The effectiveness of intermediate reinforcement in large masonry beams

Bryan Boutilier

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EFFECTIVENESS OF INTERMEDIATE REINFORCEMENT IN LARGE MASONRY BEAMS

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AUTHOR’S DECLARATION OF ORIGINALITY

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ABSTRACT

In large masonry beams, the development of flexural cracks arises due to flexure. It is believed that flexural cracks are largest between the neutral axis and flexural reinforcement in large masonry beams. The addition of intermediate reinforcement in large masonry beams is believed to reduce side face crack flexural cracks widths. The experiment was carried out to determine the effect of intermediate reinforcement on flexural cracking behaviour in large masonry beams. An experimental study using fifteen prisms and eight beam specimens was completed. It was found that the addition of intermediate reinforcement effectively reduced intermediate widths and provided an evenly distributed cracking pattern.
DEDICATION

I would to thank my family and friends, who have exhibited unbelievable support and consideration.
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LIST OF SYMBOLS

a – shear span
a_c – depth of theoretical compression block
A – area of concrete having the same centroid as the flexural reinforcement measured from the extreme tension fibre divided by the number of flexural reinforcing bars
A (kg/m^3) – amount of water absorbed per unit volume of block
A (%) – amount of water absorbed as a mass percentage
A_b – net surface area of a stretcher unit
A_c – area of concrete
A cb – average area of concrete surrounding each bar
A cs – effective area of concrete surrounding skin reinforcement
A e – effective area of concrete surrounding steel reinforcement
A en – effective area of grouted prisms loaded normal to bed joint
A ep – effective area of grouted prisms loaded parallel to bed joint
A int – area of intermediate reinforcement
A mb – effective mortar bed area
A s – area of steel reinforcement
A sk – area of skin reinforcement
A st – area of flexural steel reinforcement
b – beam width
b_s – bar spacing
b(x) – bond shear stress as a function of ‘x’
\( \beta \) – strain gradient factor
\( \beta_1 \) – stress block factor
c – actual compression zone depth
C – compression force
d – effective depth
d_b – bar diameter
d_c – bottom concrete cover of flexural reinforcement
\(d_{\text{int}}\) – distance between centroid of intermediate reinforcement and the extreme compression zone fibre
\(d_s\) – effective depth of concrete covering skin reinforcement
\(d_{\text{at}}\) – distance between centroid of flexural reinforcement and the extreme compression zone fibre
\(d_t\) – distance between extreme tension zone fibre and neutral axis

\(D\) – experimental deflection at mid-span of the beam
\(D_b\) – density of the block
\(\Delta\) – theoretical deflection at mid-span of the beam

\(e\) – effective horizontal distance for two bars, case 2 (Broms, 1965)
\(e_1\) – horizontal distance from left bar to point of interest (Figure 2-16)
\(e_2\) – horizontal distance from right bar to point of interest (Figure 2-16)

\(E_m\) – compressive elastic modulus for masonry
\(E_s\) – modulus of elasticity for steel reinforcement

\(\varepsilon_{\text{cm}}\) – average concrete tensile strain
\(\varepsilon_{\text{int}}\) – intermediate reinforcement steel strain
\(\varepsilon_{\text{my}}\) – masonry compressive strain at yield
\(\varepsilon_s\) – steel strain
\(\varepsilon_{\text{st}}\) – flexural steel strain
\(\varepsilon_y\) – yield strain of steel

\(f\) – compressive strength of the block
\(f_{\text{av}}\) – average compressive strength
\(f_0\)’ – specified compressive strength of the block
\(f_m\)’ – masonry compressive strength
\(f_r\) – modulus of rupture for concrete
\(f_s\) – steel stress

\(f_l\) – lintel block splitting tensile strength
\(f_t(x)\) – concrete tensile stress

\(f_y\) – yield strength of steel

\(h\) – height of the beam
h₁ – distance between the centroid of the flexural reinforcement and the neutral axis
h₂ – distance between the extreme tension fibre and the neutral axis
H – height of the block
Iₐᵣ – moment of inertia for a cracked section
Iₑffective – effective moment of inertia for masonry beam
Iₑffective(s) – effective moment of inertia for masonry beam taking into account splice
I₀ – moment of inertia for an un-cracked section
j – shortest distance between concrete/masonry surface and centroid of the closest reinforcing bar
lₜ – transfer length
L – beam span
Lₛ – split length for lintel block
Mₐ – applied bending moment
Mᵧ – yield moment
n – number of observations
P – applied load
Pᵣ – reference load
Pᵧ – applied load at yield
Ψₛ – crack spacing factor
q – uniformly distributed load due to self weight
Q – maximum applied load for block compression test and lintel split test
ρₛₖ – skin reinforcement ratio
ρᵥ – shear reinforcement ratio
s – skin reinforcement spacing
sᵣ – bond shear stress
sᵣ – standard deviation
S – crack spacing
σₛₖᵣ – steel stress at a cracked section
σₛₖˌmin – minimum steel stress
t – the distance between the centroid of the closest reinforcing bar and the point of interest on the concrete/masonry surface
t_e – the effective distance between the centroid of the reinforcing bars and the point of interest on the concrete/masonry surface
T(0) – the force in the steel bar at a cracked section (x = 0)
T(x) – the force in the steel bar as a function of ‘x’
v – coefficient of variation
v(%) – coefficient of variation
V – volume of block
w – crack width
w_r – crack width at the flexural steel level
w_t – crack width at the extreme tension zone fibre
w(x) – crack width as a function of ‘x’
w_w – crack width in the web
Wave – average crack width
W_max – maximum crack width
W_d – dry weight of the block
W_i – immersed weight of the block
W_s – saturated weight of the block
W_x – initial weight of the block
x – unit of length measured horizontally from the cracked section (Figure 2-7)
y – unit of length measured vertically from the neutral axis (Figure 2-20)
z – crack width factor
1 Introduction

1.1 Background

The use of high strength and ductile reinforcing bars in masonry structures has allowed them to support higher loads and ductility before failure. Masonry, like concrete, is very weak in tension. In reinforced masonry beams, when the tensile strength of masonry is exceeded, flexural cracks develop. Masonry cracks in general can cause serviceability issues such as corrosion of reinforcing steel through water intrusion, increased deflections from reduced stiffness, and aesthetic issues. Complete prevention of masonry cracks is uneconomical and impractical, due to the low tensile strength of masonry. Rather masonry crack widths are limited to a maximum acceptable value to satisfy the serviceability requirements. Current crack control requirements (CSA S304.1 – 04a clause 11.2.6.2) only limit crack width in the region of the flexural reinforcement. In large reinforced masonry beams (beam height > 600mm), the tension zone depth becomes large and the above crack control requirements may not effectively control the width of flexural cracks between the neutral axis and the flexural reinforcement. In this thesis such cracks will be called intermediate cracks, not to be confused with shear cracks. It is assumed that intermediate cracks ($w_w$) may be wider than the cracks at the extreme tension fibre ($w_t$) as shown in Figure 1-1 (Frantz & Breen, 1980).

![Figure 1-1: Intermediate Cracks](image_url)
It is also assumed that intermediate reinforcement effectively limits intermediate crack widths to an acceptable level as shown in Figure 1-2. However, no research based information for large masonry beams (h > 600 mm) is currently available to support any of these two assumptions.

![Figure 1-2: Benefit of Intermediate Reinforcement](image)

Based on these assumptions, the current Canadian standard CSA S304.1-04a (CSA, 2004a) suggests the following guideline.

- Where the beam height exceeds 600 mm, longitudinal reinforcement shall be uniformly distributed over the height of the beam. A single No. 15 bar for beams up to 240 mm wide, and a No. 15 bar on each side for wider beams, shall be provided at 400 mm vertical spacing.

Though this does not specifically indicate a problem in controlling intermediate cracking in large masonry beams, it is implied that the intention is to reduce intermediate crack width using additional reinforcement (to be called *intermediate reinforcement*) placed away from the flexural reinforcement. Since there is no research data available on intermediate cracks and control of such crack widths in large masonry beams, current
recommendation in CSA S304.1 (2004a) on use of intermediate reinforcement is based on similar recommendation on skin reinforcement for large concrete beams (beam height > 750 mm) in the Canadian concrete standard CSA A23.3 (2004b) Clause 10.6.2. However, there are differences between skin reinforcement and intermediate reinforcement. The major difference is that skin reinforcement is provided near the side face of beam webs in reinforced concrete (30 mm to 50 mm clear cover); whereas intermediate reinforcement is provided at mid-width (b < 240 mm) of reinforced masonry beams. It is not feasible to provide intermediate reinforcement close to the side face of the masonry beam since masonry beams are constructed using precast masonry units. In addition, another difference is that masonry beams are rectangular in cross-section, whereas concrete beams can be I or T-shaped. Recommendation of use of skin reinforcement in CSA A23.3 (2004b) is based on several studies undertaken on reinforced concrete T-beams (Frantz & Breen, 1980). These studies concluded that side face crack widths in the concrete T-beam’s web could well be larger than crack width at extreme tension zone (see Figure 1-1) and addition of side face or skin reinforcement is effective in controlling such crack widths (see Figure 1-2).

1.2 Objectives

Hence, the primary objectives of this study are as follows:

- Determine the similarities and differences in cracking behaviour of large reinforced masonry beams and large reinforced concrete beams.
- Determine the effectiveness of intermediate reinforcement at controlling intermediate crack widths in large masonry beams.
- Determine the adequacy of the current CSA S304.1 (2004a) Clause 11.2.6.2 and recommend, if any, changes to the current guideline is required.

1.3 Scope of Work

This study was completed using experimental method. Following are the activities completed under the scope of this project:
• A detailed literature review on masonry and concrete flexural crack behaviour in large beams
• Tests on fifteen prisms and eight full-scale masonry beams
• Analysis of beam and prism test data
• Comparison of masonry beam test data on crack behaviour with those found in previous studies on concrete beams
• Conclusions and recommendations

1.4 Methodology

The current work was completed using experimental method. A total of eight full-scale large masonry beams were built and tested to achieve the goals. 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.

1.5 Thesis Layout

The components of this thesis are laid out in six chapters. Chapter two discusses the detailed literature review of previous works related to the concept of skin and intermediate reinforcement. Chapter three contains a meticulous description of the experimental setup and procedures. Chapter four includes results acquired from tests on different materials used in this study such as: masonry units, grout, steel, mortar, and prisms. Chapter five is the primary and most important one and this chapter discusses beam test results and logical discussions about cracking behaviour of large masonry beams in comparison with large concrete beams. Finally, chapter six provides the summary, conclusions, and recommendations for future research work.
2 Literature Review

2.1 Introduction

Masonry construction is one of the oldest methods of construction and it has been around for centuries. The first materials used for masonry consisted of natural stones that were used to make walls. Masonry construction has evolved over the years and has become standardized. The most commonly used modern masonry units are clay bricks and concrete blocks. A new application for masonry is the construction of masonry beams. Masonry beams are constructed using concrete or clay bricks, grout, mortar, and reinforcing bars. Large masonry beams are assumed to be vulnerable to flexural intermediate cracking. In this section, design codes, results found from previous experiments, and other relevant research are summarized and explained in an effort to gain a better understanding of intermediate beam cracking in masonry and concrete beams. No research directly related to intermediate reinforcement in large reinforced masonry (RM) beams (h > 600 mm) were found in the public domain, however, a large collection of research on skin reinforcement in large reinforced concrete (RC) beams (h > 750 mm) was found which is currently believed to be similar to intermediate reinforcement in RM beams.

2.2 Concrete Masonry Units

Concrete masonry units are made from Portland cement, aggregates, and water. The ingredients are mixed together and pressed into a mould of the block. The blocks are then steam cured for about 18 hours to allow hydration during hardening. After this, the blocks are then stacked onto palettes and stored until use. The standard block sizes in Canada are shown in Table 2-1 (CSA A165.1, 2004c)
Table 2-1: Dimensions for Standard Concrete Masonry Units  
(CSA A165.1, 2004c, Table 2)

<table>
<thead>
<tr>
<th>Width (mm)</th>
<th>Height (mm)</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal</td>
<td>Actual</td>
<td>Nominal</td>
</tr>
<tr>
<td>100</td>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>150</td>
<td>140</td>
<td>200</td>
</tr>
<tr>
<td>200</td>
<td>190</td>
<td>250</td>
</tr>
</tbody>
</table>

The nominal dimensions are the actual dimensions plus the 10 mm mortar joint. The most commonly used concrete blocks have nominal dimensions of 200 mm x 200 mm x 400 mm (W x H x L) and was used for this experimental program. The different types of concrete blocks that were manufactured and used in this experiment are the standard stretcher unit, splitter unit, and lintel unit. The stretcher units were the most common type and were primarily used in this project. The splitter units were split in half easily and used when only half blocks were needed. Typical places where half blocks are used are at movement joints and around openings. The lintel units were used as the bottom course in construction of beams and lintels. The lintel units have a longitudinal gap to allow reinforcement to be laid along the length direction of the block. A stretcher unit with the common terms used to describe the parts is shown in Figure 2-2. The faceshells are the front and back faces of the stretcher unit. The webs are the three legs that connect the two faceshells. The flared faceshells and webs allow for easy removal of the mould when being manufactured. The flared faceshells and web also help with gripping the unit and have an increased area of mortar bedding. The two indentations at the length edges of the unit are called frogged ends. The area adjacent to the frog is called the ear. The ear of the masonry unit is the surface for the mortar head joint.
Figure 2-1: Concrete Block Types Used In This Study

(a): Stretcher Unit  

(b): Splitter Unit

(c): Lintel Unit

Figure 2-2: Stretcher Unit Components

(a): Top View

(b): Front View

(c): Side View

Figure 2-2: Stretcher Unit Components
Concrete masonry units are classified using four properties: Solid Content, Compressive Strength, Density, and Moisture Content, as shown in Table 2-2 (CSA A165.1, 2004c, table 1). Solid content is based on the net area of a top face. The unit is considered to be 100% solid when the cells and frogged ends are filled, otherwise the unit is considered to be semi-solid or hollow depending on the net area. The semi-solid unit has a net area greater than 75% of the gross area, whereas, the hollow unit has a net area less than 75% of the gross area.

Table 2-2: Classification Properties of Masonry Units (CSA A165.1, 2004c, Table 1)

<table>
<thead>
<tr>
<th>Facet/Property</th>
<th>Hollow (H) (&lt;75% Solid)</th>
<th>Semi-Solid (SS) (&gt;75% Solid)</th>
<th>Solid (SF) (100% Solid)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2. Minimum Compressive Strength(Net Area)(MPa)</td>
<td>10</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>3. Concrete Type Symbol Density (kg/m³) Absorption (kg/m³)</td>
<td>M  &lt;0.03</td>
<td>M  0.03 – 0.045</td>
<td>M  &gt;0.045</td>
</tr>
<tr>
<td>4. Shrinkage Symbol Shrinkage (%) Maximum Moisture Content (% of Total Absorption)</td>
<td>RH ≥75%* 45</td>
<td>RH&lt;75%* 40</td>
<td>35</td>
</tr>
<tr>
<td>2.3 Mortar</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The purpose of mortar is to bond the masonry units together. Portland cement, lime, sand, and water are the ingredients used to make mortar. The most important properties of mortar are workability, bond strength, and compressive strength. Workability is a measure of the ease of use during construction. The mortar must be workable and provide a strong and durable bond between masonry units. The addition of lime to mortar increases the workability. Although the measure of workability is highly qualitative, a flow test can be used to determine a quantitative measure of workability. The mortar
must have enough water content to spread and adhere to the rough face shell surface. The bond is also affected by the air content of the mortar. The air content of the mortar is limited to 18% (CSA A179, 2004d, Clause 7.2.2.6). The air content also contributes to the mortar compressive strength. The mortar compressive strength is largely affected by the cement to water ratio. Mortar with high cement to water ratio will have a greater compressive strength than a low cement to water ratio. Although a high strength mortar (high cement ratio) will have a higher compressive strength, workability and bond strength may be compromised. A lower strength mortar (low cement ratio), which has improved workability, may be desirable (Drysdale & Hamid, 2005). Mortar is classified into five types: M, S, N, O, and K. The list of mortar types are in order of highest compressive strength (Type M) to lowest compressive strength (Type K). The list is also in order of lowest workability (Type M) to highest workability (Type K). Two types of mortar are used in Canada: Type S and Type N. Type S is used where strength is important such as a load bearing wall. Type N is used for non-structural purposes such as interior partitions or veneer. Types M, O, and K mortars are only used in special circumstances. The different types of mortars are specified in CSA A179 – 04d, Annex A. Mortars can be specified in two different ways, by property or by proportion. The property based mortar is specified using a performance based criteria (compressive strength), where as the proportion based mortar specifies volumetric ratios (CSA A179, 2004d). The following two tables show mortar specifications by proportion (Table 2.3) and by property (Table 2.4) for Types S and N (CSA A179, 2004d).
Table 2-3: Proportion Specifications of Mortar by Volume (CSA A179, 2004d, Tables 3 & 4)

<table>
<thead>
<tr>
<th>Mortar Type</th>
<th>Proportions by Volume</th>
<th>Portland Cement</th>
<th>Masonry Cement</th>
<th>Hydrated Lime or Lime Putty</th>
<th>Fine Aggregate in Damp, Loose Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>1 - - 0.5 3.5 to 4.5</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
<td>3.5 to 4.5</td>
</tr>
<tr>
<td>N</td>
<td>0.5 - 1 - 3.5 to 4.5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>3.5 to 4.5</td>
</tr>
<tr>
<td></td>
<td>- 1 - - 2.25 to 3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2.25 to 3</td>
</tr>
</tbody>
</table>

Table 2-4: Property Specifications of Mortar (CSA A179, 2004d, Table 6)

<table>
<thead>
<tr>
<th>Mortar Type</th>
<th>Minimum Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 Day Test</td>
</tr>
<tr>
<td>S</td>
<td>5</td>
</tr>
<tr>
<td>N</td>
<td>2</td>
</tr>
</tbody>
</table>

2.4 Grout

The four main ingredients of grout are water, Portland cement, Lime, and (fine and coarse) aggregate. Occasionally small quantities of hydrated lime are added as well (Drysdale & Hamid, 2005). Hydrated lime is rarely added to grout, only when an increase in workability is required. The purpose of grout is to increase compressive load capacity and/or bond between reinforcing bars and masonry units. Grout must have a high slump to allow filling of the empty spaces without any voids and good contact with reinforcing bars for bond strength. A high slump corresponds to a high water-cement ratio. The masonry units readily absorb the water from the grout ultimately lowering the water-cement ratio, hence another reason why a high water-cement ratio is needed. There are two types of grout mixes: fine and coarse. Table 2-5 shows the volumetric proportions of the main ingredients for each grout type as specified in the standard (CSA A179, 2004d).
It is recommended that enough water be added to produce a high slump (200 mm – 250 mm) (CSA A23.2, 2004e).

Table 2-5: Proportion Specifications of Grout by Volume (CSA A179, 2004d, Table 5)

<table>
<thead>
<tr>
<th>Grout Type</th>
<th>Parts by Volume</th>
<th>Aggregate Measured in a Damp, Loose State (Times Sum of Cementitious Materials)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Portland Cement</td>
<td>Hydrated Lime or Lime putty</td>
</tr>
<tr>
<td>Fine</td>
<td>1</td>
<td>0 – 0.1</td>
</tr>
<tr>
<td>Coarse</td>
<td>1</td>
<td>0 – 0.1</td>
</tr>
</tbody>
</table>

The compressive strength of grout is of importance as it affects the compressive strength of masonry. The grout compressive strength is determined by casting cylindrical specimens in non-absorbent moulds for compressive testing. Table 2-6 shows the minimum specified compression strength for performance criteria recommended by CSA A179 (2004d).

Table 2-6: Property Specifications of Grout (CSA A179, 2004d, Table 7)

<table>
<thead>
<tr>
<th>Grout Type</th>
<th>Minimum Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Non-Absorbent Mould)</td>
</tr>
<tr>
<td></td>
<td>7 day</td>
</tr>
<tr>
<td>Fine</td>
<td>6</td>
</tr>
<tr>
<td>Coarse</td>
<td>7.5</td>
</tr>
</tbody>
</table>

In addition to casting grout into non-absorbent for compression testing, grout is also cast into masonry units and extracted for compression testing (ASTM C1019, 2011). When grout is cast into the masonry units, part of the total water is absorbed into the masonry units lowering the water-cement ratio of the grout. The difference in water-cement ratio for grout in non-absorbent moulds and grout in-situ may cause a significant difference in the compressive strengths. Hence, the grout compressive strength in-situ is usually higher.
than the grout compressive strength obtained from non-absorbent moulds. It is believed that the grout compressive strength in-situ is typically more than 50% higher than when cast in a non-absorbent mould (Drysdale and Hamid, 2005).

2.5 Behaviour of Masonry Beams

In masonry construction, typical wall construction projects include window or door frames that could span longitudinally for several meters. In these situations, it is necessary to include masonry beams to resist shear and flexural forces. Masonry, like concrete, is very weak in tension. Therefore, steel is required to resist tensile forces in beams and lintels. Lintel units, as shown in Figure 2-1 (c), allow placement of longitudinal reinforcing bars in masonry beams. All masonry beams are grout filled to bond the reinforcing bars with the beam for composite behaviour.

![Figure 2-3: Masonry Beam Application](image)

The analysis and behaviour of RM beams under applied load is similar to that of RC beams, however crack patterns could be different. During the application of moment, the masonry beam undergoes three main stages: pre-cracking, post-cracking, and post-yielding (Drysdale & Hamid, 2005). Before cracking, the applied moment and deflection are very low and the masonry and reinforcement are essentially linear elastic (Figure 2-4a). Under the assumption of a linear strain distribution, the neutral axis, second moment
of inertia, and stresses and strains in the element can be calculated using the section shown in Figure 2-4 (a). Once the tensile stress in the masonry exceeds the tensile strength of masonry (modulus of rupture), cracks begin to form. These cracks form from the energy dissipation of the masonry in tension. The result is a decrease in stiffness and an increase in deflection. After cracking, the neutral axis, second moment of inertia, and stresses and strains in the element can be calculated using the section shown in Figure 2-4 (b). The flexural cracks usually initiate at the head joints of the bottom course where moment is maximum, and as a result the mortar de-bonds from the lintel unit (Ring, 2009). Post-yielding occurs when the steel begins to yield and significant decrease in flexural stiffness is observed.

