load-displacement behaviors of unreinforced masonry and PP band retrofitted masonry walls. The test data were obtained through displacement controlled lateral loading as depicted in the paper. The comparison of load-displacement behaviors revealed that retrofitting of URM wall using PP band enhances the ductility capacity and energy absorption capacity by almost 3 and 2 times, respectively, in comparison to unreinforced masonry wall. This improvement is considerable which enables the structure to avoid collapse at least in moderate earthquake ground shaking. This endeavor as a whole can be a starting point for this retrofit technique scientifically acceptable and practically applicable.

4.6 ACKNOWLEDGMENTS

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4.7 REFERENCES


CHAPTER 5

Application of DIC Technique to Determine Mechanical Properties of Construction Materials and Structures

5.1 INTRODUCTION
The non-contact optical metrology called digital image correlation (DIC) which is also able to extract full-field deformation. It does this by coordinating locations in the images that were taken pre- and post-deformation of a specimen [1, 2]. DIC recent development [3, 4] has led it to become a powerful method for determining full-field deformation, motion. A common non-interferometric optical metrology, DIC has unique benefits, including easy setup for experiments, low environmental requirements, and broad usage. DIC has commonly been utilized for measurement of shape and deformation. This information can be used for characterizing mechanical parameters. Most common subset-based DIC is straightforward. It tracks identical subsets that can be found in the reference picture and the picture that has been deformed to collect full-field displacements. While DIC is easy to use, it has two primary difficulties that may be encountered. The first one is the accuracy of sub-pixels; and the second one is the computational efficiency.

Two-dimensional DIC can be applied in a variety of ways as discussed below.

- DIC can be utilized to quantitatively determine the field of deformation and to characterize the mechanism of deformation of different materials (such as, composites, wood, metals, biological materials, and polymers) that have experienced loading that is mechanical, or thermal [5, 6].
Mechanical parameters can be determined using the computed fields of displacement and strain. Such parameters include Young’s modulus, residual stress, Poisson’s ratio, the thermal expansion coefficient, and the stress intensity factor [7-14]. A precise identification explanation of the elastic properties of materials utilizing two-dimensional DIC has also been studied [15, 16].

The field of deformation that has been computed may be utilized to validate the theoretical analysis and the finite element model (FEM) [16, 17], or to close the difference between simulation, experiment, and theory as well.

DIC depends on a high correlation between intensities of the reference picture and the target pictures. A deformation that is sizeable can result in a decorrelation between the first undeformed picture and the deformed picture. In such a situation, the initial reference picture needs to be substituted with a picture that is intermediate so that the subsequent pictures can be correlated with it later [18]. When a sizeable deformation or an alteration in viewpoint causes deformation, DIC that is incremental is needed to amend the reference picture and to utilize the correspondences in the new picture as points of reference in later DIC computation.

Two-dimensional DIC may only be utilized for determining the in-plane deformation for measuring the deformation of a macroscopic body (for example, industrial items with curved surfaces or structural components), advanced three-dimensional DIC is more effective and perfect for utilized for the three-dimensional profile and measurement of deformation of curved and planar surfaces, as it is not sensitive to displacement that is out-of-plane. As a stereovision calibration method that is highly accurate, three-dimensional digital image correlation is expected to gain increasing usage and attention. Nonetheless,
in quantitative measurement of micro deformations, two-dimensional DIC, mixed with a resolution imaging device that has a high spatial resolution, is expected to continue to be the most effective method.

Determination of the behaviour and mechanical properties of a material requires a technique that is able to measure strains and DIC is one such method. It is a measuring technique that is optical-numerical and full-field. This technique has the potential to accurately determine the in-plane displacement of an object experiencing different types of loading and thus can be done by collecting pictures with a digital charge couple device (CCD) at variety of load steps. Such method has been successfully utilized in different technical areas and a variety of applications [19, 20]. The ability of the technique is to measure fields of displacement that are not homogenous has been discussed [21]. The limitations and accuracy of the displacement measurements have also been examined [22]. The paper presents a solution for implementing digital image correlation (in regards to the subset size) that is cost effective. Literature review found that the size of the subset is crucial to the process of correlation at all times [23,24]. Nonetheless, this study is unique in that the pictures show the images representing the different speckle patterns and simulating the full-scale experiments. This approach offers two main advantages. The first is that cost-effective equipment can be used to run digital image correlation instead of costly industry-level tools, and the second is that displacements and strains measured by digital image correlation can be compared to results obtained using traditional gauges.
5.2 DIC PRINCIPLE

