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LAP SPLICES OF DEFORMED BARS
IN REINFORCED CONCRETE BLOCK MASONRY WALLS

by

© DANIEL B. UNIAT, B.ENG.

A thesis submitted to
The Faculty of Graduate Studies and Research
in partial fulfillment of the
Master of Engineering

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December 5, 1983

The undersigned recommend to the Faculty of Graduate Studies and Research acceptance of the Thesis "Lap Splices of Deformed Bars in Reinforced Concrete Block Masonry" submitted by Daniel B. Unia't, B.Eng., in partial fulfillment of the requirements for the degree of Master of Engineering.

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ABSTRACT

The present trend to Limit States Design has highlighted the need for more knowledge of the failure modes and capacities of all structural materials. Although in the recent past, advances have been made in the understanding of the behavior of reinforced masonry block members, including research on masonry block walls for shear and axial compression capacity, as well as bond and development length tests on reinforcement in masonry block beams, very little has been done in testing masonry walls for flexural capacity alone. The present design practice is to use working stress in assessing the load resisting capacity of reinforced masonry members. One area that requires research is the flexural capacity of thin masonry block walls, and more specifically the establishment of criteria for bond and development length of reinforcement in these walls.

This thesis presents the evolution of the test program performed on 17 thin reinforced masonry walls and 2 unreinforced walls, loaded in the flexural mode, in order to establish bond and development lengths of reinforcement for these walls. The effects of lapped reinforcement versus continuous bars, bar size, effective depth, multiple bars, and fill material are investigated for their effect on bond on the reinforcement and the flexural capacity of the walls. The test results on two unreinforced walls loaded in the flexural mode are also compared.

This program evaluated the development lengths required to develop yield of the reinforcement, presents recommendations for further tests on lapped bars in thin masonry block walls, and suggests recommendations for lapped splices based on the test results. It also evaluates the ultimate flexural capacity of these walls.

It is important to note that the lapped splices in these test walls were located in a constant moment, zero shear zone and that the conclusions derived in this paper are for these load conditions only.

The paper firstly, defines the purpose and scope of the thesis, reviews the current literature, describes the solution, testing and instrumentation of specimens, evaluates and compares the data to present code requirements, suggests recommendations for development length and flexural moment capacity, summarizes the conclusions reached from the test results and makes recommendations for further research.

The limitations on the program were constraints of time, space and economy in the construction of the associated and wall specimens, and test frame. No bond failures resulted in the tested walls prior to yield of the reinforcement to accurately evaluate the required development length. The variability of the grout and mortar fill, meant to simulate actual construction in the field provided insufficient test data to evaluate the performance of the grout versus mortar fill.

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LIST OF SYMBOLS

A_b	Area of steel bar
A_s	Area of steel reinforcement
b	Width of beam or wall
d_b	Bar diameter
d	Effective depth of beam
E_s	Modulus of elasticity of steel
f'_c	Compressive strength of concrete
f'_m	Compressive strength of masonry
f_s	Stress in main reinforcement
f_y	Yield stress of steel
l_d	Development length
M_u	Maximum bending moment
RC	Reinforced concrete
u	Bond stress
ϵ	Strain
σ	Stress
ϕ	Capacity reduction factor
ρ	Ratio of steel area to effective beam area (A_s/bd)

CHAPTER 1

INTRODUCTION

1.1 GENERAL REMARKS

The building codes embody a summary of research tests and provide guidelines for structural designers. Generally, most designers will follow the methods of design and apply the design loads specified by the codes. For the design of reinforced masonry structural members, the building codes in Canada and the United States outline a working stress design (WSD) approach, a method whereby a structural member is designed so that the stresses that result from assumed magnitudes of service loads intended for the structure do not exceed specified allowable values. The stresses are computed on the basis of elastic members, so that the stresses under the action of applied service loads are well within the elastic range, and a "straight-line" variation between stress and strain is used.

Since 1963, with the appearance of the ultimate strength design method for reinforced concrete in the ACI Code there has been a transition towards the more rational design approach of limit states design (LSD) methods. In the limit states design method, the service loads for intended use of the structural element are increased by appropriate factors and the member is designed to provide the required performance with acceptable levels of safety against varying limiting conditions such as ultimate strength, damage and serviceability.