The initiation of flexural cracking causes a redistribution of internal stresses and strains in the tension zone of the masonry beam between masonry and steel. The tensile stress carried by the masonry is assumed to be zero as a conservative approach. However, the masonry carries a small amount of tension; this phenomenon is referred to as the tension stiffening effect. The actual non-linear stress-strain distribution of masonry with a strain limit of 0.003 is used to determine the stress distribution when analyzing the ultimate load capacity of the masonry beams (Figure 2-5c). The CSA S304.1 (2004a) standard has adopted a procedure similar to CSA A23.3 (2004b) standard which allows the replacement of the parabolic stress distribution with an equivalent rectangular stress block as shown in Figure 2-5b when analyzing a beam for design. The equivalent rectangular stress block has the same magnitude and location of resultant as the parabolic stress distribution (Drysdale & Hamid, 2005).
Figure 2-4: Masonry Beam Analysis

(a): Before Cracking

(b): After Cracking

Figure 2-4: Masonry Beam Analysis
Concrete has very similar characteristics to concrete masonry units. Cracking patterns for reinforced concrete (RC) are currently assumed to be very similar to reinforced masonry (RM). The cracking behaviour of RC and RM are influenced by concrete/masonry tensile strength, reinforcing and concrete/grout bond properties, reinforcing stress, and the internal redistribution of stresses in RC or RM. For example, if a reinforced concrete member is subjected to pure tension, concrete and steel both resist the tensile force together (Figure 2-6) until cracks develop in the concrete. Cracks are assumed to develop once the tensile stress in the concrete has exceeded the tensile strength of the concrete. It is very difficult to predict the number and location of the tensile cracks. The initial cracks
may depend on localized imperfections and weaknesses in concrete. For masonry, the weakest location is usually at the mortar joints and hence, cracks are expected to form there. Reinforced masonry cracks usually initiates by de-bonding of mortar and masonry unit. Once cracking has initiated, the entire tensile force at the cracked section is transferred to the steel bar (Braam, 1990).

Figure 2-6: Steel Reinforced Concrete Tension Member

If a perfect steel-concrete bond exists, the tensile steel stress at the cracked section is transferred to the concrete. A tensile member section cut from the cracked section to a distance ‘x’ from the crack is shown to illustrate the shear transfer effect (Figure 2-7).

Figure 2-7: Bond Shear Force Effect
In Figure 2-7, \( b(x) \) and \( f_t(x) \) are the bond shear stress and concrete tensile stress as functions of ‘x’, respectively. The following relationship can be established from equilibrium of the forces.

\[
\int_0^x \pi d_b b(x) dx = f_t(x)A_c \tag{2-1}
\]

\[
T(x) = T(0) - \int_0^x \pi d_b b(x) dx \tag{2-2}
\]

Equation 2-1 and 2-2 can be further simplified by assuming that the bond shear stress remains constant. Figure 2-8 illustrates the internal stress changes in concrete and steel assuming bond shear stress is constant. The notations \( S \), \( w \), \( s_b \), \( f_r \), \( \sigma_{s,cr} \), and \( \sigma_{s,min} \) are the crack spacing, crack width, bond shear stress, concrete modulus of rupture, steel stress at cracked section, and steel stress at undisturbed section, respectively.

Figure 2-8: Internal Stress Distribution for Cracked Tension Member (Braam 1990)
The concrete tensile stress will increase as the distance away from the crack increases until the concrete tensile strength is reached at which point the section is undisturbed (Figure 2-8d). The distance between the cracked section and the undisturbed section is called the transfer length \( (l_t) \) (Braam, 1990). The transfer length is the distance it takes for the force in the bar at the cracked section to transfer back to the concrete until the un-cracked state of stress is reached. The transfer length can be determined by setting \( f_t(x) = f_r \) and solving for ‘x’ in Equation 2-1. A new crack will form at or beyond the transfer length \( (l_t) \). New cracks will continue to form until the concrete tensile stresses at all points in the member are below the concrete tensile strength \( (f_r) \). At this point, the crack pattern is considered to be stabilized. The crack spacing \( (S) \) of a stabilized crack pattern could theoretically be anywhere between \( l_t \) to \( 2l_t \) (Braam, 1990). Therefore, the average crack spacing can be estimated at \( 1.5l_t \). The crack width \( (w) \) can be estimated by considering the change in length of the steel bar relative to the concrete between two cracks.

\[
w = S(\varepsilon_{sm} - \varepsilon_{cm}) \tag{2-3}
\]

The values of \( \varepsilon_{sm} \) and \( \varepsilon_{cm} \) are the average steel and concrete tensile strains, respectively. The concrete tensile strain is very small compared to the steel strain and can be neglected without significant error. Under the assumption that the bond shear stress is constant, the average crack spacing and width can be estimated from Equations 2-4 and 2-5, respectively.

\[
S = \frac{1.5f_rA_c}{\pi d_b s_b} \tag{2-4}
\]

\[
w = \frac{1.5f_rA_c\varepsilon_{sm}}{\pi d_b s_b} \tag{2-5}
\]

\( f_r \) = concrete tensile strength
\( d_b \) = reinforcing bar diameter
2.7 Crack Types

Reinforced Masonry (RM) beams, like Reinforced Concrete (RC) beams, are designed to resist moment and shear forces. The applied moment and shear forces on RM beams can cause flexural and shear cracks to form (Hatzinikolas & Korany, 2005). The type of crack that develops and its location depends on the geometry and loading arrangement of the beam. Moment and shear stress distributions along with small rectangular elements (A, B, C, D, and E) are shown in Figure 2-9. Each element represents a location along the height of the beam to illustrate how the internal principal stresses change orientation for each element (Figure 2-10).

![Figure 2-9: Internal Stress Distribution for Un-Cracked Beam](image)

Flexural cracks develop in high moment regions. Flexural cracks develop in the tension zone and have a vertical orientation (Figure 2-10) (Hatzinikolas & Korany, 2005). Diagonal shear cracks develop in high shear stress regions. Diagonal shear cracks develop near the neutral axis (N.A.) and are generally inclined at a 45° from the horizontal (Figure 2-10) (Hatzinikolas & Korany, 2005). For a small element ‘C’ located at the neutral axis, the principal stresses are at a 45° angle and the shear cracks will be perpendicular to the principal tensile stress. For any small element, it can be assumed that
the cracks will be perpendicular to the principal tensile stress as shown in Figure 2-10. A typical crack pattern is shown Figure 2-10b for a simply supported beam subjected to a single point load at mid-span.

![Figure 2-10: Crack Types](image)

The flexural cracks that have developed at the bottom of a beam can extend past the neutral axis while rotating to a shear crack orientation as shown in Figure 2-10, such a crack will be called a flexural-shear crack (Ring, 2009). A flexural-shear crack will start out as a flexural crack, then when the beam is loaded further, progress into a flexural-shear crack. Flexural-shear cracks develop in regions where the beam is loaded under moment and shear (Figure 2-10) (Ring, 2009).

In reinforced masonry (RM) beams, flexural cracks usually initiate in the lintel block at the head joints. The flexural cracks are caused by the de-bonding of the mortar and block at the mortar-block interface. Once the head joints have cracked, flexural cracks can also develop in the lintel blocks between the head joints if the internal tensile stress exceeds the tensile strength of the block (Figure 2-11c). Flexural cracks progress in the vertical direction and stop short of the neutral axis. Shear cracks develop in the shear span of the beam near the neutral axis. Shear cracks can develop through the block at about a 45°
angle from the horizontal or progress in a step like manner through the bed and head joints (Figures 2-11(a) and (b)).

2.8 Reinforced Masonry and Concrete Beam Cracking

In masonry beams, flexural cracks initiate at the extreme tension zone. The flexural crack widths are controlled using steel reinforcement. Steel reinforcement acts to resist the opening of a flexural crack. The AS 3700 (2007) Australian masonry standard clause 2.5.2.1 suggests an upper limit on the crack width of 1 mm. However, the Masonry Joints Standards Committee (MJSC, 2008) and the Eurocode 6: design of masonry structures (BSI, 2005) do not have any guidelines in terms of crack control. The current crack control requirements that limit crack widths in the vicinity of the flexural reinforcement is governed by the Canadian masonry standard, CSA S304.1 (2004a) clause 11.2.6.2 which has been adopted from the Canadian concrete standard, CSA A23.3 (2004b) clause 10.6.1. A factor, z, empirically derived, is used to gauge and control the flexural crack width at the extreme tension fibre. The value of this ‘z’ factor is limited to 25,000 N/mm and 30,000 N/mm for exterior and interior exposures, respectively. The ‘z’ factor is calculated using Equation 2-6.
\[
z = f_s \times \frac{3}{\sqrt{d_c A}} \quad (2-6)
\]

\(f_s\) = service load steel stress (MPa)

\(d_c\) = bottom concrete cover of flexural reinforcement

\(A\) = area of concrete having the same centroid as the main reinforcement measured from the extreme tension fibre divided by the number of main reinforcing bars (mm²)

The above mentioned limiting values of ‘\(z\)’ correspond to limiting the values of crack width to about 0.33 mm and 0.4 mm for exterior and interior exposures, respectively, under the assumption that \(h_2/h_1 = 1.2\) in Equation 2-7 (CSA A23.3 2004b, Clause N10.6.1). Equation 2-6 is a simplified version of the empirical equation developed by Gergely and Lutz (1968). Gergely and Lutz performed a non-linear regression analysis on crack width data obtained from Kaar & Mattock (1963), and Broms (1965). The empirical equation proposed by Gergely and Lutz (1968) to calculate the crack width directly is shown in Equation 2-7.

\[
w = 11f_s \times \frac{3}{\sqrt{d_c A}} \frac{h_2}{h_1} \times 10^{-6}mm \quad (2-7)
\]

Where, \(h_1\) and \(h_2\) are the distances from the neutral axis to the level of the main reinforcement and to the extreme tension fibre respectively.

The main factors that affect crack widths are steel stress \(f_s\) and concrete cover \(j\). Kaar and Mattock (1963) developed an experimental program to investigate the effects of high strength reinforcing bars in concrete. Cracking was one of the issues investigated in this study. The experimental program consisted of testing I–shaped and T–shaped concrete bridge girders and rectangular shaped slab strip sections. The objective was to investigate how the amount and distribution of reinforcing bars influenced crack widths, crack spacing, and crack patterns. Three specimens were provided with longitudinal reinforcement in the web along half the girder length. The purpose was to determine the effectiveness of longitudinal web reinforcement at controlling crack widths in the girder.
webs. The test data obtained from the experiment included: steel strains in each layer of the main reinforcement level of at mid-span, flexure and shear crack widths all over the length of the beam, concrete strain on the top compression face of the member, stirrup strains, and deflection of the member at mid-span. Based on the test data, crack widths (w) at reinforcement level were found to be linearly proportionate to steel stress (f_s). The use of skin reinforcement was found to be effective at decreasing flexural crack widths in the web. The results of different specimens were then compared at a service load steel stress of 40 ksi (275 MPa). The service level steel stress was assumed to be two-thirds of 60 ksi (413 MPa) yield stress. Crack width throughout the beam height were found to increase for members with more widely spaced reinforcing bars and larger concrete covers, and thus, Equations 2-8 to 2-10 were proposed. The \( A_e \) is the area of concrete surrounding all the reinforcing bars having the same centroid as the reinforcing bars (in\(^2\)). \( A_{cb} \) is the average area of concrete surrounding each bar (in\(^2\)), \( f_s \) is the steel stress (psi), and \( W_{ave} \) and \( W_{max} \) are the average and maximum crack widths (in), respectively (Figure 2-12).

\[
\begin{align*}
A_{cb} &= \frac{A_e}{\text{# of bars}} \\
3in^2 &\leq A_{cb} \leq 50in^2 \\
W_{ave} &= 0.077\sqrt[4]{A_{cb}(f_s)(10^{-6})} \\
W_{max} &= 0.115\sqrt[4]{A_{cb}(f_s)(10^{-6})}
\end{align*}
\] (2-8) (2-9) (2-10)

Figure 2-12: Profile View of Main Flexural Steel Reinforced Concrete Member (Kaar & Mattock, 1963)
Broms (1965a) investigated width, spacing, and behaviour of concrete cracks near the flexural steel in reinforced concrete (RC) members. A theory was also developed based on the results of the experiment. The theory is explained in the following section. A reinforced concrete tensile member or the tension zone of a concrete flexural member reinforced with a single bar is shown in Figure 2-13a.

![Figure 2-13: Steel Reinforced Concrete Cracking Theory for One Bar (Broms, 1965a)](image)

Tensile stresses develop inside the circle between two existing cracks and the diameter of this circle is the crack spacing at the level of the main reinforcement. When the tensile strength of concrete is exceeded, another crack forms inside the circle at mid-point between the existing cracks (broken lines in Figure 2-13b-i). A crack that penetrates the concrete surface is called a primary crack (solid lines in Figure 2-13b-ii). A crack that has not yet penetrated the concrete surface is called a secondary crack (broken lines in Figure 2-13b-iii). The development of new cracks can be seen in Figure 2.13b. The crack pattern
is considered to be fully developed when all the primary cracks have formed. The value of the primary crack spacing (S) can be anywhere between one to two times the concrete cover (t). The value of ‘t’ is the distance from the reinforcing bar to the point at which the crack spacing is being calculated. The crack width (w) on the surface can be related to the primary crack spacing (S) and the average steel strain ($\varepsilon_{sm}$), and thus, Equations 2-11 and 2-12 were proposed by Broms (1965a) to calculate maximum crack width and maximum crack spacing.

\[
S = 2t \quad (2-11)
\]
\[
w = 2t\varepsilon_{sm} \quad (2-12)
\]

Experimental data by Broms (1965a) indicated that Equations 2-11 and 2-12 predicted the maximum crack width and crack spacing for concrete members under pure tension as shown in Figure 2-13. However, Equation 2-12 under-predicts the extreme tension fibre cracks for flexural members due to the presence of strain gradient in beams (Broms, 1965a). The crack width calculated in Equation 2-12 is therefore, multiplied by a factor of $h_2/h_1$ (Equation 2-7) for flexural members to take into account the strain gradient effect (Figure 2-14). The factor, $h_2/h_1$, is larger than one. The strain gradient in flexural members causes larger cracks widths to form at the extreme tension fibre.

![Figure 2-14: Strain Gradient Effect](image)
Broms (1965b) also investigated how the arrangement of multiple bars in flexural and pure tensile members would affect behaviour of cracks in the vicinity of the main tensile reinforcement bars. Based on this study, the theory was extended to consider flexural and tensile members reinforced with multiple bars (Figure 2-15).

A reinforced concrete tensile member or the tension zone of a concrete flexural member reinforced with two bars is shown in Figure 2-15a. Tensile stresses develop inside the circle between two existing cracks around each reinforcing bar. A primary crack is formed when two adjacent circles overlap and the radius of the circle is greater than the concrete cover (Figure 2-15a). Secondary cracks are formed when the diameter of the circle is less than two times the concrete cover (t) and the bar spacing (bₘ) (Figure 2-15b). Primary crack spacing (S) is affected by the concrete cover (t) and bar spacing (bₘ). Based
on the test data (Broms, 1965b), the following procedure can be used to calculate the effective concrete cover ($t_e$). The value of the concrete cover ($t$) in Equations 2-11 and 2-12 is replaced by the effective concrete cover ($t_e$) to calculate the maximum crack spacing and maximum crack width. An example of how the effective concrete cover ($t_e$) is calculated at point A in Figure 2-16 is shown using Equations 2-13 to 2-18. Based on the test data, when the vertical concrete cover ($j$) exceeds the bar spacing ($b_s$), the two bars act as one bar and the effective concrete cover ($t_e$) is equal to the vertical concrete cover.

![Figure 2-16: Two Bar Situation (Broms, 1965b)](image)

$$S = 2t_e$$ \hspace{1cm} (2-13)

$$w = 2t_e e_s$$ \hspace{1cm} (2-14)

\textit{Case 1:} $\frac{b_s}{j} > 1$

$$t_e = \sqrt{e^2 + j^2}$$ \hspace{1cm} (2-15)

$$e = \frac{e_1 e_2}{b}$$ \hspace{1cm} (2-16)
\begin{equation}
\frac{b_s}{j} \leq 1
\end{equation}

\begin{equation}
t_e = j
\end{equation}

For the region outside the reinforcing bars:

\begin{equation}
t_e = t
\end{equation}

\subsection*{2.9 Intermediate Cracking}

Similar to RC beams, beams exceeding 600 mm in height need to be reinforced with intermediate reinforcing bars to control cracking (CSA S304.1, 2004a). Intermediate cracks are flexural cracks that develop in between the flexural reinforcement and the neutral axis of a masonry beam. Intermediate reinforcement is required for controlling intermediate cracks (CSA S304.1, 2004a). Guidelines for the use of intermediate reinforcement can be found in the CSA S304.1-04a masonry design standard clause 11.2.6.3 (CSA, 2004a) which states the following:

- Where the beam height exceeds 600 mm, longitudinal reinforcement shall be uniformly distributed over the height of the beam. A single No. 15 bar for beams up to 240 mm wide, and a No. 15 bar on each side for wider beams, shall be provided at 400 mm vertical spacing.

The CSA S304.1-04a masonry design standard is the only such standard that has guidelines on the control of flexural cracking in large masonry beams. The AS 3700 (Australian Standards, 2007), Eurocode 6 (British Standards Institute, 2006), and MSJC (MSJC, 2008) do not have any guidelines on the issue of flexural cracking. Based on the lack of guidance for crack control in masonry beams, the philosophy of side face cracking in large concrete beams is applied intuitively to intermediate cracking in large masonry beams.
2.10 Side Face Cracking in RC Beams

In reinforced concrete (RC) beams, flexural cracks start at the maximum tension fibre and progress up the side face of the beam towards the neutral axis. Side face beam cracks are those that are located on the side face of a beam web away from the flexural reinforcement bars. These cracks form in between the neutral axis and flexural reinforcement bars. Side face beam cracks may be problematic in large RC beams. The width of side face beam cracks is controlled by providing additional longitudinal web reinforcement called skin reinforcement (Figure 2-17). Skin reinforcement is provided in the tension zone of the beam web and close to the outer faces of the web.

The most extensive study on side face beam cracking and use of skin reinforcement was carried out by Frantz and Breen (1980). In this study, 44 inverted reduced-scale T-beams were tested to investigate the different parameters that affect side face crack widths in large reinforced concrete beams. The main parameters investigated were: amount, location, and distribution of skin reinforcement, concrete cover, skin reinforcement bar type, beam depth, and beam web width. Results from the investigation indicated that three main parameters significantly affect side face beam crack width and they are: level of steel stress ($f_s$), value of concrete cover ($j$), and the beam depth ($h$). Crack widths were shown to be linearly proportional to steel stress ($f_s$). Skin reinforcement was proven to be less effective in controlling crack widths if concrete cover ($j$) increases which is found by work by Broms (1965a). However, Broms (1965a) studied flexural crack behaviour at the extreme tension zone. Bar diameter was shown to be an insignificant factor affecting side face crack width. A high number of bars closely spaced were shown to be more effective at controlling side face cracks than a small number of widely spaced bars with the same steel area. Side face crack widths were shown to increase with increasing beam depth ($h$). Beam width was shown to have little or no affect on side face beam cracking.

In addition to comparing crack behaviour of different specimens, Frantz and Breen (1980) developed a new parameter called crack magnification ratio (CMR) which was also compared. The CMR is defined as the ratio of the crack width in the web ($w_w$) and
the crack width at the main reinforcement level \( (w_r) \) (Figure 1-1). Frantz and Breen (1980) recommended that the crack width in the web be limited to the crack width at the extreme tension fibre which corresponded to limiting CMR to 1.4. Frantz and Breen (1980) determined that for a beam with no skin reinforcement, a CMR of 1.4 occurs at a tension zone depth \( (d_t) \) of 280 mm (11 in). Frantz and Breen (1980) recommended that skin reinforcement needs to be added when the tension zone depth \( (d_t) \) exceeds 280 mm (11 in) (Figure 2-17). Frantz and Breen also suggested that a minimum skin reinforcement ratio using Equations 2-20 and 2-21 to limit CMR to 1.4. The ratio of skin reinforcement area divided by the effective area of concrete is called the skin reinforcement ratio \( (\rho_{sk}) \) and is calculated in Equation 2-19 (Figure 2-17). In Equations 2-20 and 2-21, the amount of skin reinforcement linearly proportional to the tension zone depth \( (d_t) \). The distance between the neutral axis (N.A.) and the extreme tension fibre is called the tension zone depth \( (d_t) \) and is shown in Figure 2-17.

![Figure 2-17: Effective Area of Skin Reinforcement (Frantz & Breen, 1980)](image)
Another investigation on side face cracking was conducted by Adebar and Leeuwen (1999), who conducted an experiment to study side face flexural and shear cracking. The beam specimens were concrete-steel hybrid T-shaped girders, which have two steel flanges that are connected to the concrete by shear studs on the top and bottom of the beam web and beam flange (Figure 2-18). A total of 21 specimens were tested and were split up into ten F-series (F1 – F10) and eleven FS-series (FS1 – FS10, FS3X). The F-series specimens were subjected to pure flexure and the FS-series specimens were subjected to both flexure and shear.

All the specimens were T-shaped with a 1200 mm concrete web height and a 180 mm web width. Specimens had various amounts of side reinforcement and shear reinforcement (stirrups) to determine the influence of each on side face cracking.

\[
\rho_{sk} = \frac{\text{total skin reinforcement area}}{2(2f)(d_e)}
\]  
(2-19)

\[
\rho_{sk} = 0.00058(d_e - 11) \quad \text{for } 11 \text{ in} < d_e \leq 40 \text{ in}
\]  
(2-20)

\[
\rho_{sk} = 0.011 + 0.00015d_e \quad \text{for } d_e > 40 \text{ in}
\]  
(2-21)
behaviour. The side face reinforcement types consisted of 10M deformed bars, welded wire fabric and 0.9% volume of 30 mm hooked steel fiber and the stirrup reinforcement types consisted of 10M deformed bars and welded wire fabric. The concrete clear cover for all specimens was 30 mm. Specimens F1 – F3 and FS1 – FS3 had 10M skin reinforcement inside the stirrups (30 mm cover to stirrups) and specimens F4 – F6 and FS4 – FS6 had 10M skin reinforcement outside the stirrups (30 mm cover to skin reinforcement). The reinforcement ratios and 95th percentile crack widths from the test results of the experiment are summarized in Table 2-7. The crack widths shown in Table 2-7 are for longitudinal service strains of 0.00075 and 0.001 for vertical cracks and for service shear stresses of 1.4 and 2.1 MPa for diagonal cracks. The variables $\rho_{sk}$ and $\rho_v$ are the skin and shear reinforcement ratios, respectively.