Digital image correlation (DIC) provides one with the ability to qualitatively and quantitatively study the mechanical behavior of various materials when subjected to various loading conditions. Each image shot with a CCD camera relates to a distinct loading step. The camera utilizes a tiny rectangular silicon bit that has been divided into number of arrays of individual cells based on the resolution of the CCD camera. The cells are light sensitive and are also called pixels or photo-sites. Every pixel contains a particular scale value that is grey and ranges from 0 to 255, which is related to the intensity of the light that the tested specimen reflects.

Two pictures of the specimen at varied deformation states are compared utilizing a photo-site and the signature it has in the undeformed picture, and looking for a photo-site in the deformed picture to reach the largest possible similarity function that has been given. The majority of the time, such a function has a basis on the least-squares formula. Signature of a pixel can be anything that differentiates it from other pixel signatures and may be the grey-value of the pixel, the derivatives from the grey value, or the colour of the pixel. In such a situation, the grey value of the pixel is utilized. An individual grey value is not a distinct signature of photo site; therefore, nearby pixels are utilized in practice. These collections of photo sites are referred to as a subset or as a correlation window. The result of the displacement, shown in the center point of the subset, is an average of the pixel displacements in the subset.

The size of the step characterizes the amount of pixels the subset is shifted over in x-direction and y-direction to reach the subsequent result. Size of the subset may vary, for example, 10 x 10 or 30 x 30 pixels. The size of the step may also vary, for example, 5 or 7
pixels. The distinct quality of every signature may only be guaranteed if the outside has a pattern that is isotropic, non-repetitive, and high contrast. Appropriate criteria may be found by a random speckle pattern applied to the surface of the object, or offered by the texture of the specimen materials. Potential matches at a variety of locations are examined and a score of similarity, or a correlation function, is utilized for grading them. A correlation function is the sum of the squared differences of the pixel values. The image correlation technique facilitates the finding of each subset of the first picture in the deformed picture. The software then decides the values of displacement of the centers of the subsets, which yield an entire field of displacement. Figure 1 shows the sequence of shooting an image of an object both pre- and post-loading, saving the pictures in a computer using a frame grabber, correlating the images (that is, finding the distinct undeformed subsets in the picture that is deformed), and finding the correlating displacement of the centers of subsets, which results in the field of displacement in the region of interest (ROI).

![Fig. 5.1. Schematic of the DIC setup](image)

**5.3 DIC PROCESS**

As discussed earlier, the measurements depend on some variables: a) the image resolution (total number of pixels which is equal to number of the pixels in columns times number of
pixels in rows), b) dimensions of the specimen (width and height), c) the distance between the specimen and the CCD camera which is related to the focal length of the camera. The DIC requires a five-step process as discussed below.

Step 1. Image sample preparation

The DIC process starts with the creation of patterns for a two-dimensional (2D) image or three-dimensional (3D) image. In 2D image a speckle pattern or surface texture may be used to prepare the image sample. This is because the DIC process is heavily dependent on the high contrast created by the random pattern that is applied on the surface of the specimen object. The preparation must ensure that the pattern is not too sparse, dense or use speckles that are too large as these will affect the spatial resolution of the measurement. Also, if the speckles are too small, they will cause aliasing and thus, causing noisy data during measurements [25]. In general, a good speckle pattern is the pattern that has enough black dots with different sizes. The nature of DIC algorithm requires the dots to be generated with a random pattern. At least three different sizes are needed to have the acceptable pattern (Figure 2).

**Fig. 5.2.** Speckle Pattern on Masonry Prism Specimen
Step 2. Acquiring reference image

This is the object image before the sample is subjected to the deformation or strain factors. The object of this image taken before the beginning of the processes is used in the comparison process. The reference image contains the original positioning of the correlation points for all subsets of the random pattern. The reference image will, therefore, serve as a useful component in the determination of the transformations that occur as a result of the deformations applied on the target object. The reference image also serves as a documentation tool indicating the nature of the random pattern used for this particular DIC process [26].