This thesis is concerned with the investigation of ultimate strength limits, specifically the development lengths required to develop yield stress of the reinforcement without loss of bond between the reinforcement and fill material in concrete block masonry walls. Further, the thesis evaluates the ultimate flexural strength of thin reinforced block walls and compares the results to present code

requirements. The performance of the reinforced masonry walls under load to failure also provides an indication of the ductility of walls in the flexural mode.

1.2 PURPOSE AND SCOPE

The present codes for reinforced masonry members CSA S304 in Canada (18) ACI 531-79 in the United States (7) indicate procedures for design for bond and anchorage. Thin masonry walls in practical applications are often used as a backup structural member to support exterior cladding. Under such applications, the masonry walls, if not loadbearing, are subject to relatively light laterally applied loads normal to the face of the wall, resulting from wind or earthquake forces. As such, because of the relatively light lateral loads, development lengths for anchorage of the reinforcement is not a significant problem, but lapping of the reinforcement, dependant on bond stresses from flexural action, is significant in the performance of these walls. The main object of the thesis is related to the problem of lapped splices in reinforced concrete masonry walls.

CSA S304(18) specifies an allowable bond stresses of 160 psi for all deformed bars in grouted cores of masonry walls, and is based on the WSD method.

The development length, l_d , can be computed from the following basic equation:
$$l_d = \frac{d_b f_s}{4u}$$

where d_b = bar diameter

f_s = steel stress of the reinforcing steel

u = ultimate bond stress per unit of surface area of the bar.

The ultimate bond stress is known to be influenced by many factors:

- (a) Size of bar
- (b) Clear cover to the reinforcement
- (c) Bar spacing

- (d) Strength of concrete block masonry
- (e) Number of bars in a layer
- (f) Embedment length
- (g) Shear and flexural cracks
- (h) Dowel force
- (i) Bar deformation heights and spacing
- (j) Bar deformation patterns
- (k) Position of bars: top or bottom bars
- (l) Dimensions of the structural member
- (m) The nature of force in the bar: tensile force or compressive force.

The present codes, noted above, indicate a single allowable bond stress for deformed bars in reinforced masonry construction, and do not account specifically for multiple bar splices, masonry and fill strength, or cover to reinforcement. The level of safety against bond failure in reinforced masonry construction, using present code recommendations, is difficult to predict.

This paper does not attempt to touch on all the variables associated with ultimate bond stress for establishing development lengths of reinforcement in masonry walls.

The main objectives of this program are:

- (1) To investigate, by means of a wall test program, the development lengths required for 10, 15 and 20mm bars in lapped splices for flexural bond stresses in thin masonry walls. This investigation is to determine the splice length of reinforcement necessary to develop yield stress without bond failure under these conditions. It is to be noted that the lapped splices in these test walls were made in a high moment, zero shear zone.

- (2) To investigate the effect of using more than one bar splice on ultimate bond stress.
- (3) To investigate the effect of grout fill versus mortar fill on ultimate bond stress. This is of practical importance, since mortar is a viable material for core filling in actual construction.
- (4) To investigate the effect of bar diameter on ultimate bond stress for spliced reinforcement.
- (5) To investigate the flexural capacities of the reinforced masonry walls using methods similar to the ultimate strength design methods used in reinforced concrete.
- (6) To compare the wall test results to present masonry code requirements and to reinforced concrete behavior for similar conditions.
- (7) To recommend development lengths for lapped splices in reinforced masonry block walls for flexural bond stresses.
- (8) To recommend areas for further research for reinforced masonry walls under the action of bond and flexure.

CHAPTER 2

LITERATURE REVIEW

Publications dealing with bond strength in reinforced masonry appear to be limited to reinforced brick masonry and date back to 1930 or earlier. There does not appear to be literature on concrete masonry block bond stresses, although studies have been conducted recently at Carleton University on reinforced masonry block beams (10,50). In reinforced concrete there is a great deal of published material dealing with bond and development of reinforcement.

2.1 REVIEW OF PREVIOUS RESEARCH IN REINFORCED MASONRY

HANSEN's paper, "Developments in Reinforced Brick Masonry" (28) published in 1933, collected data of numerous tests made on brick masonry. Bond failures occurred in only eight of the beams tested. Bar sizes were No. 2, No. 4 and No. 6 plain bars; ultimate bond stresses ranged from 82 to 236 psi. Hansen recommended 80 psi as the allowable bond stress for plain bars, a value still used in present codes.