Table 2-7: Summary of 95th Percentile Crack Widths in Service Load Range (Adebar and Leeuwan, 1999)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\rho_v$ (%)</th>
<th>$\rho_{sk}$ (%)</th>
<th>Vertical Crack Widths (mm)</th>
<th>Specimen</th>
<th>$\rho_v$ (%)</th>
<th>Diagonal Crack Widths (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.00075 Strain</td>
<td>0.001 Strain</td>
<td></td>
<td>1.4 MPa S. Stress</td>
</tr>
<tr>
<td>F1</td>
<td>0.56</td>
<td>0.32</td>
<td>0.317</td>
<td>0.394</td>
<td>FS1</td>
<td>1.11</td>
</tr>
<tr>
<td>F2</td>
<td>0.56</td>
<td>0.48</td>
<td>0.186</td>
<td>0.222</td>
<td>FS2</td>
<td>1.11</td>
</tr>
<tr>
<td>F3</td>
<td>0.56</td>
<td>0.89</td>
<td>0.157</td>
<td>0.157</td>
<td>FS3</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>FS3X</td>
<td>0.56</td>
</tr>
<tr>
<td>F4</td>
<td>0.56</td>
<td>0.89</td>
<td>0.164</td>
<td>0.188</td>
<td>FS4</td>
<td>1.11</td>
</tr>
<tr>
<td>F5</td>
<td>0.56</td>
<td>1.30</td>
<td>0.103</td>
<td>0.131</td>
<td>FS5</td>
<td>1.11</td>
</tr>
<tr>
<td>F6</td>
<td>0.56</td>
<td>1.79</td>
<td>0.092</td>
<td>0.114</td>
<td>FS6</td>
<td>1.11</td>
</tr>
<tr>
<td>F7*</td>
<td>1.26</td>
<td>0.89</td>
<td>0.115</td>
<td>0.123</td>
<td>FS7*</td>
<td>1.81</td>
</tr>
<tr>
<td>F8*</td>
<td>1.26</td>
<td>1.38</td>
<td>0.107</td>
<td>0.129</td>
<td>FS8*</td>
<td>1.81</td>
</tr>
<tr>
<td>F9*</td>
<td>1.26</td>
<td>2.59</td>
<td>0.088</td>
<td>0.101</td>
<td>FS9*</td>
<td>1.81</td>
</tr>
<tr>
<td>F10**</td>
<td>0.56</td>
<td>0.48</td>
<td>0.173</td>
<td>0.215</td>
<td>FS10**</td>
<td>1.11</td>
</tr>
</tbody>
</table>

*Contains welded wire fabric.
**Contains 0.9% hooked steel fibre.
The vertical crack width values in the F-series decrease with increasing skin reinforcement ratio which is consistent with the study by Frantz and Breen (1980). An interesting and apparent contradictory result is that the crack widths were found to be smaller when the skin reinforcement was placed on the inside of the stirrups (rather than outside of stirrups as in Figure 2-19b) causing an increased concrete cover (Figure 2-19a). In Table 2-7, specimen F3 exhibited crack width of 0.157 mm which is smaller than crack width of specimen F4 (0.164 mm) even though concrete cover was higher, 40 mm in F3 compared to 30 mm in F4.

This seems to be contradictory to the findings by Frantz and Breen (1980), however, the presence of stirrup bars could explain the lower value of the crack width which was not included in the Frantz and Breen (1980) study. Welded wire fabric was found to be better at controlling vertical crack widths than the reinforcing bars as is indicated when comparing F3 and F7. However, this result could be explained by the additional shear reinforcement. An alternative explanation could be that the welded wire fabric has horizontal bars that are vertically spaced only 51 mm apart whereas the reinforcing bars were spaced at 160 mm. The welded wire fabric bar diameter was 6.4 mm whereas the deformed reinforcing bar diameter was 11.2 mm. The lower crack width values of F7 as compared to F3 would be consistent with the results of the Frantz and Breen (1980) study which indicated that a high number of bars closely spaced together is most effective in controlling side face crack widths, regardless of the bar diameter. The results from the
diagonal shear cracks indicate that increasing skin reinforcement effectively decreases the diagonal crack widths as can be seen when comparing FS1 with FS2 and FS3. The diagonal crack widths are smaller when the stirrups are placed on the outside of the skin reinforcing bars (FS3 and FS4). The diagonal crack widths also depend on the transverse reinforcement when comparing FS3 with FS3X and FS7 with FS4. The difference in diagonal crack width between FS7 and FS4 could also be explained by the use of welded wire fabric instead of 10M deformed bars. The results from specimen FS10 indicates that 0.9% hooked steel fibre has a rather small effect on reducing diagonal crack widths (FS10 and FS2). However more testing was suggested to better predict the influence of hooked steel fibre. The diagonal shear cracks are about 1.5 to 2 times larger than the vertical flexural cracks and the FS series specimens had more transverse reinforcement then the corresponding F series specimens. This suggests that the width of diagonal shear cracks may be the governing variable when it comes to crack control in the design of reinforced concrete.

2.11 Crack Width Model

Based on the reinforced concrete cracking theory and work by Broms (1965a), a crack width model was developed by Frosh (2002) to calculate flexural crack width and crack spacing at any location in the tension zone of RC beams. Broms (1965a) performed tests on pure tension and flexural members for concrete covers up to 6 in (152.4 mm). According to the investigation, it was found that the flexural crack spacing (S) can be directly related to the concrete cover (j). The minimum crack spacing was determined to be equal to the concrete cover (j). The maximum crack spacing was determined to be twice the minimum crack spacing. The flexural crack spacing (S) is calculated using Equation 2-22 (Broms, 1965a). The variable (t) in Equation 2-22, is the distance from the closest reinforcing bar to the point of interest (A or B) on the concrete surface where crack spacing and width are being measured (Figure 2-20). The variable ψs, is the crack spacing factor which is one for minimum crack spacing, one and a half for average crack spacing, or two for maximum crack spacing.
The crack width \( w \) is calculated in Equation 2-23 by determining the average bar extension over the crack spacing (Equation 2-23). The strain gradient factor \( \beta \) is used in Equation 2-23 to take into account the linearly increasing strain from the neutral axis to the extreme tension fibre. The strain gradient factor \( \beta \) is calculated using Equation 2-24 with reference to Figure 2-20.

\[
S = \psi_t t \quad (2-22)
\]

\[
w = \beta \varepsilon_{sm} S \quad (2-23)
\]

\[
\beta = \frac{y}{d - c} \quad (2-24)
\]

The model states that the minimum crack spacing is at the level of the reinforcement and increases towards the neutral axis for a beam with no skin reinforcement and this statement is consistent with side face cracking patterns observed by Frantz and Breen (1980). The maximum crack widths are expected to be present on the extreme tension face where the strain value is greatest, however the maximum crack spacing would occur at the point furthest away from the reinforcing bar. Depending upon the tension zone depth, the maximum crack width could occur between the neutral axis and the main reinforcement. The crack width, \( w(y) \), is calculated using Equations 2-25 and 2-26 with reference to Figure 2-20.
The value of ‘y’ can vary between zero at the neutral axis to the tension zone depth at the extreme tension fibre. The variable ‘t’ is the distance between the concrete surface (Points A or B) and the closest reinforcing bar. Equation 2-25 is used to calculate the distance to the closest reinforcing bar and Equation 2-26 can be used to calculate the maximum crack width either at the extreme tension fibre or in the beam web. The addition of side face (skin or intermediate) reinforcement decreases the value of ‘t’ which reduces the maximum crack width, \( w(y) \), in the web. According to the model, a higher number of skin reinforcing bars closely spaced together would be the most effective way to reduce web crack widths which is consistent with the findings by Frantz and Breen (1980). The maximum web crack width increases as the tension zone depth increases which is also consistent with Frantz and Breen (1980). As an example, Figure 2-21 shows a crack width profile graph along the side face of a RC beam assuming a concrete cover of 38 mm (1.5 in), a steel stress of 275 MPa (corresponding steel strain (\( \varepsilon_s \)) of 1400 \( \mu \varepsilon \)) and a tension zone depth of 280 mm (11 in). The beam has no skin reinforcement and one level of main flexural reinforcing bars. Figure 2-22 is the same example except the tension zone depth is increased to 395 mm. The model suggests that the side face crack

\[
\begin{align*}
    t &= \sqrt{(d - c - y)^2 + j^2} \\
    w(y) &= 2 \left( \frac{y}{d-c} \right) \varepsilon_s t 
\end{align*}
\]
width becomes wider than extreme tension zone fibre crack width when the tension zone depth is larger than 280 mm. This is consistent with Frantz and Breen (1980) Equation 2-20 and 2-21 as skin reinforcement is recommended after the tension zone depth is larger than 280 mm (11 in).

Figure 2-21: Crack Width Profile for Tension Zone Depth of 280 mm (11 in) (Frosh, 2002)
2.12 North American Concrete Design Codes and Standards

The four major design codes and standards that govern the design of concrete structures in North America are as follows: CSA A23.3-04b: Canadian Concrete Design Standard (CSA 2004b), CSA S6-06: Canadian Highway Bridge Design Code (CHBDC, 2006), American Concrete Institute (ACI) 318 (ACI, 2008) and American Association of State Highway and Transportation Officials (AASHTO, 2002).

The Canadian concrete design in regards to skin reinforcement (CSA A23.3-04b clause 10.6.2) states that if the beam height (h) exceeds 750 mm, then skin reinforcement shall be provided. The skin reinforcement needs to be evenly distributed along the side faces of the beam between 0.5h and the flexural reinforcement (Figure 2-23). The total area of skin reinforcement ($A_{sk}$) is calculated using Equations 2-27 and 2-28 with reference to Figure 2-23. ‘$A_{sk}$’ is the area of skin reinforcement, ‘$\rho_{sk}$’ is skin reinforcement ratio.
(Equation 2-19), \( A_{cs} \) is the effective area of concrete (shaded area in Figure 2-23), \( j \) is the skin reinforcement cover, \( d_{st} \) is the effective depth, and \( h \) is the beam height.

\[
A_{sk} = \rho_{sk}A_{cs} \quad (2-27)
\]
\[
A_{cs} = 8jd - 6jh \quad (2-28)
\]

CSA A23.3 (2004b) concrete design standard recommends a minimum skin reinforcement ratio (\( \rho_{sk} \)) of 0.8\% for interior exposure and 1.0\% for exterior exposure (CSA A23.3, 2004). The spacing of the skin reinforcement (\( s \)) shall not exceed 200 mm and the value of \( 2j \) shall not exceed half the beam width (CSA A23.3, 2004b). CSA A23.3-04b clause 10.6.2 is based on experiments carried out on side face beam cracking by Frantz and Breen (1980).

CSA-S6-06 clause 8.12.4 (CHBDC, 2006) states that if \( h \) exceeds 750 mm, longitudinal reinforcement shall be evenly distributed over both faces up to 0.7\( h \). The total area of the
skin reinforcement shall not be less than 0.01bd and the spacing of skin reinforcement shall not exceed 200 mm. The value of ‘b’ shall not be taken greater than 250 mm.

The CSA A23.3-04b and the CSA S6-06 with regards to side face reinforcement are very similar with minor differences. The CSA A23.3 (2004b) standard only requires skin reinforcement up to 50% of the beam height whereas the CHBDC requires skin reinforcement up to a longer height (70% of the beam height). In addition, the CSA A23.3-04b standard requires the total skin reinforcement area be a minimum of 0.8% of the effective concrete area $A_{cs}$ whereas the CSA S6-06 requires a minimum of 1.0% of bd. Under the assumption of a minimum beam width is 160 mm (CSA S6 (commentary), 2006, Table 8.20.4) and the effective depth (d) equals 0.9h, a minimum $\rho_{sk}$ of 1.3% of longitudinal steel has to be provided between 0.7h and the extreme tension zone fibre. According to CSA S6-06, in the majority of cases, a $\rho_{sk}$ of 2 to 3% is required. A probable reason for the CSA-S6-06 requiring a higher amount of skin reinforcement is because highway bridge girders are exposed to harsher weather conditions and therefore, require a higher degree of crack control. Further, highway bridge girders are typically very large in height; hence, side face beam cracking is more of a critical issue.

The American concrete standard (ACI 318, 2008, clause 10.6.7) states that when ‘h’ exceeds 36 in (914 mm), skin reinforcement shall be uniformly distributed along both side faces extending to a distance of 0.5h from the tension face. The maximum allowable spacing of the skin reinforcement (s) can be calculated in Equation 2-29, where $f_s$ is steel stress (psi) and ‘j’ is concrete cover (in).

$$s = 15 \left( \frac{40000}{f_s} \right) - 2.5j \leq 12 \left( \frac{40000}{f_s} \right) \text{(in)} \quad (2-29)$$

Bar size is not specified in clause 10.6.7 of the ACI 318 standard. However, bar No. 3 to No. 5 (No. 10 to No. 15 Canadian equivalent) are typically used. It was determined that bar spacing and concrete cover are the primary factors for crack control and effect of bar
size is relatively insignificant. This conclusion is supported by experimental studies carried out by Frantz and Breen (1980) and Broms (1965b).

The 17th edition of the Standard Specifications for Highway Bridges (American Association of State Highway and Transportation Officials or AASHTO, 2002, clause 8.17.2.1.3) states that if the depth of the side face of a member exceeds 36 in (914 mm), skin reinforcement shall be evenly distributed along both sides extending up to 0.5d from the flexural reinforcement. The skin reinforcement area per foot depth shall be greater than 0.012(d – 30). The maximum skin reinforcement spacing shall not exceed the lesser of d/6 or 12 in (305 mm). The ACI 318 and the AASHTO code requirements are similar except that the AASHTO requires a minimum area of reinforcement per foot depth, also the bar spacing requirements are different. The minimum area of skin reinforcement per foot increases as the depth of the beam increases. The bar spacing requirement is directly related to the beam depth and will be between 6 in and 12 in (152 mm and 305 mm) whereas the bar spacing requirement for the ACI 318 code depends upon the concrete cover. The ACI code allows for a maximum bar spacing of 12 in (305 mm) for concrete covers up to 1.2 in (30 mm) at a service steel stress of 40 ksi (275 MPa). The maximum bar spacing allowed for both codes is 12 in (305 mm) assuming 40 ksi (275 MPa) steel stress.

2.13 International Masonry Codes and Standards

The four masonry design codes that were found in the literature were: CSA S304.1 (2004a), MSJC (2008), BSI (2005), and AS 3700 (2007). CSA S304.1 (2004a) is the only standard that mentions the use of intermediate reinforcement to control intermediate cracking. The MSJC (2008), BSI (2005), and AS 3700 (2007) do not have any guidelines for controlling flexural cracking in masonry beams. The use of intermediate reinforcement in large masonry beams is similar to the use of skin reinforcement in large concrete beams. The use of skin reinforcement in large concrete beams will therefore be used as guidance for the use of intermediate reinforcement in large masonry beams.
2.14 Summary

Based on the previous research and experiments that have been conducted, it can be concluded that side face beam cracking behaviour is a function of concrete cover, steel stress, and tension zone depth. It has been shown that the North American Design Codes/standards for concrete and past research are in agreement in a general sense with minor differences. The Canadian standard CSA S304.1 (2004a) recognizes the problem of intermediate cracking in large masonry beams. The ideologies and theories about skin reinforcement in concrete beams will be applied to intermediate reinforcement in masonry beams during the scope of this experiment.
3 Experimental Program

3.1 Introduction

The main purpose of this experimental program is to study the effectiveness of intermediate reinforcement in large (h > 600 mm) masonry beams. A total of fifteen prism specimens and eight beam specimens were tested and data was analyzed. The experimental program was divided into two phases. The first phase consisted of constructing and testing fifteen prisms and four beam specimens (B1 to B4). The second phase consisted of constructing and testing of the remaining four beam specimens (B5 to B8). Fifteen prisms were constructed at the University of Windsor Structures lab. Beam specimens B1 to B4 of phase one were constructed offsite and transported to the University of Windsor. Two beams of phase one were damaged during handling and shipping (Section 3.5.2.4) hence, beam specimens B5 to B8 were constructed at the University of Windsor. All prism and beam specimens were tested at the University of Windsor Structures lab.

3.2 Materials

3.2.1 Masonry Units

3.2.1.1 Masonry Block Cutting

The placement of intermediate reinforcement in the masonry beams required part of web of the masonry units to be cut out. The webs were cut using a wet saw. Two cuts were made and they were about 100 mm deep and at 35 mm away from the centre width of the block on each side. A hammer was then used to knock out the middle portion to make room for intermediate reinforcement. Figure 3-1 shows the various steps of cutting a unit and placing the intermediate bar.
3.2.1.2 Compression and Absorption Testing Procedure

The masonry units for this research project were supplied by Santerra Stonecraft. 3 types of blocks were supplied and they are: regular stretcher blocks, splitter blocks, and lintel blocks (Figure 2-2). The experimental program consisted of casting 40 palettes of 75 blocks (50 stretcher and 25 splitter units). The stretcher blocks were the most abundant unit in the experiment and used 81% of the time. The lintel blocks were used for the bottom course of beam specimens and used 17% of the time. The splitter blocks were used at the end of beam specimens and used 2% of the time. The use of splitter blocks in masonry beams is not permitted because the small cavity at mid-length of the block may not be fully grouted and reduce beam strength (CSA S304.1, 2004a, Clause 11.2.1.6).
However, the splitter blocks were used as half blocks at the ends of the beams where beam strength is not important. Actual dimensions of each block were 390 mm long x 190 mm wide x 190 mm high (nominal dimensions were 400 mm long x 200 mm wide x 200 mm high). The specified compressive strength of the stretcher blocks was estimated to be 20 MPa. Fifteen stretcher blocks were chosen at random for absorption and strength tests. Tests were conducted at Santerra Stonecraft lab with the help of a certified technician. The testing was completed in accordance with ASTM (ASTM C140, 2010b) and CSA (CSA A165.1, 2004c). Each block was initially labeled, measured, and weighed to determine the length, width, height, faceshell and web thicknesses, and the initial weight ($W_i$). The first five blocks were immersed in water for 24 hours to ensure full saturation. The five blocks were than weighed under water to determine the immersed weight ($W_i$), then they were taken out of the water and surface dried with a damp cloth for a minute and weighed again to determine the saturated weight ($W_s$). Figure 3-2 shows the block being weighed and placed in water for saturation.

These five units were then dried in a kiln set at 100 °C for slightly over 24 hours and weighed again to determine the dry weight ($W_d$). The remaining ten blocks were used for compressive tests. These blocks were first capped using “hydrocal 105”, a gypsum
cement, and lightly greased glass plates. A level was used to ensure a flat surface for uniform loading (Figure 3-3). The test setup is shown in Figure 3-4a.

![Figure 3-3: Capping and Leveling of Block](image)

The blocks were subjected to monotonically increasing compression load until failure occurred due to crushing. The failure occurred fast and the failure mode was brittle. A typical failed masonry block is shown in Figure 3-4b. A metal barrier was used during each test for safety. Each compression test took 30 seconds to 1 minute to complete.

![Figure 3-4: Compression Test of Masonry Block](image)
3.2.2 Sand, Cement, and Lime

The materials used to make mortar and grout were sand, cement, and hydrated lime. Type 10 (GU) Portland cement was used in the mortar and grout, which was suitable according to the standard (CSA A179, 2004d, Clause 5.2.1). The cement and sand were supplied by St. Marys Cement Inc. and Forwell Materials Inc., respectively. The cement was delivered in 40 kg bags. The sand was delivered in loose form for phase 1 and in 20 kg bags for phase 2. The hydrated lime was locally purchased from Target Building Materials in 50 lbs bags. The cement and hydrated lime used were in accordance with the standard (CSA A179, 2004d). The cement, sand, and hydrated lime are shown in Figure 3-5. A sieve analysis was performed on the sand used for the mortar and grout to ensure the aggregate gradation adhered to CSA (CSA A179, 2004d). A sample of about 2 kg was taken from the loose sand. The sample was then put into a kiln for 24 hours at 100 °C for drying. From the 2 kg sample, 400 mg was then poured on the top of the stacked sieves and machine shaken for a few minutes. The weight of sand on each sieve was then measured and the percentage passing calculated. This procedure was repeated three times to ensure consistent results. The same procedure was also used for bagged sand in phase two.

(a): Cement  
(b): Sand  
(b): Hydrated Lime

Figure 3-5: Grout and Mortar Base Materials
3.2.3 Mortar

The mortar used in this experiment was a mixture of sand, cement, hydrated lime, and water. The volumetric proportions of the mortar mix was 1:0.5:4 (cement : hydrated lime : sand) as suggested by the standard for Type S mortar (CSA A179, 2004d, Table 3). A mass density of 640 kg/m$^3$ for hydrated lime, 1505 kg/m$^3$ for cement, 1280 kg/m$^3$ for sand, and 1000 kg/m$^3$ for water was used to calculate mass ratio (CSA A179, 2004d, Table 2). The batch size was calculated for a wheel barrow and is shown in Table 3-1. Enough water was added to attain 115% mortar flow on the flow test. The amount of water added in various batches varied a little depending upon the mortar flow. Mortar flow was measured using the flow test shown in Figure 3-6. The form shown in Figure 3-6a has a 100 mm diameter at the base, 70 mm diameter at the top, and 50 mm in height. An increased diameter of 215 mm is a 115% mortar flow. The flow test procedure was done in accordance with CSA (CSA A3005, 2008). The mortar was also poured into metal forms that were cubed shaped with a 50 mm dimension (Figure 3-6c). The metal forms were removed one week after casting. The mortar cubes were used for compression tests.

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass (kg)</th>
<th>Volume Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>28.5</td>
<td>4</td>
</tr>
<tr>
<td>Cement</td>
<td>7.7</td>
<td>1</td>
</tr>
<tr>
<td>Hydrated Lime</td>
<td>1.6</td>
<td>0.5</td>
</tr>
<tr>
<td>Water</td>
<td>6.8</td>
<td>As needed</td>
</tr>
</tbody>
</table>

Table 3-1: Mortar Mix
3.2.4 Grout

A fine grout was used in this experiment which was a mixture of sand, cement, and water. Volume ratio of 3:1 (sand : cement) was used for the grout mix as recommended by the standard (CSA A179, 2004d, Table 5). A mass density of 1505 kg/m$^3$ for cement, 1280 kg/m$^3$ for sand, and 1000 kg/m$^3$ for water was used to calculate mass ratio (CSA A179, 2004d, Table 2). The grout was mixed at the University of Windsor with a 0.1 m$^3$ mixer and at a storage facility offsite with a 0.25 m$^3$ mixer. Table 3-2 presents the grout mixtures by mass (kg). Enough water was added to attain a slump slightly above 250 mm. The slump was measured using the slump test as shown in Figure 3-7a. The slump test procedure was performed in accordance with the standard (CSA A23.2-5C, 2004e). Grout was also poured into 100 mm diameter by 200 mm high plastic cylinders and
masonry unit for compressive testing (Figure 3-7b). The grout samples were plastic covered for one week for curing. The grout samples were then drilled out from the masonry units as shown in Figure 3-7c and the plastic molds were removed from the remaining grout samples. The grout cores were 45 mm in diameter and 90 mm in height (ASTM C1019, 2011). All the grout samples were then labeled and stored until test day (Figure 3-7d).