Step 3. Applying deformation or motion

In the laboratory environment, the target object is subjected to deformation or motion forces that the objects are likely to experience in a realistic environment. Different samples of the same object are subjected to compression, rigid body motion, tensile and shear forces. Compressive, tensile, and shear forces facilitate the observation of the deformation characteristics of the object sample. The object sample may be subjected to rotatory motions such as a 2-degree rotation. The deformed object image is then captured using either static or dynamic imaging equipment but similar to the one used in capturing the reference image [27].

Step 4. Defining correlation points

This process establishes a subset around each point in the random pattern and thus forming a grid of correlation points. Each of these subsets must contain a unique pattern that will be easily identifiable in the deformed image of the target object. Proper setup of the correlation points and subsets will serve as an important factor in the quality and accuracy
of the correlation functions that will be derived from the comparison operation between the reference image and the deformed image. The pattern selected in each subset of the deformed image must be easy to search for and locate in the deformed image [28].

Step 5. Calculating transformation

This includes the calculation of the transformation that maps each reference image subset onto the distorted image subsets with the most optimal correlation. For deformation mapping, the coordinates of the reference image grid points are mathematically related by the translations that appear between the reference and deformed image subsets. Some of the considerations while determining the transformation is the magnitude of the deformation and the direction of such a deformation in relation to the camera optical axis.

This method of determining the transformation results in a description of the displacement field. The initial solution which is in the displacement field is subjected to a differentiation process to determine the strain field. [29].

DIC method should be validated prior to utilizing in full scale experimental tests. In order to validate the DIC method, tensile test was conducted on steel coupon using both DIC and traditional extensometer. The obtained results are discussed below.

5.4 DIC VALIDATION

The accuracy of DIC was validated during the tension test of a steel coupon before utilizing DIC in full scale experiments (Figure 3). DIC technique requires a huge amount of precision. A Cannon T5 SLR Camera was used in the verification process. The comparison was done by extracting data from an extensometer and comparing it with the data obtained from DIC post-processing analysis. The analysis was done using VIC-2D version 2009
software produced by Correlated Solutions. The region of interest (ROI) was defined in the same span as the 50 mm extensometer (Figure 4).

Several strain measurement techniques were used in VIC software to accurately simulate the behaviour of the tensile coupons. The mean and median strain accurately predicted the initial behaviour of the coupon. However, they did not properly represent the plastic behaviour since there was no possible way to verify it. However, by mounting a virtual extensometer that is a tool of VIC, it was found that both the elastic and plastic strains were consistent with that of the physical extensometer.

Figure 4 depicts the correlation between the virtual extensometer of VIC (shown by dots) when compared with the actual instrument measurements (black line). A good correlation is found.
The rate of loading was 2.5 mm/sec (ASTM E8/E8M [31]) and the time-lapse for taking pictures was set to 5 seconds. As shown in the Figure 4, error in the elastic past of the curve is almost zero since the scale is 1/1000 of a millimeter. In the elastic-plastic part, there is almost no change between the data obtained from the virtual and actual extensometers. The last part of the graph where the load started to drop, less than 2% error was recorded, which is in the acceptable range.

5.5 DIC APPLICATION

5.5.1 Concrete Beams

Digital image correlation (DIC) can be used for accurately measuring strains and displacements on concrete beams, particularly on the vertical faces. Of particular interest in reinforced concrete is measurement of the crack widths and the distribution of cracks, as well as visualization of crack patterns. In this study Concrete beams were tested in four point bending as shown in Figure 5. As can be seen in Figure 5b, two digital cameras were placed on either side of the beam to collect photos at a rate of one photo per five seconds.
The beams were loaded using displacement control at a rate that at least 3 to 4 photos were taken for every 1 mm displacement of the loading actuator.