Table 3-2: Grout Mixes

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass (kg) (0.1 m³)</th>
<th>Mass (kg) (0.25 m³)</th>
<th>Volume Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>133.0</td>
<td>372.4</td>
<td>3</td>
</tr>
<tr>
<td>Cement</td>
<td>50</td>
<td>140</td>
<td>1</td>
</tr>
<tr>
<td>Water</td>
<td>26.9</td>
<td>75.2</td>
<td>As needed</td>
</tr>
</tbody>
</table>

Figure 3-7: Grout Slump Test and Sampling
3.3 Steel Reinforcement

Grade 400R steel reinforcement was used for flexural, intermediate, and shear reinforcement. The bar sizes used in this experiment was No. 20 (20 mm diameter) for flexural reinforcement, No. 15 (15 mm diameter) for intermediate reinforcement, and No. 10 (10 mm diameter) for shear reinforcement. A tension test was performed on a sample of steel reinforcement (Figure 3-8a) to determine the stress-strain curve for the steel as well as the yield strength and strain and modulus of elasticity. The tension test was performed in accordance with ASTM (ASTM A370, 2010a). An extensometer was used to measure the strain during the test as shown in Figure 3-8b. A total of three tests were completed to ensure consisted results were obtained.

![Steel Tension Test Setup](image)

Figure 3-8: Steel Tension Test Setup

3.4 Prism Specimens

3.4.1 Specifications

All prism specimens were 790 mm tall, 390 mm long, and 190 mm wide. Three types of prisms were constructed as shown in Figure 3-9. The prisms were loaded in compression to determine the specified compressive strength ($f_{cm}^*$) and modulus of elasticity ($E_m$) of the masonry assemblage. A summary of the prism specifications can be seen in Table 3-3 with reference to Figure 3-9.
3.4.2 Construction

The prisms were constructed with the help of an experienced mason from the Canada Masonry Design Centre (CMDC). Plastic sheet was laid out on the floor to make sure mortar and grout did not stick to the floor. Sand, cement, hydrated lime and water was mixed in wheel barrows to make Type S mortar in accordance with the standard (CSA A179, 2004d, Table 3). Six mortar cube samples were cast for each type of prism for compression testing and quality assurance. The mortar cubes were cast in metal forms that had dimensions of 50 mm x 50 mm x 50 mm. A flow test was performed on every mortar batch to ensure the flow measured at 215 mm to 230 mm (Figure 3-6b). The prism specimens took 1.5 hours to construct, after which they were allowed to set. Approximately one week later, prism specimens P6 to P15 were grouted. The grout was mixed using an electric mixer at the University of Windsor. Wheel barrows were used to
transport the grout and scoops were used to fill the prisms with grout. Thin wooden poles were used to rod the grout to ensure all the voids were filled. The construction of the prism specimens can be seen in Figure 3-10.

![Figure 3-10: Construction of Prism Specimens](image-url)

### 3.4.3 Test Setup

The test setup of the prisms included a 75 mm thick bottom capping plate, a 50 mm thick top capping plate, another two 100 mm thick steel loading plates (top and bottom), four linear potentiometers, a 3000 kN load cell, a loading jack, and a loading frame. The prisms were capped using gypsum cement called ‘Hydrocal 105’ purchased locally from Target Building Materials. The Hydrocal 105 was mixed with water for good workability and spread on the bottom capping plate. The prism was then placed on the bottom
capping plate and leveled. The same procedure was carried out for the top capping plate. Two 100 mm thick steel loading plates were used on the top and bottom of the prism specimen to ensure uniform loading was achieved. The load was applied using a spherical head to ensure loading was centered. The 3000 kN load cell was first calibrated and then used during testing to measure applied load. Four linear potentiometers, two on each face, were mounted on the prism to measure the shortening of the prism over a 500 mm distances during testing. Each linear potentiometer was setup 75 mm away from the edge of the prism and vertically centered across three mortar joints. Steel wires were attached to the device at one end and fixed to a screw at the other end. The maximum travel of the linear potentiometers was 12.5 mm and the effective gauge length was 500 mm (Figure 3-12). Based on previous prism tests, a maximum travel of 1 mm was observed before the linear potentiometers were removed to avoid damages (Ring, 2009). Figure 3-11 illustrates the test setup. The lab technician helped to ensure the test was carried out properly and safely.
3.5 Beam Specimens

3.5.1 Specifications

The purpose of testing the beam specimens is to monitor and measure crack patterns and crack widths at different locations in the beam and at various load levels to determine how the crack widths are influenced by the intermediate reinforcement. The span to depth ratio (L/h) of each beam was kept above eight to ensure flexural beam action occurs. The beams also had to be taller than 790 mm (4 courses) to ensure side face cracking would form. Single-legged stirrups were placed in the shear spans of the beams to ensure shear failure did not occur. The beam specimens were 990 mm tall and 8.8 m long for B100 series specimens and 1190 mm tall and 10 m long for B120 series specimens. Each beam was labeled with a ‘B’ for beam, 100 or 120 for the height in centimeters, ‘N’ for no intermediate bar and ‘Y’ for a single No. 15 intermediate bar, and the last number represents the beam number for that type. All eight beam specimens were 190 mm wide and reinforced with 2 – No. 20 ($A_s = 600 \text{ mm}^2$) deformed bars and No. 10 stirrups at 400 mm spacing (every alternate cell). The flexural reinforcement was placed at 90 mm above the bottom of the beam. The intermediate reinforcement was placed at 490 mm above the bottom of the beam (Figure 3-8). The beam specifications and intermediate bar details are shown in Table 3-4 with reference to Figures 3-14 and 3-15.
Table 3-4: Beam Specimen Matrix

<table>
<thead>
<tr>
<th>ID</th>
<th>L (m)</th>
<th>h (mm)</th>
<th>( \frac{L}{\bar{h}} )</th>
<th>( d_a ) (mm)</th>
<th>a (m)</th>
<th>Intermediate Reinforcing Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Type</td>
</tr>
<tr>
<td>B100N1</td>
<td>8.4</td>
<td>990</td>
<td>8.5</td>
<td>900</td>
<td>3.7</td>
<td>-</td>
</tr>
<tr>
<td>B100N2</td>
<td>8.4</td>
<td>990</td>
<td>8.5</td>
<td>900</td>
<td>3.7</td>
<td>-</td>
</tr>
<tr>
<td>B100Y1</td>
<td>8.4</td>
<td>990</td>
<td>8.5</td>
<td>900</td>
<td>3.7</td>
<td>1 x No. 15</td>
</tr>
<tr>
<td>B100Y2</td>
<td>8.4</td>
<td>990</td>
<td>8.5</td>
<td>900</td>
<td>3.7</td>
<td>1 x No. 15</td>
</tr>
<tr>
<td>B120N1</td>
<td>9.6</td>
<td>1190</td>
<td>8.1</td>
<td>1100</td>
<td>4.3</td>
<td>-</td>
</tr>
<tr>
<td>B120N2</td>
<td>9.6</td>
<td>1190</td>
<td>8.1</td>
<td>1100</td>
<td>4.3</td>
<td>-</td>
</tr>
<tr>
<td>B120Y1</td>
<td>9.6</td>
<td>1190</td>
<td>8.1</td>
<td>1100</td>
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<td>1 x No. 15</td>
</tr>
<tr>
<td>B120Y2</td>
<td>9.6</td>
<td>1190</td>
<td>8.1</td>
<td>1100</td>
<td>4.3</td>
<td>1 x No. 15</td>
</tr>
</tbody>
</table>

Figure 3-13: Sketches of Beam Specimens

(a): B100 Series

(b): B120 Series
The beams were designed with minimum flexural reinforcement ratio (0.35% for B100 beams and 0.29% for B120 beams) to maximize the tension zone depth ($d_t$). The strength contribution of the intermediate bar was included to minimize error between predicted and actual yield moment. The yield moment ($M_{y}$) is defined as the moment resistance at the yielding of the flexural reinforcement. The following assumptions were made to calculate the yield moment of the beams:

- Linear Strain Distribution (plane sections before bending remain plane after bending).
- Strain Compatibility (perfect bond between concrete and steel).
- Linear stress distribution for masonry in compression zone is assumed.
- Masonry tensile strength neglected.
• The specified compressive strength ($f_{m'}$) was 10.7 MPa (Table 4-6). The specified compressive strength ($f_{m'}$) was obtained from grouted prism tests for load applied parallel to bed joint.

• The masonry compressive strain at yield ($\varepsilon_{my}$) was obtained from beam tests (Table 5-1)

• The yield strength ($f_y$), modulus of elasticity ($E_s$), and yield strain ($\varepsilon_y$) of steel reinforcement are 495 MPa, 215 GPa, and 0.0023, respectively. These values were obtained from tension test on steel reinforcement as shown in chapter 4.5 (Figure 4-12).

• The strain in the flexural reinforcement is 0.0023 at yield moment

• The $\chi$ factor and all safety factors were taken as unity.

The full calculation for beam specimen B120Y1 is shown to demonstrate how the moment capacity was calculated. The same procedure was followed for the other beam specimens:

1) Calculate the compression zone depth using the following equation

$$c = \frac{\varepsilon_{my}}{\varepsilon_{my} + \varepsilon_y} (d) = \frac{0.0008}{0.0008 + 0.0023}(1100) = 284 \text{ mm} \quad (3-1)$$

2) If there is an intermediate bar, calculate the strain in intermediate bar.

$$\varepsilon_{int} = \frac{(d_{int} - c)}{(d - c)} (\varepsilon_y) = \frac{(700 - 275)}{(1100 - 275)}(0.0023) = 0.0012 \quad (3-2)$$
3) Calculate yield moment

\[ M_{ry} = A_s f_y \left( d - \frac{c}{3} \right) + A_{\text{int}} e_{\text{int}} E_s \left( d_{\text{int}} - \frac{c}{3} \right) \]

\[ = (600)(495) \left( 1100 - \frac{284}{3} \right) \]

\[ + (200)(0.0012)(215000) \left( 700 - \frac{284}{3} \right) \]

\[ = 330.5 \text{ kN} \cdot \text{m} \]  

\[ (3-3) \]

3.5.2 Construction

3.5.2.1 Reinforcement Cage

The flexural reinforcement consisted of one level of 2 No. 20 at 50 mm on centre placed at 90 mm from the bottom of the beam. The intermediate bar was placed at 400 mm above the flexural reinforcement (Figure 3-14). Stirrups of No. 10 bar were provided at 400 mm spacing on centres in the shear span (a) for shear reinforcement. Beams in B100 series were constructed with 900 mm long single legged stirrups with a standard 180° hook at each end of the stirrup (Figure 3-15b). These beams had eighteen stirrups, nine on each shear span (a) starting at 100 mm from each side of the beam (Figure 3-13a). Beams in B120 series were provided with 1100 mm long stirrups with a standard 180° hook at each end of the stirrup. These beams had twenty-two stirrups, eleven on each shear span (a) starting at 300 mm from each end of the beam (Figure 3-13b). At the time of construction, the maximum bar length available was 9 m. For the B120 series, a required bar length of 10 m was necessary. Hence, the flexural reinforcing bars for beams in B120 series were spliced together using spot welds with an overlap of 800 mm on each side (Figure 3-15a). A general purpose electrode with specified yield strength of 490 MPa was used to splice the bars together. The weld was located between 500 mm and 1300 mm from each end of the beam. It was determined that relatively low flexural stresses would develop in this area, hence, the weld length was adequate for shear transfer between the two bars. The moment capacity at the spliced locations is expected to have doubled. However, it was also determined that the effect of the reinforcement splices is would be minimal due to the low flexural stresses. The flexural reinforcement was
marked with chalk at the stirrup locations and spaced using wood spacers (Figure 3-15c). The stirrups and flexural reinforcement were tied together using bar ties as is shown in Figure 3-15d.

3.5.2.2 Block Laying

The beams were constructed in two phases. Phase one consisted of B100 specimen and phase two consisted of B120 specimen. The beams were constructed with the help of an experienced mason provided by the Canada Masonry Design Centre (CMDC). A bed of mortar was laid down on the floor to level the first course of the beams for accurate construction. A polyethylene sheet was placed between the mortar and the unit to ensure the leveling could be easily removed prior to testing. Each phase of the beam was
constructed in a two-day period. The lintel blocks were laid the first day and allowed to set (Figure 3-16a). On the second day, the reinforcement cage was placed into the lintel blocks (Figure 3-16b) and the remaining courses were laid (Figure 3-16d). Cut masonry units (Figure 3-1c) were used to allow intermediate reinforcement to be placed in the B100Y and B120Y beam specimen series (Figure 3-16c).

Steel plates were welded to the reinforcement of each end of the beam. The steel plates were 200 mm x 200 mm and 10 mm in thickness. The purpose of the steel plates was to ensure the steel reinforcement was anchored at the ends of the beams and no bond failure occurs during loading.
3.5.2.3 Grouting

The beam specimens were filled with fine grout (Section 3.2.4) to bond the reinforcing bars to the masonry and to add additional compression area. Beams are to be solid or fully grouted in accordance with CSA S304.1-04a Clause 11.1.3. The beams were grouted approximately one week after the beams were built. The grout ingredients were weighed and mixed together until a uniform consistency was achieved (Figure 3-17a). The grout was then poured into the beams using wheel barrows and scoops (Figure 3-17c).

A slump test was performed on every batch of grout to ensure the water to cement ratio was correct and consistent (Figure 3-17b). Grouting of each phase of the beams took
approximately two days. After grouting, the beams were covered with plastic sheet for one week during the initial curing period (Figure 3-17d). The plastic sheet ensured that water was retained in the grout and did not evaporate for complete hydration of the cement. The beams were allowed to cure for minimum of 28 days before testing.

3.5.2.4 Transportation

B120 beam specimen series were constructed offsite and transported to the University of Windsor for testing. During transportation, beam specimens B100N1 and B100Y2 were damaged. B100N1 beam was damaged on the truck during transportation. B100Y2 was damaged by sliding the beam before grouting. The damage is shown in Figures 3-19 (a) and (b). The crack on B100Y2 started at just outside the constant moment region at the top and continued down in a step-wise manner towards the centerline of the beam. B100Y2 was repaired with the help of a mason. Each unit with a cracked mortar joint was taken out, the dry mortar was scrapped off and new mortar was applied. B100N1 was repaired using a two-part epoxy called “Sikadur 330” (Figure 3-19a). A thin trowel was used to insert the epoxy into the crack. The repaired crack is shown in Figure 3-19b. The crack in B100N1 was located approximately 2 meters away from the midspan and hence, outside the constant moment region.

(a): B100N1 Damage  (b): B100Y2

Figure 3-18: Damages to Beams
3.5.3 Test Setup and Procedure

3.5.3.1 Test Setup

A four-point bending load was applied. The constant moment region was 1000 mm long. A load jack was used to apply bending load through a steel spreader beam (Figures 3-22a). The spreader beam was a 1200 mm long W200 x 52 steel section that had four web stiffeners at 300 mm spacing. A 305 mm x 205 mm x 10 mm steel bearing plate (Plate 1 in Figure 3-22a) was used at the top centre of the spreader beam where the load jack was positioned. One pin support and one roller each with a 250 mm x 200 mm x 19 mm steel bearing plate (Plate 2 in Figure 3-22a) was used at each end of the spreader beam on the bottom side (Figure 3-22a). The beams had a roller and pin support positioned 200 mm from each end of the beam (Figure 3-22 (b) and (c)). The supports consisted of 400 mm steel stands and 200 mm x 200 mm x 50 mm steel bearing plates (Plate 3 in Figure 3-22(b) and (c)). The beam test setup is illustrated in Figures 3-20, 3-21, and 3-22.
Figure 3-20: B100 Series Beam Test Setup

Figure 3-21: B120 Series Beam Test Setup
3.5.3.2 Instrumentation

The test data collected during each beam test included; load, deflection, strain in the flexural and intermediate reinforcement, strain in the masonry distributed along the height of the beam (Figures 3-25b and 3-26b), and crack width. Test data was obtained using the Dalite Data Acquisition System with a scan rate of one scan/second. A total of
five load cells were used to monitor load and reactions during each beam test. Loadcell 1 was used at the load jack, Loadcells 2 and 3 were used at the point loads under the spreader beam, and Loadcells 4 and 5 were used at the supports (Figures 3-21, 3-22, and 3-23). Loadcells 2 and 3 had a 220 kN (50,000 lbs) capacity and Loadcells 1, 4, and 5 had a 440 kN (100,000 lbs) capacity. The deflections were measured using six linear variable displacement transducers (LVDT). LVDT 1 measured load jack displacement, LVDT 2 measured longitudinal displacement at roller support displacement, LVDT 3 and 5 measured deflection at quarter points underneath of the beam, LVDT 4 measured deflection at mid-span underneath of the beam, and LVDT 6 measured lateral (out-of-plane) displacement at mid-span. LVDT 2 was set at 700 mm above the bottom fibre of the beam. LVDT 6 was set at 700 mm above the bottom fibre of beam. The LVDT locations can be seen in Figure 3-23. Steel strain was monitored using 120 Ω resistance strain gauges with a gauge length of 5 mm. Strain gauges were installed on the flexural steel bars at 100 mm from the mid-span on each side. Strain gauges were installed on the intermediate steel bar at 100 mm from the mid-span on each side. The strain gauges were applied using CC-36 kyowa adhesive. Each strain gauge was coated using an air drying polyurethane m-coat (Vishay micro-measurements) for protection against grout during construction.

Strain gauge locations are shown in Figure 3-24. Strain data over a large gauge length (630 mm - 740 mm) was also acquired using ten linear potentiometers (LP). Two LP were placed at the top horizontal face at mid-span of the beam spaced at 60 mm from each face of the beam (Figure 3-25). A gauge length of 630 mm was used for these two LP. The remaining eight LP were placed on the north vertical face of the beam at 110 mm spacing for B100 series beam specimens and at 150 mm spacing for B120 series beam specimens. A gauge length of 740 mm was used for the eight LP along the side face of the beam. The LP was fastened to the concrete surface using machine screws as shown in Figure 3-27. Each LP has a displacement range of 15 mm. Wax paper and light oil was used where the wire contacts the LP to ensure the wire did not stick.
Crack width was measured using two different instruments: Concrete clip gauge and Microscopic digital camera. The concrete clip gauges had a gauge length of 87 mm and was mounted to the vertical surface using 2-part steel-concrete epoxy (Lepage) (Figure 3-28a). The clip gauge locations for phase one and phase two beam specimens can be seen in Tables 3-5 and 3-6, respectively. In these tables, locations of clip gauges are identified using an alphanumeric grid shown in Figures 3-25 and 3-26 for Tables 3-5 and 3-6, respectively. The microscopic camera is shown in Figure 3-28c. The microscopic camera had a capability of 200 times zoom. Crack width pictures can be uploaded to the computer and the crack width can be measured using computer software called Dinocapture (Figure 3-28d). Crack width pictures were taken on the south face of the beam. A grid was created to identify the height and location of the pictures as shown in Figures 3-25, 3-26 and 3-28b.
Figure 3-23: LVDT Location

(a): B100 Series LVDT Locations

(b): B120 Series LVDT Locations

Figure 3-23: LVDT Location
Figure 3-24: Strain Gauge Locations

(a): Flexural and Intermediate Bar Strain Gauge Locations

(b): Stirrup Strain Gauge (west)

(c): Stirrup Strain Gauge (east)

(d): Flexural Bar Strain Gauges

(e): Intermediate Bar Strain Gauges

Figure 3-24: Strain Gauge Locations
Figure 3-25: North and South Face Test Setup for B100 Series
Figure 3-26: North and South Face Test Setup for B120 Series

(a): South Face
(b): North Face
(c): Top Face

Figure 3-27: Installation of Linear Potentiometers

(a): Linear Potentiometer
(b): North Face
Figure 3-28: Crack Width Measurement

(a): Crack Width Clip Gauge  (b): Crack Width Grid

(c): Microscopic Camera  (d): Crack Width Picture
Table 3-5: Crack Width Clip Gauge Locations for Phase One: B100 Beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Clip</th>
<th>Location</th>
<th>Schematic – at Midspan</th>
</tr>
</thead>
<tbody>
<tr>
<td>B100N1</td>
<td>1</td>
<td>CD100</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>CD500</td>
<td>1</td>
</tr>
<tr>
<td>B100N2</td>
<td>1</td>
<td>CD100</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>CD500</td>
<td>1</td>
</tr>
<tr>
<td>B100Y1</td>
<td>1</td>
<td>CD100</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>CD500</td>
<td>1</td>
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<tr>
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<td>CD100</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>CD500</td>
<td>1</td>
</tr>
</tbody>
</table>
Table 3-6: Crack Width Clip Gauge Locations for Phase Two: B120 Beams

<table>
<thead>
<tr>
<th>Beam</th>
<th>Clip</th>
<th>Location</th>
<th>Schematic – at Midspan</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1</td>
<td>DE100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>DE200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
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<td>DE400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>DE500</td>
<td></td>
</tr>
<tr>
<td>B120N2</td>
<td>1</td>
<td>BC300</td>
<td></td>
</tr>
<tr>
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<td>2</td>
<td>BC400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
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<tr>
<td></td>
<td>4</td>
<td>CD300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>CD400</td>
<td></td>
</tr>
<tr>
<td>B120Y1</td>
<td>1</td>
<td>BC300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>BC400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>CD200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>CD300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>CD400</td>
<td></td>
</tr>
<tr>
<td>B120Y2</td>
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<td>DE100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>DE200</td>
<td></td>
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<tr>
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<td>3</td>
<td>DE300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>DE400</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>DE500</td>
<td></td>
</tr>
</tbody>
</table>
3.5.3.3 Test Procedure

Displacement control method was used for loading beam specimens. First, each beam was loaded to 5 kN load by the load jack to make sure beam specimen was stabilized. After 5 kN of load was added, each beam specimen was checked for any cracks and marked. The beam was then loaded to 10 kN and was checked for cracks again. Once cracking initiated, the length and location of each crack within the grid was recorded along with the log number of the data acquisition system. Crack width pictures were then taken along the height of the beam. Camera’s magnification factor, location, and picture number were recorded for each picture taken. The crack width pictures were labeled by two letters and followed by a number depending on where the location of the picture. Crack width picture AB100, for example, was a picture taken between lines at ‘A’ and ‘B’ of the grid and 100 mm from the bottom of the beam. The beams were then loaded at 10 kN load intervals until yielding of the reinforcement occurred. Crack initiations, progressions and other cracking behaviour were recorded and the crack width pictures being taken and saved. The beams were also checked for shear cracks in the shear span. The beams were loaded to yield at which point the beam was considered to have failed. Further load was applied until a deflection of 35 to 40 mm was reached to achieve a well defined yield point. The maximum applied load was 110 kN for beams without intermediate reinforcement and 130 kN for beams with intermediate reinforcement. The clip gauges and LVDT 3, 4 and 5 were removed at 100 kN load to avoid any damages to the instrumentation. The cracks were then marked with a black marker. The beams were then unloaded and the crack spacing and lengths were recorded on the south face of the beam. Each beam test took approximately two hours to complete.
4 Material Property Results

4.1 Masonry Units

4.1.1 Absorption Testing Results

A total of five stretcher units were randomly selected for absorption tests. The purpose of the absorption tests is to determine the third property for classifying block types (Table 2-2). Saturated weight ($W_s$), dry weight ($W_d$), immersed weight ($W_i$), and block height ($H$) were used in Equations 4-1 to 4-5 and the values were recorded in Table 4-1 (ASTM C140, 2010). In these equations, $A$ ($kg/m^3$) is a measure of how much water (kg) was absorbed per unit volume of block ($m^3$), $A$ (%) is the amount of water (kg) absorbed as a mass percentage based on the saturated weight, $D_b$ ($kg/m^3$) is the dry density of the block, $V$ ($mm^3$) is the net volume of the block, and $A_b$ ($mm^2$) is the net surface area of the block.

\[
A (kg/m^3) = \left(\frac{W_s - W_d}{W_s - W_i}\right) \times 1000 \quad \text{(4-1)}
\]
\[
A (%) = \left(\frac{W_s - W_d}{W_s}\right) \times 100 \quad \text{(4-2)}
\]
\[
D_b (kg/m^3) = \left(\frac{W_d}{W_s - W_i}\right) \times 1000 \quad \text{(4-3)}
\]
\[
V (mm^3) = (W_s - W_i) \times 10^6 \quad \text{(4-4)}
\]
\[
A_b (mm^2) = \frac{V}{H} \quad \text{(4-5)}
\]

Table 4-1: Masonry Unit Data and Absorption Results

<table>
<thead>
<tr>
<th>Unit #</th>
<th>H (mm)</th>
<th>A ($kg/m^3$)</th>
<th>A (%)</th>
<th>$D_b$ (kg/m$^3$)</th>
<th>$V$ ($10^6 mm^3$)</th>
<th>$A_b$ ($10^3 mm^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>189</td>
<td>113.5</td>
<td>5.0</td>
<td>2141</td>
<td>7.40</td>
<td>39.2</td>
</tr>
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<td>2</td>
<td>189</td>
<td>113.2</td>
<td>5.0</td>
<td>2140</td>
<td>7.42</td>
<td>39.3</td>
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<td>3</td>
<td>190</td>
<td>121.0</td>
<td>5.4</td>
<td>2137</td>
<td>7.44</td>
<td>39.2</td>
</tr>
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<td>4</td>
<td>189.5</td>
<td>126.7</td>
<td>5.6</td>
<td>2119</td>
<td>7.42</td>
<td>39.2</td>
</tr>
<tr>
<td>5</td>
<td>189</td>
<td>113.5</td>
<td>5.0</td>
<td>2135</td>
<td>7.40</td>
<td>39.2</td>
</tr>
<tr>
<td>Avg.</td>
<td>-</td>
<td>117.6</td>
<td>5.2</td>
<td>2134</td>
<td>-</td>
<td>39.2</td>
</tr>
</tbody>
</table>

The length, height, and width of the masonry units were all within the permissible variations of the concrete block masonry unit dimensions according to the standard (CSA

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A165.1, 2004c, Table 4). The faceshell and web thicknesses of the masonry units were all above the minimum faceshell and web thicknesses specified in the standard (CSA A165.1, 2004c, Table 3). The average masonry unit had a 53% solid content and a 2134 kg/m$^3$ density. According to the 4 facet properties in the standard, the masonry units are designated as H/20/A/M (CSA A165.1, 2004c, Table 1).

### 4.1.2 Compression Test Results

A total of ten stretcher units were randomly selected for compression testing. All failed blocks were inspected for any unusual failure modes. The failure loads for nine masonry blocks were consistent ranging from 800 kN to 930 kN (Table 4-2). One masonry block (unit 8) failed below the range at 750 kN, it was determined that the failure mode for this specimen was inconsistent with the other specimens due to uneven loading conditions (Figure 4-1). As can be seen in Figure 4-1, the uneven loading conditions caused a premature failure to occur on one side of the unit.

![Figure 4-1: Uneven Failure](image)

The compressive strength for this specimen was disregarded for this reason, when determining the specified compressive strength. The specified compressive strength was calculated to be 20.1 MPa in accordance with the Canadian standard (CSA A165.5, 2004c, clause 10.2). The compressive strength, average compressive strength, standard deviation, coefficient of variation, and specified compressive strength are shown in Table 4-2 using Equations 4-6 to 4-10.
### Table 4-2: Masonry Stretcher Unit Compression Test Results

<table>
<thead>
<tr>
<th>Unit #</th>
<th>Maximum Load, Q (kN)</th>
<th>Strength, f (MPa)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>862</td>
<td>22.0</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>843</td>
<td>21.5</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>750</td>
<td>19.1</td>
<td>Discarded</td>
</tr>
<tr>
<td>9</td>
<td>826</td>
<td>21.1</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>844</td>
<td>21.5</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>836</td>
<td>21.4</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>796</td>
<td>20.3</td>
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<tr>
<td>13</td>
<td>829</td>
<td>21.2</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>870</td>
<td>22.2</td>
<td>-</td>
</tr>
<tr>
<td>15</td>
<td>930</td>
<td>23.7</td>
<td>-</td>
</tr>
</tbody>
</table>

- **Average, \( f_{av} \) (MPa)**: 21.7
- **Standard deviation, \( s_t \) (MPa)**: 0.95
- **Coefficient of variation, \( v \) (%)**: 4.4
- **Specified compressive strength, \( f_{bl} \) (MPa)**: 20.1

\[
f = \frac{Q}{A_b} = \text{Compressive strength} \tag{4-6}
\]

\[
f_{av} = \frac{\sum f}{n} = \text{Average compressive strength} \tag{4-7}
\]

\[
s_t = \frac{\sqrt{\frac{\sum (f - f_{av})^2}{n - 1}}}{n - 1} = \text{Standard deviation} \tag{4-8}
\]

\[
v(\%) = \frac{s}{f_{av}} \times 100 = \text{Coefficient of variation} \tag{4-9}
\]

\[
f_{bl} = f_{av} - 1.64s = \text{Specified compressive strength} \tag{4-10}
\]

### 4.2 Sand Sieve Analysis

The results of the sieve analysis performed on the loose and bagged sand are shown in Figures 4-2 and 4-3. The upper and lower aggregate gradation limits are also shown in Figures 4-2 and 4-3 (CSA A179, 2004d, Table 1). The ISO sieve sizes were not available; however similar sieve sizes were used which gave adequate results. The
following eight sieve sizes were used: 15.9 mm, 9.5 mm, 4.8 mm, 2 mm, 1 mm, 0.85 mm, 0.425 mm, 0.15 mm. According to the results, the loose sand is significantly coarser than the bagged sand. The loose sand sieve analysis results are in between the upper and lower aggregate gradation limits suggesting that the loose sand is well suited for mortar and grout (CSA A179, 2004d). The sieve analysis results of the bagged sand were at the upper aggregate gradation limit (CSA A179, 2004d). Hence, this result suggests that loose sand is more appropriate for grout and mortar than bagged sand according to the standard (CSA A179, 2004d). However, both loose and bagged sand were acceptable in accordance with CSA A179 2004d. The loose sand was used in phase one and the bagged sand was used in phase two. Based on the compression strength results shown in Tables A-1 to A-4 in Appendix A and Tables B-1 to B-4 in Appendix B, there was a negligible effect of sand type (loose or bagged) on the mortar and grout.

Figure 4-2: Sieve Analysis Results for Loose Sand (CSA A179, 2004d)
4.3 Mortar

A total of 102 mortar cubes were tested for compressive strength. The mortar cubes were cast from 34 batches (three mortar cubes per batch). 51 mortar cubes were tested 28 days after casting and the remaining 51 mortar were tested in between 40 to 180 days after casting (test day of prism or beam). All the mortar cubes were tested at the University of Windsor structures lab. Compression strength between each mortar batch can be seen in Appendix A, Tables A-1 to A-4. The test setup is shown in Figure 4-4a. Each mortar cube was loaded at a rate of 1 kN/sec and each test took 1 minute to complete. All mortar cubes failed due to crushing with an hour glass shape as shown in Figure 4-4b. The 28-day compression strength of the mortar cubes ranged from 14 to 29 MPa with an average strength of 21 MPa and coefficient of variation of 19.3% (Figure 4-5). The 40 to 180 day compression strength of the mortar cubes ranged from 16 to 33 MPa with an average strength of 23 MPa and coefficient of variation of 18.1% (Figure 4-6). The 28-day and 40 to 180-day (prism/beam test day) mortar strength data are plotted in a histogram in
Figures 4-5 and 4-6, respectively. The large difference in compression strength between each mortar batch is attributed to variations in water content. The amount of water added to each mortar batch varied slightly from Table 3-1 depending on the workability. All 102 mortar cube compressive strength results can be seen in Appendix A, Tables A-1 to A-4.

Figure 4-5:的积极性HS00Mortar Compression Test

(a): Test Setup
(b): Failed Mortar Cube

Figure 4-4: Mortar Compression Test

Figure 4-5: Histogram with Normal Distribution for 28-Day Mortar Strength

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4.4 Grout

A total of 108 grout specimens were tested for compressive strength. Fifty-four grout specimens were cast into plastic non-absorbent molds and 54 grout specimens were cast and cored out of masonry units. Among 54 specimens in each group, 27 grout specimens were tested at 28-day strength and remaining 27 grout samples were tested at 30 to 180-day strength (prism/beam test day). The grout specimens were capped before testing using a sulfur mortar to ensure a flat surface for uniform loading during compressive testing (Figure 4-7a). All grout specimens were tested at the University of Windsor structures lab.
The test setup for a grout cylinder is shown in Figure 4-7b. The grout cylinders were loaded at a rate of 4 kN/sec and the grout cores were loaded at a rate of 1 kN/sec. Each test took 1 minute to complete. A typical failed grout cylinder and grout core is shown in Figure 4-7 (c) and (d), respectively. The compressive strength for each grout batch can be seen in Appendix B, Tables B-1 to B-4. The 28-day compressive strength of the grout cylinders ranged from 25 to 40 MPa with an average strength of 33.3 MPa and 11.5% coefficient of variation (Figure 4-8). The 28-day compressive strength of the grout cores ranged from 28 to 45 MPa with an average strength of 36.7 MPa and 12.8% coefficient of variation (Figure 4-10). The 30 to 170 day (prism/beam test day) strength of the grout cylinders ranged from 23 to 40 MPa with an average strength of 30 MPa and a 17.4% coefficient of variation (Figure 4-9). The 30 to 180 day strength of the grout cores ranged
from 22 to 43 MPa with an average strength of 31.4 MPa and a 20.8% coefficient of variation (Figure 4-11). A large difference in compressive strength between grout batches is attributed to variations in water content. The amount of water added to each grout batch varied slightly from Table 3-2 depending on flow. All grout cylinder and core strength data can be seen in Appendix B, Tables B-1 to B-4. It is to be noted that cores and cylinders grout specimens were not made from the same batch.

Figure 4-8: Histogram of Grout Cylinders with Normal Distribution for 28-Day Strength
Figure 4-9: Histogram of Grout Cylinders with Normal Distribution for 30 to 180-Day Strength

Figure 4-10: Histogram of Grout Cores with Normal Distribution for 28-Day Strength
Figure 4-11: Histogram of Grout Cores with Normal Distribution for 30 to 180-Day Strength

4.5 Steel Reinforcement

A total of three steel specimens were tested to determine the stress-strain curve, yield strength ($f_y$), yield strain ($\varepsilon_y$), and elastic modulus ($E_s$). The yield strength, yield strain, and elastic modulus were taken as the average of each of the three tensile tests. The results from each tensile test were observed to have good consistency. The stress-strain curve from a tensile test are shown in Figure 4-12a. A linear stress-strain is seen up to the yield point. The yield stress and strain were taken at the tip of the linear portion of the stress-strain curve. The modulus of elasticity for steel was taken as the ratio of the average yield stress and yield strain. The average yield strength and strain and the modulus of elasticity were calculated to be 495 MPa, 2300 $\mu\varepsilon$, and 215 GPA, respectively. Beyond the yield point, strain hardening was observed to occur up to a maximum strength. Subsequent to this point, the steel specimen was observed to rapidly decrease in cross-sectional area and rupture occurred shortly after (Figure 4-12a).
Figure 4-12: Tensile Test Results and Failure

(a): Failed Specimen

\( f_y = 495 \text{ MPa} \)

\( \varepsilon_y = 2300 \mu \varepsilon \)

\( E_s = 215 \text{ GPa} \)

(b): Stress – Strain Curve for Steel

Figure 4-12: Tensile Test Results and Failure
4.6 Prisms

A total of fifteen prism specimens were tested to determine the specified compressive strength ($f_{m'}$) and elastic modulus ($E_m$) of the masonry assemblage. First five prisms (P1 to P5) were hollow and loaded normal to the bed joint, next five prisms (P6 to P10) were grouted and loaded normal to the bed joint, and the last five prisms (P11 to P15) were grouted and loaded parallel to the bed joint (Figure 3-11). All prism specimens were tested at the University of Windsor structures lab. Each prism was loaded in intervals of 50 kN with 3 to 5 seconds pause between each interval. During each test, when the load reached 60% of the expected failure load, the load was held and the linear potentiometers were removed to avoid any damages. After the linear potentiometers were removed, the test resumed until failure by crushing was reached. Each test took 5 to 10 minutes to complete.

4.6.1 Prism Specimens P1 – P5

Prism specimens P1 to P5 were tested 60 to 120 days after construction. Each prism specimen developed web cracks prior to failure (Figure 4-13a). The failure of P1 to P5 was very sudden and brittle. A typical failed prism specimen is shown in Figure 4-13b.

(a): Web Crack  (b): Failed Prism

Figure 4-13: Failure Mode of Prisms P1 to P5
A summary of data for prisms P1 to P5 is shown in Table 4-3. The compressive strength of each prism was calculated using the effective mortar bed area ($A_{mb}$). The effective mortar bed area ($A_{mb}$) was calculated from the overlap area of a single masonry unit and two half units (Figure 4-14) and is shown in Equation 4-13. The effective mortar bedded area for prisms P1 to P5 is shown as the shaded area in Figure 4-14.

![Figure 4-14: Effective Area for Hollow Prisms](image)

$$A_{mb} = 28191 \text{ mm}^2$$ (4-13)

The elastic modulus was calculated in accordance with the standard (CSA S304.1, 2004a, Annex D, clause D.4.6). The average strain data for each prism was calculated by the ratio of the average displacement of the four linear potentiometers and a gauge length of 500 mm. The elastic modulus ($E_m$) was calculated as the slope of the line between 5% and 33% of the mean compressive strength for the stress-strain curve (Figure 4-15). The method for which the elastic modulus was obtained for P2 is shown in Figure 4-15. The stress-strain relationship for prism specimens P1 to P5 can be seen in Appendix C, Figures C-1 to C-5. The value of $E_m$ was found to be 17946 MPa in Figure 4-15. The values of compressive strength and elastic modulus of prism P1 to P5 are shown in Table 4-3.
Table 4-3: Summary of Results for Prism Specimens P1 to P5

<table>
<thead>
<tr>
<th>Prism ID</th>
<th>Elapsed Time (days)</th>
<th>Strength (MPa)</th>
<th>Mean (MPa)</th>
<th>C.O.V. (%)</th>
<th>Modulus (MPa)</th>
<th>Mean (MPa)</th>
<th>C.O.V. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>63</td>
<td>26.3</td>
<td>25.7</td>
<td>7.8</td>
<td>23056</td>
<td>19165</td>
<td>16.9</td>
</tr>
<tr>
<td>P2</td>
<td>65</td>
<td>23.9</td>
<td></td>
<td></td>
<td>17200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P3</td>
<td>66</td>
<td>24.9</td>
<td></td>
<td></td>
<td>22296</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P4</td>
<td>67</td>
<td>24.7</td>
<td></td>
<td></td>
<td>16136</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P5</td>
<td>119</td>
<td>29.0</td>
<td></td>
<td></td>
<td>17135</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Based on the hollow, normal to the bed joint, prism results; the specified compressive strength can be calculated as from Equation 4-14 (CSA S304.1, 2004a, Annex C, Clause C.2.2).

\[ f'_m = f_{av}(1 - 1.64v) \]  \hspace{1cm} (4-14)
The specified compressive strength ($f_{m'}$) and average elastic modulus ($E_m$) of prisms P1 to P5 was found to be 22.4 MPa and 19165 MPa, respectively.

4.6.2 Prism Specimens P6 – P10

Prism Specimens P6 to P10 were tested in between 85 to 110 days after construction. The failure of the prisms occurred instantaneously and without any warning. The failure mode of the prism was caused by the crushing of the faceshell. De-bonding of the faceshell from the inner grout core also occurred at failure. A typical failure of prisms P6 to P10 is shown in Figure 4-16.

Figure 4-16: Failure Mode and Effective Area of Prism Specimens P6 to P10

(b): P7 Failure

(b): P8 Failure

(c): Effective Area for Grouted Prisms Loaded Normal to Bed Joint
A summary of data for prisms P6 to P10 is shown in Table 4-4. The compressive strength of each prism was calculated using the effective area ($A_{en}$). The effective area ($A_{en}$) was calculated to be $72025 \text{ mm}^2$ (Equation 4-15) from the shaded area in Figure 4-16c. The effective area of prisms P6 to P10 was calculated as the overlap area of a single masonry unit and two half units with the cells included, a small area of the frogged end is not included. The elastic modulus was calculated in accordance with the standard (CSA S304.1, 2004a, Annex D, clause D.4.6). The average strain data for each prism was calculated by the ratio of the average displacement of the four linear potentiometers and a gauge length of 500 mm. The elastic modulus was taken as the slope of the line between 5% and 33% of the mean compressive strength for the stress-strain curve (Figure 4-17). The method for which the elastic modulus was obtained for P9 is shown in Figure 4-17. The stress-strain relationship for prism specimens P6 to P10 can be seen in Appendix C, Figures C-6 to C-10. In Figure 4-17, the elastic modulus ($E_{m}$) was found to be 22413 MPa.

![Figure 4-17: Stress – Strain Relationship of P9](image)

$$A_{en} = 72025 \text{ mm}^2$$  \hspace{1cm} (4-15)
Table 4-4: Summary of Results for Prism Specimens P6 to P10

<table>
<thead>
<tr>
<th>Prism ID</th>
<th>Elapsed Time (days)</th>
<th>Strength (MPa)</th>
<th>Mean (MPa)</th>
<th>C.O.V. (%)</th>
<th>Modulus (MPa)</th>
<th>Mean (MPa)</th>
<th>C.O.V. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P6</td>
<td>85</td>
<td>13.7</td>
<td>14.8</td>
<td>13.0</td>
<td>21940</td>
<td>21983</td>
<td>7.7</td>
</tr>
<tr>
<td>P7</td>
<td>85</td>
<td>18.0</td>
<td>2</td>
<td>20736</td>
<td>20674</td>
<td>18042</td>
<td></td>
</tr>
<tr>
<td>P8</td>
<td>88</td>
<td>13.0</td>
<td>20736</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P9</td>
<td>107</td>
<td>14.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P10</td>
<td>110</td>
<td>14.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The prism strength results (Table 4-4) are all within 2 MPa except P7 which was 22% stronger than the second highest strength (prism P10). The increased strength of prism P7 is due to variations in material properties of grout and block. Based on the grouted, normal to the bed joint, prism results; the specified compressive strength can be calculated as from Equation 4-16 (CSA S304.1, 2004a, Annex C, Clause C.2.2).

\[ f'_m = f_{av}(1 - 1.64v) \]  \hspace{1cm} (4-16)

The specified compressive strength \( (f'_m) \) and average elastic modulus \( (E_m) \) of prisms P6 to P10 was found to be 11.6 MPa and 20674 MPa, respectively.

4.6.3 Prism Specimens P11 – P15

Prism Specimens P11 to P15 were tested in between 90 to 115 days after construction. The failure of the prisms occurred instantaneously and without any warning. The failure mode of the prism was caused by the crushing of the prism. De-bonding of the faceshell from the inner grout core was also characteristic of the failure. A typical failure of prisms P11 to P15 is shown in Figure 4-18.
A summary of data for prisms P11 to P15 is shown in Table 4-5. The compressive strength of each prism was calculated using the effective area ($A_{ep}$). The calculation of the effective area ($A_{ep}$) is shown in Equation 4-17, where 390 mm is the average prism width and 190 mm is the average prism thickness. The effective area for prisms P11 to P15 is shown in Figure 4-18c.

$$A_{ep} = (390)(190) = 74100 \text{ mm}^2$$

The elastic modulus was calculated in accordance with the standard (CSA S304.1, 2004a, Annex D, clause D.4.6). The average strain data for each prism was calculated by the ratio of the average displacement of the four linear potentiometers and a gauge length of 500 mm. The elastic modulus was taken as the slope of the line between 5% and 33% of the mean compressive strength for the stress-strain curve (Figure 4-19). The method for
which the elastic modulus was obtained for P13 is shown in Figure 4-19 as an example. The stress-strain relationship for prism specimens P11 to P15 can be seen in Appendix C, Figures C-11 to C-15.