![Diagram of beam setup](image1)

**Fig. 5.5.** Beam specimen and DIC setup

Crack Patterns and Widths

The crack widths were extracted from the correlated images after post-processing in VIC-2D. The images were also scaled using two points in the reference photo prior to processing. The line element was placed horizontally on the bottom of the beam at a particular level of deformation. The value \( U(\text{mm}) \), which represents the infinitesimal change in length of the line element, was then extracted and plotted against the value \( X(\text{mm}) \) which represents the horizontal location at which \( U(\text{mm}) \) is calculated. Figure 6 shows the plot of \( U(\text{mm}) \) vs. \( X(\text{mm}) \). The photos in the figure show the horizontal strain, \( e_{xx} \). As can be seen in the plot, the maximum crack width is about 0.5 mm.
Deflection Profile

During testing, the displacement of the beam was measured using linear variable displacement transducer (LVDT) at the mid and quarter span points (Figure 5a). The displacement profiles were extracted from the DIC by using the same line element. However, V(mm) instead of U(mm) was extracted. The V(mm) represents the vertical displacement of the deformed image. Figure 7 shows the vertical displacement of the bottom chord of the beam at the same level of deformation shown in Figure 6. As can be seen in this figure, the DIC data and LVDT data are in a very good agreement.
5.5.2 Mechanical Properties of Concrete and Masonry Constructions

Mechanical properties of concrete and masonry constructions such as modulus of elasticity, can be determined by using DIC. In this study, DIC was utilized to determine the modulus of elasticity (E) of masonry. The test setup is shown in Figure 8. The camera time was synchronized with the computer time and the files were saved using the timestamp to ensure they were organized for digital image correlation. DIC is a non-contact-virtual substitute for instrumentation that is placed directly onto the test specimen. The main benefit is that it has the ability to put virtual strain gauges and extensometers of flexible gauge lengths on the specimen. A potential also exists to monitor strain that is in real time with digital image correlation. As DIC works on pixels, one may define a nearly unending amount of strain gauges and extensometers. For example, the amount of strain gauges that may be utilized in a usual four-course high masonry prism is over 147000 (Figure 9).

Fig. 5.8. Setup of DIC for prism specimen
Several elements can affect the DIC output such as camera resolution and shutter speed, the size of the subset, the method of correlation (incremental or fixed), and the position of reference point. In 2010, Jakson et al., demonstrated the effect of different sizes of subsets and speckle patterns [32]. Speckle combination and subset size may greatly influence the precision in measurements by DIC [33].

Figure 10 compares the failure mode with digital image correlation. It should be noted that the failure in masonry prism is brittle and sudden. Hence, the camera was removed prior to reaching the maximum load (failure) to protect the camera. Horizontal strain $e_{xx}$ is shown in Figure 10a. Area of high strain concentration is shown by red color. For vertical strain $e_{yy}$, the zone of high strain is at the top of the prism, where the load was applied. However, negative strain values are also on the second block from the bottom in the direction of $e_{yy}$, which demonstrates the area that is critical (Figure 10b). Combining $e_{xx}$ and $e_{yy}$ results showed that the failure is in a triangular shape beginning at the second block (Figure 10c).
A typical load-deformation plot for the prism specimen is presented in Figure 11. A linear vertical LVDT of 5 mm stroke was utilized to acquire displacement data. Since LVDT is an electro-mechanical device, some errors always have to be corrected properly prior to the analysis of the data. Figure 11 demonstrates a common error type (a sliding LVDT). This error was corrected as shown in Figure 11b. To determine the modulus of elasticity of the masonry, the slope of the stress-strain curve was found.
The load-deformation plot obtained from DIC data is shown in Figure 12a. The two plots are compared in Figure 12. Generally, the load-deformation behavior of the masonry prisms is not a perfect line. The value of the slope found for the LVDT data (Figure 12b) is significantly higher than the recommended by Canadian standards CSA S304 [34]. However, DIC data estimated the value which is smaller to the recommendation of this standard. DIC also predicts more realistic behaviour that is no longer linear. Figure 12a demonstrates the DIC result when finding the masonry prism load-deformation behaviour. The second degree polynomial is in an appropriate range of error ($R^2=0.93$) and may model the masonry assemblage non-linear behaviour as well.

![Load-deformation plots](image)

(a) DIC load deflection  
(b) LVDT load deflection

**Fig. 5.12.** Load-deflection behaviour of Masonry prism

### 5.5.3 Mechanical Properties of FRP

DIC technique can also be utilized to obtained the material properties of the different types of fiber reinforced polymer (FRP) fabric such as carbon FRP (CFRP) and Basalt FRP (BFRP). In this study, the material properties of BFRP was investigated. The test setup for BFRP dry fabric is shown in Figure 13. Properties of the BFRP dry fabric were found by
testing coupons in accordance with ASTM D3039/D3039M-14 [35]. The ultimate stress, ultimate strain, and modulus of elasticity were found by averaging the results of five coupon tests.

![Image](image.jpg)

**Fig. 5.13. Setup for BFRP tensile coupon**

The virtual strain gauge of 150 mm gauge length is shown in Figure 14 applied in the last step of loading just before the rupture occurred and the value obtained from the test is 2.32%. It is worth noting that the thickness of the specimen is small which does not allow mounting of a physical extensometer (contact) on the tensile specimen. However, a non-contact technique such as DIC can be easily utilized to obtain the required data such as displacement, strain, and modulus of elasticity. These values can be used for the finite element (FE) modelling of BFRP fabrics externally bonded to corroded steel beams.
The advantage of using DIC is not only limited to obtain the strain and deflection data from the specimen. BFRP is a uni-directional braided fabric. This kind of behaviour can be observed in Figure 15a as the squared shape pattern of strain contours occurred in Y direction \( (e_{xx}) \). The highest strain value in X direction \( (e_{yy}) \) occurred in the middle portion of the BFRP tensile specimen as shown in Figure 15b. This is a critical region with high strain concentration where failure occurred (Figure 15c).

\[ \text{DIC for monitoring the behavior of tensile coupon} \]
5.6 CONCLUSIONS

The cost effective solution for implementing digital image correlation was presented in this paper. During the validation process, it was found that both the elastic and plastic strains were consistent with that of the physical extensometer. Several full scale experiments were conducted to obtained the viability of the DIC method in experimental mechanics. The experimental results indicate that the DIC method is very accurate in obtaining the mechanical properties of different materials such as modulus of elasticity and also the determining the behavior of structural elements such as concrete beams.

5.7 REFERENCES


CHAPTER 6
GENERAL DISCUSSION AND CONCLUSION

The current study was conducted to investigate the structural behaviour of masonry load-bearing elements such as beams built with stack pattern construction. In doing so, full-scale masonry beams and prisms combined with the numerical and optics approaches were carefully designed and implemented. This chapter concludes the findings in previous chapters discussed in detail and provides recommendations for future study.

6.1 CONCLUSIONS

The following conclusions are made based on the full-scale tests and digital image correlation (DIC) conducted under the scope of this research.

1. Flexural and shear capacities obtained for four-course high beams built in running bond and stack pattern constructions are similar. Both stack pattern and running bond beam specimens failed same way. SN4 and RN4 failed in shear whereas, SY4 and RY4 failed in flexure. Hence, it can be concluded that construction pattern has no effect on the strength, deformability, and failure mode for four-course high masonry beams.

2. The differences in deformability and as well as the difference in the maximum load carrying capacity between four-course high beam specimens with shear reinforcements (SY4 and RY4) are 3.7% and 0.5% respectively. Specimen SN4 showed about 2.2% less ultimate strength than its counterpart RN4. However, specimen SY4 exhibited about 3.2% increase in its ultimate strength as compare to specimen RY4. These differences are negligible if the range of acceptable variability for masonry material is considered.

3. Both three-course high beam specimens without shear reinforcements (RN3 and SN3) failed due to flexure. The strain data obtained from the bottom rebar confirms that these
two beams failed in ductile manner. The shear cracks formed in these beam specimens did not cause the failure. The maximum mid-span deflections experienced by specimens RN3 and SN3 were 40 mm and 42 mm, respectively. Hence, the Study shows that the deformability of these two beams were similar. Specimens RN3 and SN3 exhibited maximum load capacity of about 50 kN and 51 kN, respectively. Hence, there is no considerable difference in maximum load carrying capacity of RN3 and SN3 beam specimens. Specimens RY3 and SY3 yielded when the load was about 30 kN and 24 kN, respectively. The strain data indicates that for these two beam specimens, shear mechanism had little effect on the failure.