Table 4-5: Summary of Results for Prism Specimens P11 to P15

<table>
<thead>
<tr>
<th>Prism ID</th>
<th>Elapsed Time (days)</th>
<th>Strength (MPa)</th>
<th>Mean Strength (MPa)</th>
<th>C.O.V. (%)</th>
<th>Modulus (MPa)</th>
<th>Mean Modulus (MPa)</th>
<th>C.O.V. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P11</td>
<td>94</td>
<td>11.9</td>
<td>13.5</td>
<td>12.7</td>
<td>15765</td>
<td>16773</td>
<td>18.0</td>
</tr>
<tr>
<td>P12</td>
<td>99</td>
<td>12.5</td>
<td></td>
<td></td>
<td>13480</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P13</td>
<td>100</td>
<td>14.7</td>
<td></td>
<td></td>
<td>19016</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P14</td>
<td>102</td>
<td>15.9</td>
<td></td>
<td></td>
<td>20751</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P15</td>
<td>112</td>
<td>12.5</td>
<td></td>
<td></td>
<td>14855</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4-19: Stress – Strain Relationship of P13
Based on the grouted, parallel to the bed joint, prism results, the specified compressive strength can be calculated as from Equation 4-18 (CSA S304.1, 2004, Annex C, Clause C.2.2).

\[ f_m' = f_{av}(1 - 1.64v) \]  

(4-18)

The specified compressive strength \((f_m')\) and average elastic modulus \((E_m)\) of prisms P11 to P15 was found to be 10.7 MPa and 16773 MPa, respectively.

### 4.6.4 Summary

In conclusion, a total of 15 prisms were tested to determine the specified compressive strength \((f_m')\) and elastic modulus \((E_m)\) of the three types of prisms: 1) hollow, normal to bed joint 2) grouted, normal to bed joint and 3) grouted, parallel to bed joint. A summary of the results can be seen in Table 4-6. This information was required for determining the behaviour of beam specimens. A comparison of the specified compressive strength \((f_m')\) acquired from prism tests and CSA S304.1-04, Table 4 is shown in Table 4-6. The specified compressive strength of the hollow prisms was observed to be much higher than the specified compressive strength from CSA S304.1, Table 4. Hence, this suggests that hollow masonry prism strength is under-estimated in the CSA S304.1-04 standard. The specified compressive strength of the grouted prisms loaded normal to the bed joint is also much higher than the specified compressive strength from the CSA S304.1-04 standard. The CSA S304.1-04 standard suggests a reduction of 50% in strength of the grouted prisms loaded normal to the bed joint when there is 100% horizontal grout interruption which is the case for the prisms tested in this experiment. However, this 50% reduction in specified compressive strength for grouted prisms loaded normal to the bed joint was observed to be very conservative according to the results shown in Table 4-6.
<table>
<thead>
<tr>
<th>Prism Type</th>
<th>Mean Compressive Strength (MPa)</th>
<th>( f_{m'} ) (MPa)</th>
<th>Mean ( E_m ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block</td>
<td>Grout</td>
<td>Mortar</td>
<td>Test</td>
</tr>
<tr>
<td>P1 – P5</td>
<td>21.7</td>
<td>-</td>
<td>19.9</td>
</tr>
<tr>
<td>P6 – P10</td>
<td>35.5</td>
<td>20.8</td>
<td>11.6</td>
</tr>
<tr>
<td>P11 – P15</td>
<td>24.8</td>
<td>24.1</td>
<td>10.7</td>
</tr>
</tbody>
</table>
5 Beam Test Results

5.1 Introduction

A total of eight beams were tested in two phases. Phase one consisted of beams B100N1, B100Y1, B100N2, and B100Y2 and phase two consisted of beams B120N1, B120Y1, B120N2, and B120Y2 (Table 3-4). Beam specimens for phase 1 were tested at 140 to 170 days after construction. Beam specimens for phase two were tested at 30 to 80 days after construction. Each beam took one to two weeks to setup and test. In this research, the beam was considered to have failed at the yield moment. A reference load ($P_r$) of 60 kN for B100 and B120N beams and 70 kN for B120Y beams was also used.

5.2 Beam Data Analysis

The applied load at yield ($P_y$) for the beams was between 110 kN to 130 kN. Based on the load applied at yield ($P_y$) and the self-weight ($q$), the applied moment at yield ($M_{ry}$) was calculated. Table 5-1 compares the applied moment at yield ($M_{ry}$) in the tests with the predicted yield moment ($M_{ry}$) calculated using the same procedure in chapter 3.4.1. The applied load and masonry strain at yield are also shown in Table 5-1. The applied load at yield ($P_y$) was taken from the load displacement plots (Figures 5-1 and 5-2) as the load at the beginning of the post yielding stage. The masonry strain was taken as the measured compressive strain at the applied load at yield ($P_y$) (Figures E-1 to E-8).

Table 5-1: Experimental vs. Predicted Yield Moment

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>$P_y$ (kN)</th>
<th>$\varepsilon_{my}$ (mm/mm)</th>
<th>Predicted $M_{ry}$ (kN-m)</th>
<th>Applied $M_{ry}$ (kN-m)</th>
<th>Diff (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B100N1</td>
<td>110</td>
<td>0.0008</td>
<td>240.3</td>
<td>238.8</td>
<td>-0.6</td>
</tr>
<tr>
<td>B100N2</td>
<td>110</td>
<td>0.0011</td>
<td>238.5</td>
<td>238.8</td>
<td>+0.1</td>
</tr>
<tr>
<td>B100Y1</td>
<td>120</td>
<td>0.0008</td>
<td>261.8</td>
<td>257.3</td>
<td>-1.7</td>
</tr>
<tr>
<td>B100Y2</td>
<td>120</td>
<td>0.001</td>
<td>255.6</td>
<td>257.3</td>
<td>+0.7</td>
</tr>
<tr>
<td>B120N1</td>
<td>110</td>
<td>0.0006</td>
<td>305.7</td>
<td>291.8</td>
<td>-4.5</td>
</tr>
<tr>
<td>B120N2</td>
<td>110</td>
<td>0.0006</td>
<td>305.7</td>
<td>291.8</td>
<td>-4.5</td>
</tr>
<tr>
<td>B120Y1</td>
<td>130</td>
<td>0.0008</td>
<td>330.5</td>
<td>334.8</td>
<td>+1.3</td>
</tr>
<tr>
<td>B120Y2</td>
<td>130</td>
<td>0.0008</td>
<td>330.5</td>
<td>334.8</td>
<td>+1.3</td>
</tr>
</tbody>
</table>
Based on the comparison, the predicted yield moment is seen to be within 5% of the applied moment at yield. Variations in material strength could also explain the differences in predicted and applied moments at yield.

5.2.1 Load-Displacement

The load-displacement curve for each beam type (Type 1: B100N1 and B100N2, Type 2: B100Y1 and B100Y2, Type 3: B120N1 and B120N2, Type 4: B120Y1 and B120Y2) can be seen in Figures 5-1 and 5-2 with reference to Figure 5-3. The displacement at mid-span was measured from LVDT 1 (Figure 3-23) in these plots. Beam specimens B100Y1 and B120Y1 (with intermediate reinforcement) exhibited higher stiffness and post yielding strength compared to beam specimens B100N1 and B120N1 (without intermediate reinforcement). The addition of intermediate reinforcement did provide additional benefit for strength and stiffness. Three distinct stages can be seen from the figures: Pre-cracking, Post-cracking, and Post-yielding. The pre-cracking stage ranged from 0 to 10 kN. The first flexural cracks were observed at about 10 kN for each beam. The post-cracking stage ranged from 10 kN to 110 kN for B100N1 and B120N1 and 10 kN to 120 kN for B100Y1 and 10 kN to 130 kN for B120Y2. A decrease in stiffness can be seen between pre and post – cracking stages. The post-yielding stage can be observed from the large decrease in stiffness and the rapid increase in deflection. The yield point defines the division between the post-cracking and post-yielding stages (Figures 5-1 and 5-2). The load deformation curves for all eight beams can be seen in Appendix D (Figures D-1 to D-8).
Figure 5-1: Load-Displacement Curve for B100N1 and B100Y1

Figure 5-2: Load-Displacement Curve for B120N1 and B120Y1
The deflection under the beam was measured using three LVDTs (LVDT’s 3, 4, and 5) placed at quarter points along the beam length (Figure 3-23). Figures 5-4 to 5-7 show the deflection of the beam at \( P = 60 \text{ kN} \) for B100 and B120N beams and \( P = 70 \text{ kN} \) for B120Y beams and at \( P = 110 \text{ kN} \) for BN beams, \( P = 120 \text{ kN} \) for B100Y beams, and \( P = 130 \text{ kN} \) for B120Y beams. The results from the data were as expected, the deflection of the two LVDT’s at quarter points remained very close in value. The deflection at 60 kN and 70 kN load for all the beams ranged from 8 mm to 12 mm at the mid-span. The experimental deflections were shown to be within 10% of the estimated deflection using Equations 5-1 and 5-2.

\[
\Delta = \frac{5qL^4}{384E_mI_{eff}} + \frac{(P^2) a}{24E_mI_{eff}} (3l^2 - 4a^2) \quad (5-1)
\]

\[
I_{eff} = \left( \frac{M_{cr}}{M_a} \right)^3 I_o + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_o \quad (5-2)
\]

\[
I_{eff(s)} = \left( \frac{1.6}{9.6} \right) (2I_{eff}) + \left( \frac{9.6 - 1.6}{9.6} \right) (I_{eff}) \quad (5-3)
\]

Equation 5-1 is derived from beam theory, where ‘L’ is beam span, ‘q’ is uniform dead load, \( E_m \) is elastic modulus, \( I_{eff} \) is the effective moment of inertia (CSA S304.1, 2004a, Clause 11.4.3.2), and ‘a’ is the shear span. The actual moment of inertia varies along the length of the beam due to the presences of flexural cracks. At cracked sections, the actual moment of inertia is the cracked moment of inertia. Between cracked sections, the
moment of inertia is between un-cracked and cracked moment of inertia. Hence, an effective moment of inertia is used as an average moment of inertia along the length of the beam. The effective moment of inertia is calculated by interpolating between I_{oc} (un-cracked moment of inertia) and I_{cr} (cracked moment of inertia) using the cubic ratio of the cracking moment (M_{cr}) and the applied moment (M_{a}). The beam specimens were observed to start cracking at 10 kN to 20 kN of applied load (P). Under the assumption that all beam specimens start to crack at 15 kN of applied load, the self-weight (q) for B100 and B120 series specimens was 4 kN/m and 4.8 kN/m, respectively, the modulus of elasticity (E_{m}) is 16773 MPa for prisms grouted and parallel to the bed joint (Table 4-6) and the reference load (P_{r}) for B100 and B120 series specimens was 60 kN (70 kN for B120Y1 and B120Y2), the deflections were calculated and are shown in Table 5-2. The deflection profiles for all eight beams are shown in Appendix F, Figures F-1 to F-8. At the spliced locations, the area of steel was doubled, causing double the flexural stiffness in that region. The effect of the spliced area was taken into account using Equation 5-3. The effective moment of inertia taking into account the splice (I_{eff(s)}) is calculated by taking a weighted average of the effective moment of inertia along the length of the beam.

Table 5-2: Experimental vs. Estimated Deflection

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>P_{r}  (kN)</th>
<th>E_{m}  (MPa)</th>
<th>I_{eff}/I_{eff(s)} \times 10^{9} mm^4</th>
<th>Calculated Deflection \Delta (mm)</th>
<th>Experimental Deflection, D (mm)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B100N1</td>
<td>60</td>
<td>16773</td>
<td>5.2</td>
<td>11.3</td>
<td>11.5</td>
<td>+1.8</td>
</tr>
<tr>
<td>B100N2</td>
<td>60</td>
<td>16773</td>
<td>5.2</td>
<td>11.3</td>
<td>11.9</td>
<td>+5.3</td>
</tr>
<tr>
<td>B100Y1</td>
<td>60</td>
<td>16773</td>
<td>5.4</td>
<td>10.9</td>
<td>11.1</td>
<td>+1.8</td>
</tr>
<tr>
<td>B100Y2</td>
<td>60</td>
<td>16773</td>
<td>5.4</td>
<td>10.9</td>
<td>11.5</td>
<td>+5.5</td>
</tr>
<tr>
<td>B120N1</td>
<td>60</td>
<td>16773</td>
<td>10.4</td>
<td>9.3</td>
<td>10.2</td>
<td>+9.7</td>
</tr>
<tr>
<td>B120N2</td>
<td>60</td>
<td>16773</td>
<td>10.4</td>
<td>9.3</td>
<td>8.5</td>
<td>-9.6</td>
</tr>
<tr>
<td>B120Y1</td>
<td>70</td>
<td>16773</td>
<td>10.1</td>
<td>10.6</td>
<td>10.3</td>
<td>-2.8</td>
</tr>
<tr>
<td>B120Y2</td>
<td>70</td>
<td>16773</td>
<td>10.1</td>
<td>10.6</td>
<td>10.7</td>
<td>+0.9</td>
</tr>
</tbody>
</table>
Figure 5-4: Deflection Profile for B100N1 at $P = 60$ kN and $P = 110$ kN

Figure 5-5: Deflection Profile of B100Y1 at $P = 60$ kN and $P = 120$ kN
Figure 5-6: Deflection Profile for B120N1 at $P = 60$ kN and $P = 110$ kN

Figure 5-7: Deflection Profile for B120Y1 at $P = 70$ kN and $P = 130$ kN
5.2.2 Out-of-Plane Movement

The out-of-plane lateral displacement of each beam was measured at mid-span on the north side at 700 mm height from the bottom of each beam (LVDT 6 in Figure 3-23). The out-of-plane displacement varied for each beam specimen as shown in Figures 5-8 and 5-9. A maximum out of plane displacement of 11 mm was measured for B100Y2. The out-of-plane displacement for each beam specimen increased with increasing applied load. Each beam leaned to one side (either north or south) more than the other due to the fact that the test setup was not perfect and due to the fact that the beam heights were large. This (leaning to one side) was also supported by the strain gauge data. Each beam had two No. 20 bars, one on the south side and one on the north side. The north bar had higher strain than the south bar for beam specimens that had an out-of-plane deflection on the north side and vice versa. The cracks on each face of the beam were also influenced by the out-of-plane deflection. The crack widths were observed to be larger on the side face (north or south) with a higher strained bar when compared to the other side (north or south).

![Figure 5-8: Out-of-Plane Displacement for Phase One Beam Specimens](image-url)
5.2.3 Longitudinal Displacement

The longitudinal displacement was measured using an LVDT (LVDT 2 in Figure 3-23) on the west end (roller support) of the beam at a height of 700 mm above the bottom of the beam. The purpose of measuring the longitudinal displacement was to determine the amount of movement of the roller support during the test. The movement of the roller support for B100 beams can be seen in Figure 5-10. Longitudinal displacement was observed to increase with rising load. The maximum movement of the roller support was observed to be 9 mm for B100N1 and B100Y1, 5 mm for B100N2, 6 mm for B100Y2. The longitudinal displacement was not measured for B120 beams because it was determined that roller support movement was minimal and not needed for the purpose of the experiment.
5.2.4 Linear Potentiometer Data

The linear potentiometers were used to determine the location of neutral axis at the reference load ($P_r$ in Table 5-2) and the applied load at yield ($P_y$). Figures 5-11 to 5-14 show the linear potentiometer strain for each type of beam specimen. All the linear potentiometer strain data can be seen in Appendix E (Figures E-1 to E-8). The tension zone depth ($d_t$) for B100 beam specimens was calculated to be 660 mm at 60 kN applied load and 770 mm at $P = 110$ kN for B100N1 and B100N2 and $P = 120$ kN for B100Y1 and B100Y2. The tension zone depth for B120 beam specimens was calculated to be 815 mm at 60 kN applied load and 965 mm at $P = 110$ kN for B120N1 and B120N2 and $P = 130$ kN for B120Y1 and B120Y2. The estimated value of the tension zone depth ($d_t$) at yield was calculated by subtracting the compression zone calculated using Equation 3-1 from the beam height. Based on the comparison, the estimated tension zone depth ($d_t$) were within 10% of the experimental tension zone depth ($d_t$) at yield load ($P_y$).
Table 5-3: Experimental vs. Estimated Tension Zone Depth

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Experimental Tension Zone Depth (mm)</th>
<th>Estimated Tension Zone Depth (mm) at Predicted Yield Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At Reference Load, $P_r$</td>
<td>At Yield Load ($P_y$)</td>
</tr>
<tr>
<td>B100N1</td>
<td>660</td>
<td>770</td>
</tr>
<tr>
<td>B100N2</td>
<td>660</td>
<td>770</td>
</tr>
<tr>
<td>B100Y1</td>
<td>660</td>
<td>770</td>
</tr>
<tr>
<td>B100Y2</td>
<td>660</td>
<td>770</td>
</tr>
<tr>
<td>B120N1</td>
<td>815</td>
<td>965</td>
</tr>
<tr>
<td>B120N2</td>
<td>815</td>
<td>965</td>
</tr>
<tr>
<td>B120Y1</td>
<td>815</td>
<td>965</td>
</tr>
<tr>
<td>B120Y2</td>
<td>815</td>
<td>965</td>
</tr>
</tbody>
</table>

Figure 5-11: Linear Potentiometer Strain for B100N1
Figure 5-12: Linear Potentiometer Strain for B100Y1

Figure 5-13: Linear Potentiometer Strain for B120N1
Cracking in masonry construction is primarily a serviceability issue. Design of a masonry structure for strength always requires safety factors to conservatively avoid loss of life due to collapse. A typical masonry structure may only be subjected to load that is much lower than the actual strength capacity during the life cycle because of the safety factors. The crack width is largely controlled by the steel strain (Braam, 1990), hence, crack width needs to be assessed at steel strains based on serviceability loads. The reference level steel strain which was chosen as 1200 $\mu$e based on the recommendations of crack control parameter in the masonry standard (CSA S304.1, 2004a, Clause 11.2.6.2). The crack widths of each beam specimen were analyzed at 50 to 70 kN of applied load which corresponds to a steel strain of 1200 $\mu$e. A microscopic camera was used to measure the crack width along the south face of the beam at different load increments. Each crack width was labeled with two letters (AB, BC, CD, DE, or EF) and a number. The two letters represent where horizontally along the beam the crack is located (Figure 5-15). The number represents how high from the bottom of the beam the particular crack was
being measured in millimeters. For example, a crack width labeled ‘CD300’ is a crack width that was measured at location CD from Figure 5-15 and 300 millimeters up from the bottom of the beam.

Figure 5-15: Crack Labels for Beam Specimens
5.3.1 B100N1

Beam specimen B100N1 was loaded in 10 kN increments up to 30 kN at which point the load was increased in 20 kN increments. The reference load (1200 \(\mu\)ε steel strain) was estimated at about 60 kN. The first crack was observed at 10 kN, a single fine crack at the mortar joint CD that extended up to 100 mm. At 20 kN of load, the adjacent mortar joint crack AB was seen to form up to 100 mm. At 30 kN of load, the first block crack BC developed up to 200 mm (bottom course). At 50 kN of load, cracks AB and BC extended up to 600 mm (3 courses) while crack CD extended through the block up to 400 mm (2 courses). Crack CD remained at 400 mm of height and crack BC remained at 600 mm. A slight extension of crack AB was observed (700 mm) at 90 kN of load, however at this point, a fully developed crack pattern was observed and cracks were seen to widen with increasing load. The progression of the cracks AB, BC, and CD can be seen in Figures 5-16 to 5-20. The flexural reinforcement was located at 90 mm above the bottom of the beam.

Figure 5-16: B100N1 - 325 \(\mu\)ε Flexural Steel Strain and 30 kN
Figure 5-17: B100N1 – 670 με Flexural Steel Strain and 50 kN

Figure 5-18: B100N1 – 800 με Flexural Steel Strain and 70 kN
Figure 5-19: B100N1 – 955 µε Flexural Steel Strain and 90 kN

Figure 5-20: B100N1 – 1078 µε Flexural Steel Strain and 106 kN
The crack width profiles shown in Figures 5-16 to 5-20 were measured at 30 kN, 50 kN, 70 kN, 90 kN, and 106 kN which corresponded to 325, 670, 800, 955, and 1078 microstrain in the steel, respectively. The steel strain was measured on the south flexural bar which is the bar closest to the face of the beam where the crack widths were measured. The north bar was measured to have higher strain than the south bar as seen from the out-of-plane loading (Figure 5-8). The crack width between each adjacent crack was almost constant up to 90 kN of load. At this point, a variation in crack width between each adjacent crack was observed. The intermediate cracks were seen to get much larger after 90 kN of load where the largest intermediate crack was observed at 200 mm to 300 mm. The maximum crack width for each crack varied significantly. In Figure 5-19, the maximum crack width for crack AB is located at 300 mm where as the maximum crack width for cracks BC and CD are at the extreme tension fibre. Furthermore, a large variation of crack width is seen between adjacent cracks.

5.3.2 B100N2

Beam specimen B100N2 was loaded in 10 kN increments up to 30 kN at which point the load was increased in 20 kN increments. The first cracks were seen at 10 kN, two single fine cracks at the mortar joints CD and AB that extended up to 100 mm. At 20 kN, cracks AB and CD extended up to 200 mm (1 course). At 30 kN, crack CD was seen to extend up to 500 mm (2.5 courses) and crack AB was seen to extend up to only 300 mm (1.5 courses). At 50 kN, the first block crack BC formed and extended up to 600 mm. The service load (1200 με steel strain) for this beam is 60 kN. Cracks AB and CD remained at the same height. At 70 kN, crack BC extended up to 700 mm, cracks AB and CD did not extend any further. At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The progression of the cracks AB, BC, and CD can be seen in Figures 5-21 to 5-25. The flexural reinforcement was located at 90 mm above the bottom of the beam.
Figure 5-21: B100N2 – 111 µε Flexural Steel Strain and 20 kN

Figure 5-22: B100N2 – 716 µε Flexural Steel Strain and 30 kN
Figure 5-23: B100N2 – 994 µε Flexural Steel Strain and 50 kN

Figure 5-24: B100N2 – 1447 µε Flexural Steel Strain and 70 kN
The crack width profiles shown in Figures 5-21 to 5-25 were measured at 20 kN, 30 kN, 50 kN, 70 kN, and 90 kN which corresponded to 111, 716, 994, 1447, and 1874 microstrain in the steel, respectively. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The south bar was measured to have higher strain then the north bar as seen from the out-of-plane loading (Figure 5-8). The crack width between each adjacent crack varied significantly. Crack BC (through the block at bottom course) was observed to be the dominant crack out of the three measured and was seen to have widest intermediate cracks (Figures 5-23 to 5-25). The extreme tension zone crack widths were seen to be equal. However, a large variation for intermediate crack widths was observed between each adjacent crack. The intermediate cracks were seen to get much larger after 70 kN of load where the largest intermediate crack was seen at 200 mm to 300 mm height. In Figure 5-24, the maximum crack width for each adjacent crack was found at the extreme tension fibre. However, width of crack BC was seen to decrease from 0.5 mm at the extreme tension fibre to 0.23
mm at the flexural reinforcement level and increase to a maximum up to 0.32 mm at 200 mm. The widths of adjacent cracks were seen to decrease with the beam height.

5.3.3 B100N Series

Beam specimens B100N1 and B100N2 were both observed to have the same cracking pattern. B100N beams were observed to initiate cracking at 10 kN of load and began at the mortar joints (AB, CD, and EF in Figure 5-15a). The first block crack was not seen until the mortar joint cracks developed and until 30 kN to 50 kN, suggesting that mortar – block joint was the weakest location. A variation in crack height between each adjacent crack was observed to be a similarity between the two beams; however a large difference in steel strain and load was observed between the two beams. At 70 kN, B100N1 had a steel strain of 800 µε whereas B100N2 had a steel strain of 1447 µε. This difference in steel strain at similar loads between the two beam specimens is explained by the out-of-plane displacement. B100N1 had an out-of-plane displacement of 5 mm to the north, causing an increased strain on the north side. B100N2 had an out-of-plane displacement of 1 mm to the south causing an increased strain on the south side. The maximum crack widths at the extreme tension fibre at service load were 0.47 mm and 0.5 mm and the maximum crack widths at the intermediate level were 0.31 mm and 0.32 mm for B100N1 and B100N2, respectively. The cracks were measured on the south face of the beam which explains why the cracks widths were much lower in B100N1 as compared to B100N2. The crack widths were compared using the steel strain of the south bar. The crack widths between the two beams are similar when compared using steel strain from the south bar.

5.3.4 B100Y1

Beam specimen B100Y1 was loaded in 10 kN increments up to 30 kN at which point the load was increased to 20 kN increments. The first crack was seen at 20 kN, a single crack formed at the mortar joint CD that extended up to 300 mm (1.5 courses). At 30 kN, crack AB formed up to 300 mm (1.5 courses) and crack CD extended up to 400 mm (2 course) (Figure 5-27). At 50 kN, the first block crack BC was observed to form up to 400 mm (2 courses) (Figure 5-28). Cracks AB and CD was seen to extend up to 400 mm and 600
mm, respectively. At 50 kN, the block crack BC extended up to 600 mm. Cracks AB and CD remained at the same height. At 70 kN, cracks AB and BC extended up to 600 mm (3 courses) BC remained constant. At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The service load for this beam was assumed to be 60 kN (1200 µε steel strain). The progression of the cracks AB, BC, and CD can be seen in Figures 5-26 to 5-31. The flexural and intermediate reinforcement was located at 90 mm and 490 mm above the bottom of the beam.

Figure 5-26: B100Y1 – 134 µε Flexural Steel Strain (67 µε Intermediate Steel Strain) and 20 kN
Figure 5-27: B100Y1 - 521 µε Flexural Steel Strain (331 µε Intermediate Steel Strain) and 30 kN

Figure 5-28: B100Y1 - 968 µε Flexural Steel Strain (551 µε Intermediate Steel Strain) and 50 kN
Figure 5-29: B100Y1 – 1260 µε Flexural Steel Strain (689 µε Intermediate Steel Strain) and 70 kN

Figure 5-30: B100Y1 – 1606 µε Flexural Steel Strain (840 µε Intermediate Steel Strain) and 90 kN
Figure 5-31: B100Y1 – 1997 µε Flexural Steel Strain (1044 µε Intermediate Steel Strain) and 111 kN

The crack width profiles shown in Figures 5-26 to 5-31 were measured at 20 kN, 30 kN, 50 kN, 70 kN, 90 kN, and 111 kN which corresponded to 134, 521, 968, 1260, 1606, and 1997 microstrain in the steel, respectively. The service load is observed to be 70 kN. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The south bar was measured to have a slightly higher strain then the north bar as seen from the out-of-plane loading (Figure 5-8). The crack width between each adjacent crack varied significantly in the extreme tension fibre, however were less variable at the intermediate bar level. Cracks AB, BC, and CD were seen to extend up to the same height (600 mm) and decreased at intermediate level of the beam, suggesting that the intermediate bar reduced the intermediate crack width. In Figure 5-30, the maximum intermediate crack width (0.25 mm) was seen to be at 300 mm, however the widest crack widths were located at the extreme tension zone along the mortar joints (AB and CD at 0.36 mm and 0.38 mm, respectively).
5.3.5 B100 Beams

In phase one, two beams without intermediate reinforcement (Type 1) and two beams with intermediate reinforcement (Type 2) were tested. A cracking pattern difference was observed between these two types of beams. The crack spacing for B100N1 and B100N2 at the bottom course was measured at 200 mm, and then increased to 400 mm in between the neutral axis and the main reinforcing bars (Figures 5-32(a) and (b)). However, the crack spacing for B100Y1 remained constant at 200 mm along entire height of the beam. The crack width data for B100Y2 is not included due to the cracking damage sustained during transportation although the crack spacing did remain constant. The different crack patterns of B100N1, B100N2 and B100Y1 can be seen in Figure 5-32.

![Cracking Pattern Comparison](image)

Figure 5-32: Fully Developed Crack Pattern Comparison of B100 Beams

The difference in cracking patterns between the two types of beam specimens can be explained by the presence of intermediate reinforcement. There was no intermediate bar in B100N1 and B100N2. Therefore, between the height of 300 mm and 700 mm, the
distance between the masonry surface and the closest reinforcing bar is large. Hence, the crack spacing is large. Beam B100Y1 had an intermediate bar, therefore, decreasing the distance between the masonry surface and the closest reinforcing bar. As discussed in chapter 2.8, previous studies on RC beams showed that crack spacing increases as concrete cover increases (Broms, 1965a).

Figures 5-33 to 5-35 compare the widest crack from beams B100N2 and B100Y1 and illustrate the effect of intermediate reinforcement. The widest crack measured in the beam will be called the critical crack in the subsequent discussion. The critical crack of B100N2 was crack BC and the critical crack of B100Y1 was crack CD (Figure 5-33). In each of the Figures 5-33 to 5-35, the crack widths at the extreme tension fibre were similar in value, however a reduction in intermediate crack width is seen which suggests the intermediate reinforcement effectively reduces intermediate crack widths. The addition of intermediate reinforcement also changed the location of the critical crack. Beam B100N2 had the critical crack initiate through the block whereas beam B100Y1 was seen to have the critical crack initiate at the mortar joint. Beam B100N2 had the largest intermediate crack width between 200 mm and 400 mm which was at the head joint of the second course for crack BC. The results suggest that flexural cracking was controlled by the mortar joints where it is shown to be the weakest.
Figure 5-33: B100N2 (Crack BC) Versus B100Y1 (Crack CD) at 1000 με

Figure 5-34: B100N2 (Crack BC) Versus B100Y1 (Crack CD) at 1200 με - 1400 με
5.3.6 B120N1

Beam specimen B120N1 was loaded in 10 kN increments up until yield load of 110 kN was reached. The first crack was seen at 20 kN, crack BC at the mortar joint formed and extended up to 100 mm. At 30 kN, crack DE initiated and extended up to 100 mm. At 40 kN, The first block crack CD formed and extended up to 600 mm. Cracks BC and DE remained at 100 mm. At 50 kN, cracks BC and DE extended up to 200 mm and crack CD extended up to 800 mm. The reference load for this beam was at 60 kN (1200 με steel strain). At 70 kN, crack BC extended up to 400 mm. Crack CD remained at 800 mm and crack DE remained at 200 mm. At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The progression of the cracks BC, CD, and DE can be seen in Figures 5-36 to 5-41. The flexural reinforcement was located at 90 mm above the bottom of the beam.
Figure 5-36: B120N1 – 725 µε Flexural Steel Strain and 30 kN

Figure 5-37: B120N1 – 987 µε Flexural Steel Strain and 40 kN
Figure 5-38: B120N1 - 1208 με Flexural Steel Strain and 50 kN

Figure 5-39: B120N1 - 1383 με Flexural Steel Strain and 60 kN
Figure 5-40: B120N1 – 1555 με Flexural Steel Strain and 70 kN

Figure 5-41: B120N1 – 1716 με Flexural Steel Strain and 80 kN
The crack width profiles shown in Figures 5-36 to 5-41 were measured at 30 kN, 40 kN, 50 kN, 60 kN, 70 kN, and 80 kN which corresponded to 725, 988, 1208, 1383, 1555 and 1716 microstrain in the steel, respectively. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The south bar was measured to have higher strain than the north bar (10 cm west) due to the fact that the strain gauge on the south bar was measured at mid-span where the critical crack was located. Cracks BC and DE were the first cracks to form, and crack CD formed afterwards and extended higher than crack BC and DE. The crack width between each adjacent crack was observed to vary significantly. Crack CD was observed to be the longest crack out of the three measured and was seen to have widest intermediate cracks, however crack BC was seen to have the widest extreme tension fibre crack. A high variability in crack height and width between adjacent cracks was observed in Figures 5-38 to 5-41. Crack CD was seen to have the largest intermediate crack at 300 mm which falls in the mortar joint of the second course. Crack DE only extended up 200 mm (1 course) which caused crack CD to widen at 300 mm. The extreme tension fibre crack width for CD was observed to be smaller than the intermediate crack which was caused by crack BC. Crack BC was measured to have the highest tension zone fibre crack width holding the majority of the tensile strain between the three cracks.

5.3.7 B120N2

Beam specimen B120N2 was loaded in 10 kN increments up to 110 kN. The first cracks were seen at 20 kN, crack BC and DE at the mortar joint formed and extended up to 200 mm. At 40 kN, crack BC extended up to 500 mm and crack DE extended up to 700 mm. At 50 kN, crack BC extended up to 600 mm and crack DE extended up to 800 mm. The service load for this beam was at 60 kN (1200 µε steel strain). At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The progression of the cracks BC and DE can be seen in Figures 5-42 to 5-47. The flexural reinforcement was located at 90 mm above the bottom of the beam.
Figure 5-42: B120N2 – 583 µε Flexural Steel Strain and 50 kN

Figure 5-43: B120N2 – 808 µε Flexural Steel Strain and 60 kN
Figure 5-44: B120N2 – 1016 με Flexural Steel Strain and 70 kN

Figure 5-45: B120N2 – 1197 με Flexural Steel Strain and 80 kN
Figure 5-46: B120N2 – 1454 με Flexural Steel Strain and 90 kN

Figure 5-47: B120N2 – 1659 με Flexural Steel Strain and 100 kN
The crack width profiles shown in Figures 5-42 to 5-47 were measured at 50 kN, 60 kN, 70 kN, 80 kN, 90 kN, and 100 kN which corresponded to 583, 808, 1016, 1197, 1454 and 1659 microstrain in the steel, respectively. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The south bar was measured to have lower strain than the north bar (10 cm west) as shown by the out-of-plane displacement. The crack width between each adjacent crack was observed to vary significantly. Cracks BC and DE were the only cracks to form, no cracks developed through the lintel unit. Variability in crack height and width between adjacent cracks was observed in Figures 5-42 to 5-47. Crack DE was seen to be the critical crack. Crack BC and DE developed the widest intermediate crack at 200 mm above the bottom of the beam and were observed to have similar patterns as seen in Figures 5-42 to 5-47.

5.3.8 B120N Beams

B120N beams were seen to have different cracking patterns. B120N1 beam developed a block crack CD whereas B120N2 developed cracks at the mortar joints (BC and DE). The difference in cracking pattern suggests that cracking patterns for B120N beams are unpredictable. B120N1 and B120N2 beams both had high variability in crack width and unacceptably high crack widths ($w > 0.4 \text{ mm}$) suggesting that intermediate reinforcement may be required to satisfy serviceability requirements.

5.3.9 B120Y1

Beam specimen B120Y1 was loaded in 10 kN increments up to 140 kN. The first cracks were seen at 30 kN, crack BC and DE at the mortar joint formed and extended up to 200 mm. At 40 kN, crack DE extended up to 400 mm and crack DE remained at 200 mm. At 50 kN, crack BC and DE extended up to 600 mm. The service load for this beam was 70 kN (1200 $\mu \varepsilon$ steel strain). At 90 kN, crack BC and DE extended up to 800 mm. At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The progression of the cracks BC and DE can be seen in Figures 5-48 to
5-54. The flexural and intermediate reinforcement was located at 90 mm and 490 mm above the bottom of the beam.

Figure 5-48: B120Y1 – 398 με Flexural Steel Strain (108 Intermediate Steel Strain) and 40 kN
Figure 5-49: B120Y1 – 783 $\mu$ε Flexural Steel Strain (328 $\mu$ε Intermediate Steel Strain) and 60 kN

Figure 5-50: B120Y1 – 978 $\mu$ε Flexural Steel Strain (558 $\mu$ε Intermediate Steel Strain) and 70 kN
Figure 5-51: B120Y1 – 1156 με Flexural Steel Strain (820 με Intermediate Steel Strain) and 80 kN

Figure 5-52: B120Y1 – 1332 με Flexural Steel Strain (1168 με Intermediate Steel Strain) and 90 kN
Figure 5-53: B120Y1 - 1502 με Flexural Steel Strain (1318 με Intermediate Steel Strain) and 100 kN

Figure 5-54: B120Y1 - 1871 με Flexural Steel Strain (1589 με Intermediate Steel Strain) and 110 kN
The crack width profiles shown in Figures 5-48 to 5-54 were measured at 40 kN, 60 kN, 70 kN, 80 kN, 90 kN, 100 kN, and 110 kN which corresponded to 398, 783, 978, 1156, 1332, 1502, and 1871 microstrain in the steel, respectively. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The steel strain for the north and south bar were similar in value. Cracks BC and DE were the only cracks to form, no block cracks developed. Cracks BC and DE were observed to be very similar in crack height and width. An evenly distributed crack pattern was seen in Figures 5-48 to 5-54. Crack BC and DE developed the widest intermediate crack at 200 mm above the bottom of the beam. The maximum intermediate crack width was observed to be wider than the extreme tension fibre crack in Figures 5-53 and 5-54 suggesting that a second intermediate bar at 200 mm above the flexural reinforcement is needed.

5.3.10 B120Y2

Beam specimen B120Y1 was loaded in 10 kN increments up to 140 kN of load. The first cracks were seen at 30 kN, crack BC and DE at the mortar joint formed and extended up to 200 mm. At 40 kN, crack BC and DE extended up to 400 mm. At 60 kN, crack BC and DE extended up to 600 mm. The service load for this beam is at 70 kN (1200 με steel strain). At 80 kN, crack BC and DE extended up to 700 mm. At 100 kN, crack BC and DE extended up to 800 mm. At this point, a fully developed crack pattern was seen and cracks were seen to widen with increasing load. The progression of the cracks BC and DE can be seen in Figures 5-55 to 5-61. The flexural and intermediate reinforcement was located at 90 mm and 490 mm above the bottom of the beam.
Figure 5-55: B120Y2 – 585 $\mu$ε Flexural Steel Strain (131 $\mu$ε Intermediate Steel Strain) and 40 kN

Figure 5-56: B120Y2 – 784 $\mu$ε Flexural Steel Strain (373 $\mu$ε Intermediate Steel Strain) and 50 kN
Figure 5-57: B120Y2 – 1006 µε Flexural Steel Strain (545 µε Intermediate Steel Strain) and 60 kN

Figure 5-58: B120Y2 – 1213 µε Flexural Steel Strain (727 µε Intermediate Steel Strain) and 70 kN
Figure 5-59: B120Y2 – 1408 µε Flexural Steel Strain (800 µε Intermediate Steel Strain) and 80 kN

Figure 5-60: B120Y2 – 1576 µε Flexural Steel Strain (966 µε Intermediate Steel Strain) and 90 kN
The crack width profiles shown in Figures 5-55 to 5-61 were measured at 40 kN, 50 kN, 60 kN, 70 kN, 80 kN, 90 kN, and 100 kN which corresponded to 585, 784, 1006, 1213, 1408, 1576, and 1756 microstrain in the steel, respectively. The steel strain was measured on the south bar which is the bar closest to where the crack widths were measured. The south bar steel strain was measured higher than the north bar steel strain due to the out-of-plane movement (Figure 5-8). Cracks BC and DE were the only cracks to form, no block cracks developed. Cracks BC and DE were observed to be very similar in crack height and width. An evenly distributed crack pattern was seen in Figures 5-55 to 5-61. Crack BC and DE developed the widest intermediate crack at 300 mm in Figure 5-59 and 200 mm in Figures 5-58, 5-60 and 5-61. The maximum intermediate crack width was observed to be larger than the extreme tension fibre crack in Figure 5-60, however the extreme tension fibre crack was observed to be the largest for Figures 5-55 to 5-59 and 5-61.

Figure 5-61: B120Y2 – 1756 με Flexural Steel Strain (1008 με Intermediate Steel Strain) and 100 kN
5.3.11 B120Y Beams

B120Y1 and B120Y2 beams were both observed to have the similar crack patterns. B120Y beams developed first cracks in the mortar joints (BC and DE) at 30 kN of load. A constant crack spacing of 400 mm was seen in B120Y beams. Each crack in the B120Y beams had similar progression and extended up to 800 mm. After crack BC and DE reached 800 mm in length, no new cracks or extension of existing cracks were observed. Crack BC and DE were the only existing flexural cracks, no block cracks developed. The crack width profile for crack BC and DE were observed to be within 0.1 mm. The addition of intermediate reinforcement was observed to give uniformity to the cracking pattern.

5.3.12 B120 Beams

In phase two, two beams without intermediate reinforcement and two beams with intermediate reinforcement were constructed. Similarities and differences were observed between cracking behaviour in both types of beams. B120N1 was the only beam from the B120 beams to develop a block crack. The mortar joint cracks (BC and DE) developed first, then the block crack CD developed and instantly became the dominating crack. B120N2 had a different cracking pattern than B120N1 although they were the same type of beam. The difference in cracking pattern between B120N1 and B120N2 suggests that the cracking pattern for beams without intermediate reinforcement was highly variable. The cracking pattern for B120N2 was observed to be similar to B120Y beams, however larger crack widths. B120N beams were seen to have high variability in crack width and length between adjacent cracks, whereas, B120Y beams were seen to have constant crack widths and lengths between each adjacent crack. The addition of intermediate reinforcement was observed to add predictability to the cracking pattern. B120Y beams were seemed to have a uniformly distributed crack pattern whereas B120N1 beam had high variability in the crack pattern. The cracking patterns for B120 beams can be seen in Figure 5-62. Crack DE was observed to be the critical crack for all B120 beam specimens except for B120N1 which was crack CD. B120N1 beam was observed to have a very
different cracking pattern compared to the other B120 beams (Figure 5-62). B120N1 crack widths were therefore not comparable with other B120 beams.

In addition to a difference in cracking patterns, a difference in crack width was also observed. Figures 5-63 to 5-66 compares crack DE in B120N2 and B120Y2 at various strain levels in the main reinforcing bars. Based on the comparison, it can be seen that the intermediate bar significantly reduced crack widths throughout the beam at 1000 to 1400 με in the flexural steel. However, the difference in crack width between B120N2 and B120Y2 decreases for increasing strain levels suggesting that the intermediate bar is ineffective at high strain levels (Figures 5-63 to 5-66). The intermediate reinforcing bar reduced the crack width by 0.1 mm to 0.3 mm throughout the entire height of the beam at
1000 to 1200 µε as shown in Figures 5-63 and 5-64. The intermediate reinforcing bar also evenly distributed the crack width between each crack.

Figure 5-63: B120N2 (Crack DE) Versus B120Y2 (Crack DE) – 1000 µε
Figure 5-64: B120N2 (Crack DE) Versus B120Y2 (Crack DE) – 1200 με

Figure 5-65: B120N2 (Crack DE) Versus B120Y2 (Crack DE) – 1400 με
5.3.13 Beam Specimen Cracking Analysis

A total of eight beams were tested, four with intermediate reinforcement and four without intermediate reinforcement. The first cracks that developed in each beam started at the head joints of the bottom course. Block cracks between the mortar joints developed after the initial mortar joint cracks for B100 beams and B120N1. B100N1, B100N2, and B120N1 developed block crack that enlarged to the critical crack for the beam. The critical crack for B100Y1 formed at the head joint where first cracks were formed. In beam specimens without intermediate reinforcement, short cracks were seen to develop at the head joints followed by long block cracks except for beam B120N2 where no block cracks developed.

In addition to crack development, crack spacing was also seen to be different when comparing B100 and B120 beams. In the region of the flexural reinforcement (0 mm to 300 mm crack height), B100 beams were seen to have 200 mm crack spacing, whereas B120 beams were seen to have 400 mm crack spacing. A possible explanation for this
behaviour could be because mid-span for B100 beams was located at a head joint, whereas the mid-span for B120 beams was located in the block. Beams B100N and B120N were observed to have increased crack spacing at the intermediate level. This observation is explained by the large cover distance at the intermediate level. It was discussed in the literature review that crack spacing is directly proportional to cover distance (Broms, 1965a). The addition of intermediate reinforcement reduced the cover distance which decreased the crack spacing at the intermediate bar level.

The maximum crack width location for all beam specimens was located at the extreme tension zone at 1200 με in the flexural reinforcement. An explanation for this observation could be because each beam was highly under-reinforced. The flexural reinforcement ratio was calculated to be 0.35% and 0.29% for B100 and B120 beams, respectively. The low flexural reinforcement ratio could explain the critical extreme tension fibre cracks and hence, more flexural steel would be needed to limit this crack at the bottom fibre of the beam. Another possible explanation is the large cover distance between the extreme tension fibre and the flexural steel. The beam specimens were constructed having an 80 mm cover distance whereas typical cover in RC beams for flexural reinforcement is between 30 mm to 50 mm. It was discussed in the literature review that crack width and spacing are proportional to cover distance (Broms, 1965a). In all tested beam specimens in this research, a maximum intermediate crack width existed between the flexural reinforcement and neutral axis. This occurred at a height of between 200 mm to 300 mm above the bottom face of the beam for every beam specimen. This observation suggests that a more appropriate location for the intermediate bar is 200 mm above the flexural reinforcement.