4. Both stack pattern and running bond two-course high beams exhibited similar deformability at failure. the specimen SN2 showed 7% increase in ultimate strength compared to RN2 and SY2 exhibited about 1% increase in the ultimate strength over its counterpart RY2. Thus, based on the current study, it can be concluded that the two-course high stack pattern masonry beam (SN2 and SY2) is not inferior to its equivalent masonry beam with running bond construction (RN2 and RY2).

5. The failure modes of both running bond and stack pattern construction RM beams were identical. As well, the final crack pattern and crack growths of these two beam constructions were similar. This conclusion is valid even when the slenderness ratio is changed and thus, the mode of failure changes between shear and flexure.

6. Four beam specimens, R20N, R30N, S20N, and S30N were used to determine the effect of block size on both running bond (RB) and stack pattern (SP) beam construction. The difference in average failure load values obtained from 20 cm and 30 cm block units is about 28%. The difference in maximum load carrying capacities of R20N and R30N
specimens is about 35%. The difference is 28% between S20N and S30N beam specimens. Hence, the study found that the size of masonry block units has no or minimal effect on the load carrying capacities of masonry beams, other than the fact that larger unit provides larger load or moment carrying capacity.

7. The specimen R20H which was built with high-strength grout exhibited 34% higher capacity than its counterpart running bond beam specimen built with normal strength grout, R20N. The difference in the maximum load capacities between these two beam specimens S20N and S20H is about 29%. The difference in the strengths of two grouts (high-strength and normal-strength) is 197%. Hence, this study shows that the increase in the strength of the beam is not proportional to the increase in grout strength.

8. This study concludes that despite the significant difference in the strength (197%) between the high strength and normal strength grouts, the increase in maximum load carrying capacity of the beam specimen can only be 34% and 29% for running bond and stack pattern constructions, respectively. This is due to the fact that there are other elements such as mortar, block, and steel rebar and they all share the load as a composite construction.

9. This study found that the stack pattern construction has insignificant beneficial effect on the behavior of masonry beam when normal strength grout is used. The effect of construction pattern (running bond construction versus stack pattern construction) is negligible when high strength grout is used.

10. The current study proposed a strain-based approach for determining the yield displacement (Δy) accurately. The approach uses strain data to be obtained from the test. These data were used for deriving the empirical relationship for accurate estimation of the
yield displacement. Since the empirical relationship was derived from the yield load obtained using the strain data, the results are more accurate compared to the approach proposed by Priestley and Park which were found to be conservative.

11. This study compares the load-displacement behaviors of unreinforced masonry and PP band retrofitted masonry walls through displacement controlled lateral loading. The comparison of load-displacement behaviors revealed that retrofitting of URM wall using PP band enhances the ductility capacity and energy absorption capacity almost 3 and 2 times, respectively, in comparison to unreinforced masonry wall.

6.2 RECOMMENDATIONS

The following recommendations are made regarding the future study,

1. It is recommended to conduct more full-scale tests on masonry beam specimen with different slenderness ratio to obtain the wider range of data pool. Objective is to obtain data to validate the sensitivity of the masonry beams to different parameters.

2. Conducting more tests on masonry beams with different mortar and grout strength. The objective is to assist in better prediction of structural behaviour of masonry beams constructed in stack pattern and running bond.

3. Finite element model could be implemented on the masonry beams for parametric study. Objective is to obtain better understanding of the effect of each components in masonry construction such as mortar, grout, and block on the structural behaviour of the masonry beams. Moreover, this could be utilized to derive more accurate formula to determine the yield displacement for wider range of the masonry beam specimens.
4. Utilizing the 3D-DIC to obtain displacement and strain at every point on the specimen surface in three directions to determine the interaction between different components in masonry structures. Further, it provides the better understanding of behaviour of masonry structures in micro level.

5. It is recommended to conduct more full-scale wall test with and without reinforcements wrapped with PP-Band to determine the effect of the PP-Band on unreinforced as well as reinforced masonry wall.

6. More tests on masonry wall with different height are recommended to determine the effect of the PP-Band on the ductility of masonry walls with different slenderness ratio.
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