5.4 Cracking Patterns in Masonry Beams

Similarities were observed between the cracking pattern of large reinforced masonry beams (h > 600 mm) and large reinforced concrete beams (h > 750 mm). In the literature review, a cracking model was discussed and was capable of calculating crack width at any location in the concrete beam (Frosh, 2002). The cracking model was used in this
study to compare the model crack widths with experimental data. The comparison can be seen in Figures 5-67 to 5-70. In Figures 5-67 to 5-70, the crack widths for each crack were average at the different heights. As an example, for B100N1 at the extreme tension fibre (Figure 5-19), cracks AB, BC, and CD were 0.23 mm, 0.31 mm, and 0.30 mm. The average of those three crack widths was 0.28 mm, which is shown in Figure 5-67 for B100N1 at the extreme tension fibre. The minimum and maximum crack widths were calculated using Equation 2-26, assuming steel strain of 1000 $\mu$e for Figure 5-67 and 1200 $\mu$e for Figures 5-68 to 5-70. The tension zone depth for the beam specimens is taken from Table 5-3 for service load conditions.

![Figure 5-67: B100N Beams Versus Crack Width Model – 1000 $\mu$e Steel Strain](image-url)
Figure 5-68: B100Y1 Versus Crack Width Model – 1200 με Steel Strain

Figure 5-69: B120N Beams Versus Crack Width Model – 1200 με Steel Strain
Based on the comparison between actual crack width data obtained from testing and crack width model proposed by Frosh (2002), similarities and differences can be seen. The crack width measured at the extreme tension fibre for the majority of cases was larger than the model predicted crack width. A possible explanation for this observation is that the crack spacing is larger than expected due to the cross-section discontinuity at the head joints. The crack spacing can be calculated using Equation 2-22 (Broms, 1965a).

The following table (Table 5-4) compares the actual crack spacing for B100N2 and the model crack spacing using Equation 2-22 assuming crack spacing factor ($\psi_s$) is 1.5 which is the average value between one and two for minimum and maximum crack spacing, respectively. The test crack spacing was calculated for the constant moment region (1000 mm) as 1000 mm divided by the number of cracks formed in the constant moment region.
Table 5-4: B100N2 – Actual Crack Spacing Versus Model Crack Spacing

<table>
<thead>
<tr>
<th>Crack Height (mm)</th>
<th>Actual Crack Spacing (mm)</th>
<th>Model Crack Spacing (mm)</th>
<th>Remarks</th>
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<tr>
<td>600</td>
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<td>Model Over Predicts Crack Spacing</td>
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Based on the comparison of crack spacing, the Frosh model is seen to under predict the crack spacing at the flexural reinforcement level and below and over predict the crack width above 200 mm, therefore, it is concluded that the Frosh (2002) model needs to be corrected to accurately predict crack width for masonry beams. The experimental crack spacing for each beam is shown for 1000 \( \mu \varepsilon \) to 1200 \( \mu \varepsilon \) steel strain in Table 5-5.

Table 5-5: Experimental Crack Spacing

<table>
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<tr>
<th>Height (mm)</th>
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<th>B100N2 (mm)</th>
<th>B100Y1 (mm)</th>
<th>B120N1 (mm)</th>
<th>B120N2 (mm)</th>
<th>B120Y1 (mm)</th>
<th>B120Y2 (mm)</th>
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The experimental crack spacing’s are different than the crack spacing’s calculated using Frosh (2002) model as shown by Equation 2-22. Therefore, it is found that crack spacing may not be directly proportional to the cover for masonry which was seen for concrete; however a relationship may exist between masonry cover and crack spacing. The crack
spacing seems to be controlled by the head joint spacing. Using the experimental crack spacing in Equation 2-23, the crack width in masonry can be more accurately predicted. A comparison of the predicted crack width using Equation 2-23 and average experimental crack width is shown in Figures 5-71 to 5-74 for B100N2, B100Y1, B120N2, and B120Y1, respectively. Beam B100N2 was shown for 1000 µε steel strain (Figure 5-71) and the rest are shown 1200 µε steel strain (Figures 5-72 to 5-74).

![Figure 5-71: B100N2 Predicted Versus Experimental Crack Width](image)

Figure 5-71: B100N2 Predicted Versus Experimental Crack Width
Figure 5-72: B100Y1 Predicted Versus Experimental Crack Width

Figure 5-73: B120N2 Predicted Versus Experimental Crack Width
A similar trend between predicted and experimental crack widths is seen in Figures 5-71 to 5-74. The crack width can be calculated as the product of flexural reinforcement strain and crack spacing which was discussed in chapter 2.6 in the literature review and is included in the crack width model. The crack spacing was observed to be different from the relationship proposed in Equation 2-11 (Broms, 1965) due to the head joint in masonry construction. Based on this observation, it can be concluded that the crack spacing is largely affected by the head joint spacing.

Large masonry beams (h > 600 mm) without intermediate reinforcement were observed to have short and long crack heights. In large concrete beams (h > 750 mm) without skin reinforcement, many cracks developed near the extreme tension zone. These cracks would merge into fewer wide cracks between the flexural reinforcement and the neutral axis (Frantz & Breen, 1980). In large masonry beams (h > 600 mm), flexural cracking developed either at the head joint or through the centre of the block where the second course head joint was located. The mortar-block bond represented a weak point; hence,
the development of flexural cracking began at this location. The cracking behaviour of a masonry beam was largely influenced by the mortar joints. In this research study, a running bond pattern was used to lay the blocks; therefore, the crack spacing was 200 mm at the flexural reinforcement. B100 beams were observed to have a 200 mm cracking spacing at the flexural reinforcement, whereas B120 beams were observed to have 400 mm crack spacing at the flexural reinforcement. The addition of intermediate reinforcement kept the crack spacing constant throughout the beam height. B120N2 and B120Y2 beams were observed to have very close crack spacing (400 mm) throughout the beam; however, the crack width was reduced in B120Y2 by up to 50% in some cases. The intermediate bar was effective at changing the cracking behaviour of large reinforced masonry beams in this research project.

5.5 Clip Gauge Data

5.5.1 B100 Beams

Two clip gauges were used for all four beam specimens in phase one. The location of the clip gauges were CD100 and CD500 for all beam specimens (Table 3-8). The progression of crack width with flexural steel strain for B100N1, B100N2 and B100Y1 are shown in Figures 5-75 to 5-77. The crack width data for B100Y2 was not shown due to damage incurred during transportation. In all cases the crack width at the intermediate bar level is lower than the crack width at the flexural reinforcement level. The clip gauge data suggests that the intermediate bar is not effective, however only two points along the centre crack were monitored. B100N1 and B100N2 had the largest crack at location BC instead of CD according to the microscope crack width data, hence the reason for the narrower crack widths when compared to B100Y1. The clip gauges had to be installed before the test was conducted and the largest crack width was not possible to predict. The clip gauges were installed at the beam centre line under the assumption that the largest crack width would occur at the beam centre line. Unfortunately, the placement of the clip gauge did not capture the critical crack width.
Figure 5-75: Clip Gauge Data for B100N1

Figure 5-76: Clip Gauge Data from B100N2
Five clip gauges were used to monitor the crack width along the side face of the four beam specimens in phase two. The location of the clip gauges were CD200, CD300, CD400, BC300, and BC400 for B120N2 and B120Y1 and DE100, DE200, DE300, DE400, and DE500 for B120N1 and B120Y2 (Table 3-8). Five clip gauges were used instead of two because more crack width data was needed to accurately come to a conclusion. Based on the microscopic data from phase one beam tests, the largest crack width between the neutral axis and the flexural reinforcing bars occurred at 200 mm to 300 mm above the base of the beam. Hence, the reason for the clip gauges being installed at 200 mm to 400 mm above the base of the beam. B120N2 and B120Y1 were tested first and the crack spacing was shown to be 400 mm and at the head joints. The location of the clip gauges for the final two beam specimens (B120N1 and B120Y2) were changed to one mortar joint to measure the dominant crack.

Figure 5-77: Clip Gauge Data for B100Y1

**5.5.2 B120 Beams**

Five clip gauges were used to monitor the crack width along the side face of the four beam specimens in phase two. The location of the clip gauges were CD200, CD300, CD400, BC300, and BC400 for B120N2 and B120Y1 and DE100, DE200, DE300, DE400, and DE500 for B120N1 and B120Y2 (Table 3-8). Five clip gauges were used instead of two because more crack width data was needed to accurately come to a conclusion. Based on the microscopic data from phase one beam tests, the largest crack width between the neutral axis and the flexural reinforcing bars occurred at 200 mm to 300 mm above the base of the beam. Hence, the reason for the clip gauges being installed at 200 mm to 400 mm above the base of the beam. B120N2 and B120Y1 were tested first and the crack spacing was shown to be 400 mm and at the head joints. The location of the clip gauges for the final two beam specimens (B120N1 and B120Y2) were changed to one mortar joint to measure the dominant crack.
Figure 5-78: Clip Gauge Crack Width at 1000 µε for B120N2

Figure 5-79: Clip Gauge Crack Width at 1000 µε for B120Y1
Figures 5-78 and 5-79 show the crack width from the clip gauges for B120N2 and B120Y1, respectively. Two cracks developed in these two beams BC and DE, no crack developed at CD, hence the reason the crack width is zero in Figures 5-78 and 5-79. Figure 5-80 shows the clip gauge crack width for crack BC on B120N1. Unfortunately the largest crack developed at the centre span and crack BC only developed up to 400 mm, however the crack width increased almost linearly down the face of the beam. Figure 5-81 shows the clip gauge crack width for crack BC on B120Y2. Fortunately the clip gauges were placed in a good location and the major crack was monitored. B120Y2 had two cracks, one at BC and the other at DE. The crack width results indicate that crack width increases almost linearly down the face of the beam. The largest crack width is at the reinforcement level suggesting that the intermediate bar is effective, however upon looking at the crack width at a higher strain level in Figure 5-82, it can be seen that the largest crack width develops at 200 mm by a small margin.
Figure 5-81: Clip Gauge Crack Width at 1000 µε for B120Y2

Figure 5-82: Clip Gauge Crack Width at 1750 µε for B120Y2
6 Conclusion and Recommendations

6.1 Summary

In reinforced masonry beams, flexural cracking will develop in the tension zone. Flexural cracks can cause serviceability issues such as increased deflections from reduced stiffness and aesthetic issues which would be undesirable for public appearance. In addition, flexural cracks can also cause durability issues such as steel corrosion through water intrusion and masonry degradation. Steel reinforcement is used to control flexural cracks. Past research in concrete shows that flexural crack widths are proportional to two main parameters: steel strain and distance away from steel (Broms, 1965a). The yield strain of steel is estimated to be 2000 \( \mu \varepsilon \) (CSA S304.1, 2004a). Masonry beams are designed using material reduction factors to limit the risk of failure. Therefore, masonry beam in the field are typically subjected to loads that are much lower than those accounted for in the design. Steel reinforcement in a masonry beam need not be taken as more than 60% of the yield strain at service conditions (CSA S304.1, 2004a). Hence, flexural cracking is typically analyzed at service steel strain of 1200 \( \mu \varepsilon \).

Masonry beams are subjected to mainly flexural and shear stress. Under the assumption of linear strain distribution, strain is zero at the neutral axis and increases linearly to a maximum value at the extreme tension fibre. Flexural crack widths are expected to be maximum at the extreme tension fibre, however flexural steel reinforcement near this region limits crack widths. Steel reinforcement becomes less effective at limiting the width of cracks at large cover distances (Broms, 1965a). In masonry beams, steel reinforcement is placed in the bottom course to maximize moment resistance through a larger moment arm between compression and tensile resultants. In large masonry beams (h > 600 mm), the steel at the bottom course is less effective at controlling crack widths near mid-height of the beam. These intermediate cracks can potentially become larger than the extreme tension zone cracks (CSA S304.1, 2004a). The introduction of intermediate reinforcement is used to control side face crack widths. This provides a
better distribution of reinforcement in the tension zone limiting the distance of any masonry surface point away from the steel, hence reducing potential crack widths.

The placement of intermediate reinforcement is covered in CSA S304.1-04a clause 11.2.6.3. CSA S304.1-04a clause 11.2.6.3 states the following:

- Where the beam height exceeds 600 mm, longitudinal reinforcement shall be uniformly distributed over the height of the beam. A single No. 15 bar for beams up to 240 mm wide, and a No. 15 bar on each side for wider beams, shall be provided at 400 mm vertical spacing.

Intermediate reinforcement has many similarities to skin reinforcement covered in CSA A23.3-04b clause 10.6.2. The use of intermediate reinforcement is intended to reduce crack widths between the main reinforcement and the neutral axis. As stated above for masonry beam widths up to 240 mm wide, one No. 15 bar is used to control intermediate crack widths. The horizontal masonry cover can be up to 120 mm, where typical concrete cover which is used for skin reinforcement is 30 mm to 40 mm. The purpose of this research is to investigate the effectiveness of intermediate reinforcement requirements of CSA S304.1-04a.

In this research project, a total of eight beams were tested, four with intermediate reinforcement and the other four without intermediate reinforcement. Each beam had the minimum steel reinforcement required to maximize the tension zone depth. Three differences in the cracking behaviour were observed between beams with versus without intermediate reinforcement. The first difference is that all beams had a very similar load deformation curve, up to yielding of the steel at which point BY beams had a 20% difference in strength when compared to the BN beams due to the addition of intermediate reinforcement (Figures 5-1 and 5-2). The second difference was the change in cracking patterns. BN beams were observed to have highly variable crack height and width between each adjacent crack at first yield of the flexural reinforcement. BY beams were observed to have an evenly distributed crack pattern with constant crack height,
width and spacing (Figures 5-32 and 5-62). The third difference was the reduction in crack width. B100N beams had up to twice as wide intermediate cracks compared to B100Y beams, however the crack width in the vicinity of the flexural reinforcement were similar for all B100 beams (Figures 5-33 to 5-35). B120Y beams had up to a 50% reduction in crack width compared to B120N beams throughout in the tension zone. The experimental data suggests that the intermediate bar was effective at reducing the crack width on each side face of the beam and throughout for B120 beam (Figures 5-63 to 5-65).

In this research project, the minimum steel reinforcement was provided to maximize the tension zone depth. The small amount of steel reinforcement is less effective at limiting crack widths, hence, the reason for wider cracks at the extreme tension zone. The tension zone depth at reference load (50% of yield load) was 660 mm and 815 mm from the bottoms of B100 and B120 beams, respectively. The intermediate bar was located at a distance of 500 mm from the bottom for all BY beams, which is relatively close to the neutral axis. A higher amount of flexural steel reinforcement would decrease the tension zone depth and put the intermediate bar very close to the neutral axis or in the compression zone. Steel reinforcement near the neutral axis or in the compression zone would have little to no effect on limiting crack width or crack locations. Intermediate reinforcement will have more effect located closer to the flexural reinforcement (Figures 5-59 to 5-60). The crack width data between the flexural reinforcement and the neutral axis was observed to be maximum at 200 mm to 300 mm. A suggested location of the intermediate bar is 200 mm above the flexural reinforcement instead of 400 mm. In addition, it was observed that the crack widths were small in the upper 50% of the beam height suggesting that intermediate reinforcement is not beneficial in this region. Following this suggestion, the intermediate bar would be 300 mm above the bottom of the beam close to where the maximum intermediate crack width was located. The new location of the intermediate bar would more effectively control intermediate crack widths and increase moment capacity when compared to the old location.
According to the standard (CSA S304.1, 2004a), intermediate reinforcement needs to be installed when the beam is taller than 600 mm. The beam specimens tested in this study were 1000 mm and 1200 mm in height. Based on the results of this study, the intermediate crack widths were shown to be below the maximum allowable values of 0.33 mm and 0.4 mm for exterior and interior exposure conditions, respectively (CSA S304.1 2004a) which was adopted from CSA A23.3 2004b, Clause N10.6.1. The crack width results for B100 beams indicate that, although the addition of intermediate reinforcement reduced intermediate crack widths, the intermediate crack widths for B100N beams were all within 0.4 mm (Figures 5-33 to 5-34). This would suggest that B100 beam specimens do not require intermediate reinforcement to limit intermediate crack widths up to yield. Future research may investigate the use of joint reinforcement as longitudinal web reinforcement should be investigated. Joint reinforcement has about 10% - 18% the reinforcement area of a No. 15 bar, however joint reinforcement can easily be placed at 200 mm vertical spacing. In large reinforced concrete beams (h > 750 mm), it has been proven that skin reinforcement is more effective at limiting side face crack widths at smaller concrete covers (Frantz & Breen, 1980). Joint reinforcement has a masonry cover of 15 mm to 20 mm. In the literature review, Chapter 2.10, it was shown that welded wire fabric (Figure 2-18b) is effective at controlling side face cracks in concrete (Frosh, 1999) which has close to the same reinforcement area as joint reinforcement. Joint reinforcement can potentially be effective at side face crack control in masonry, however, research data is needed to justify this claim. Joint reinforcement can also be placed at 200 mm from the bottom of the beam where the side face crack was observed to be the widest.

6.2 Conclusion

The following conclusions are made based on the data obtained from this study, which was completed using only one concrete block type, grout mix, mortar mix, beam testing procedure. The conclusions of this study are also based on the cracking behaviour of beam specimens loaded up to the yield load ($P_y$). Hence, these conclusions are limited to
the scope of work and beam specimens in this study and may not be applicable to other masonry construction:

• There are similarities in the cracking pattern of large reinforced masonry and concrete beams; however flexural cracking in reinforced masonry beams is largely controlled by head joint spacing.

• The addition of intermediate reinforcement was found to reduce side face crack widths and gave an evenly distributed crack pattern up to yield.

• Based on these results, It may be beneficial to place the first intermediate bar 200 mm above the flexural reinforcement, distributed up to half the beam height.

6.3 Recommendations for Future Research

Based on the results of this experimental research project, the following recommendations should be considered:

• Conducting more experiments altering materials and configurations such as: block type and strength, mortar strength, grout strength, amount and type of steel reinforcement (intermediate and flexural), and beam specifications (height, width, and span).

• An investigation on other more efficient ways to collect crack width data during testing.

• The use of joint reinforcement as a replacement for intermediate reinforcement should be investigated to determine effectiveness.
## Appendix A: Mortar Strengths

Table A-1: Phase One Mortar Strength Data for Batches 1 to 10

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<th>Specimen</th>
<th>Elapsed Days</th>
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<th>C.O.V. (%)</th>
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### Table A-2: Phase One Mortar Strength Data for Batches 11 to 21

**Casting Date: August 16 - 17, 2010**

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### Table A-3: Phase Two Mortar Strength Data for Batches 22 to 30

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Appendix B: Grout Strengths

Table B-1: Phase One Grout Cylinder Strength Data for Batches 1 to 10

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Casting Date: August 23 - September 3, 2010
Table B-2: Phase One Grout Core Strength Data for Batches 1 to 10

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Table B-4: Phase Two Grout Core Strength Data for Batches 11 to 18

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Appendix C: Prism Stress and Strain Data

Figure C-1: Stress – Strain Relationship for P1

\[
y = 23056x
\]

Figure C-2: Stress – Strain Relationship for P2

\[
y = 17200x
\]
Figure C-3: Stress – Strain Relationship for P3

Figure C-4: Stress – Strain Relationship for P4
Figure C-5: Stress – Strain Relationship for P5

\[ y = 17135x \]

Figure C-6: Stress – Strain Relationship for P6

\[ y = 21940x \]
Figure C-7: Stress – Strain Relationship for P7

$y = 21983x$

Figure C-8: Stress – Strain Relationship for P8

$y = 20736x$
Figure C-9: Stress – Strain Relationship for P9

Figure C-10: Stress – Strain Relationship for P10
Figure C-11: Stress – Strain Relationship for P11

Figure C-12: Stress – Strain Relationship of P12
Figure C-13: Stress – Strain Relationship of P13

\[ y = 19016x \]

Figure C-14: Stress – Strain Relationship of P14

\[ y = 20751x \]
Figure C-15: Stress – Strain Relationship of P15

The graph shows the stress-strain relationship with the equation:

\[
y = 14855x
\]
Appendix D: Load Displacement

Figure D-1: Load-Displacement Curve for B100N1

Figure D-2: Load-Displacement Curve for B100N2
Figure D-3: Load-Displacement Curve for B100Y1

Figure D-4: Load-Displacement Curve for B120Y2
Figure D-5: Load-Displacement curve for B120N1

Figure D-6: Load-Displacement Curve for B120N2
Figure D-7: Load-Displacement Curve for B120Y1

Figure D-8: Load-Displacement Curve for B120Y2
Appendix E: Linear Potentiometer Data

Figure E-1: Linear Potentiometer Strain for B100N1

Figure E-2: Linear Potentiometer Strain for B100N2
Figure E-3: Linear Potentiometer Strain for B100Y1

Figure E-4: Linear Potentiometer Strain for B100Y2
Figure E-5: Linear Potentiometer Strain for B120N1

Figure E-6: Linear Potentiometer Strain for B120N2
Figure E-7: Linear Potentiometer Strain for B120Y1

Figure E-8: Linear Potentiometer Strain for B120Y2

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Appendix F: Deflection Profiles

Figure F-1: Deflection Profile for B100N1 at $P = 60$ kN and $P = 110$ kN

Figure F-2: Deflection Profile for B100N2 at $P = 60$ kN and $P = 110$ kN
Figure F-3: Deflection Profile of B100Y1 at $P = 60$ kN and $P = 120$ kN

Figure F-4: Deflection Profile of B100Y2 at $P = 60$ kN and $P = 120$ kN
Figure F-5: Deflection Profile for B120N1 at P = 60 kN and P = 110 kN

Figure F-6: Deflection Profile for B120N2 at P = 60 kN and P = 110 kN
Figure F-7: Deflection Profile for B120Y1 at $P = 70 \text{ kN}$ and $P = 130 \text{ kN}$

Figure F-8: Deflection Profile for B120Y2 at $P = 70 \text{ kN}$ and $P = 130 \text{ kN}$
Appendix G: Steel Strain Data

Figure G-1: Flexural Steel Strain for B100N1

Figure G-2: Flexural Steel Strain for B100N2
Figure G-3: Flexural and Intermediate Steel Strain for B100Y1

Figure G-4: Flexural and Intermediate Steel Strain for B100Y2
Figure G-5: Flexural Steel Strain for B120N1

Figure G-6: Flexural Steel Strain for B120N2
Figure G-7: Flexural and Intermediate Steel Strain for B120Y1

Figure G-8: Flexural and Intermediate Steel Strain for B120Y2
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