GALLAGHER (27) in 1935 investigated pullout and pushout specimens, using No. 2 and No. 3 bars in brick masonry. He also studied the effect of mortar fill and joint thicknesses. He concluded that ultimate bond stress increased with increased mortar fill cover and that pushout bond stresses were higher than pullout bond stresses.

RICHART (47) in 1949 investigated effects of mortar type, bar diameter, embedment length, and position of the reinforcement during casting, using brick pullout specimens and No. 4 deformed bars. He concluded that the development for the 44 ksi yield bars was 8", and that horizontal, or vertical casting of the specimens had no significant effect on the ultimate bond strength.

BAYNIT (10) at Carleton University in 1980 investigated the effects of ultimate bond stresses on a total of 15 reinforced block masonry beams using No. 4, No. 6 and No. 8 deformed bars with a minimum yield strength of 64.0 ksi. He also carried out a total of 27 pullout tests in block masonry using the same bar sizes and similar embedment lengths. The beam specimens were nominal 8" block width two different effective depths of 6" and 13.5"; beam lengths were varied as well as the number of bars lapped. The beams were cast in a horizontal position. The fill material for the beams and pullout specimens were high slump (10") fine grout ($f'_c = 3181$ psi), low slump (4") fine grout ($f'_c = 3877$ psi), and high slump (10") mortar ($f'_c = 1863$ psi).

Based on his tests, Baynit's conclusions were that in reinforced masonry block beams,

- ultimate bond stress is higher for smaller diameter bars than for larger diameter bars
- ultimate bond stress decreases with the number of bars in a layer
- ultimate bond stress is dependent on the value of f'_m , the block prism strength
- ultimate bond stress decreases with closer bar spacing
- pullout tests overestimate the bond strength of an actual concrete block masonry beam.

He recommended basic development lengths of 12" for No. 4, 17" for No. 6, and 42" for No. 8 deformed reinforcing bars with a yield strength of 60.0 ksi using high slump grout fill. He also proposed modifications to this basic development length to account for number of bars, and type of fill material used. In addition, he also recommended that a capacity reduction factor be used of 0.6 to 0.7 to account for the variability of strength in masonry construction.

2.2 PRESENT MASONRY CODE REQUIREMENTS

CSA S304(18) states the allowable bond stress of 80 psi for plain bars, and 160 psi for deformed bars. The ACI Standard 531(7) states the required development length l_d to be equal to $0.015 d_b f_s = 12"$ for deformed bars, and is also based on maximum allowable bond stress of 160 psi for deformed bars. Both codes give the same values for the same size and strength of bars. The allowable bond stress is based on Hansen's 1933 recommendation of 80 psi for plain bars which is doubled for deformed bars to allow for the increase in bond due to the bar deformations. No allowance is made in the codes for factors such as bar diameter, compressive strength of masonry, or brick versus block construction.

2.3 REVIEW OF PREVIOUS RESEARCH IN REINFORCED CONCRETE

Bond stress is the result of the rate of transfer of load from concrete to steel; where there is a bond stress there is a change of steel stress. In order that the steel and surrounding concrete act together, there must be no slip of the bar relative to the concrete, at least up to the point of development of the yield stress of the bar.

The resistance to slip can be provided by adhesion between concrete paste and the surface of the bar, as well as frictional drag against the bars. With deformed bars, the bond strength is largely dependant on bearing of the lugs against the concrete, thus providing a mechanical interlock of the bar ribs with the concrete.

Sufficient strength of concrete is required to resist forces of the reinforcement in both longitudinal and radial directions. A pullout bond failure indicates no resistance to the longitudinal movement of the steel due to crushing of the concrete at the lugs as well as loss of adhesion and loss of frictional resistance. In a splitting failure, the concrete splits radially around the bar due to

the wedging action of the lugs against the concrete, creating tensile stresses in the concrete.

Most bond failures of deformed bars are a combination of the two modes of bond failure noted above. Initially the wedging action of the lugs of the tension reinforcement leads to splitting of the surrounding concrete. With progressive splitting of the concrete and the resultant loss of concrete confinement, pullout finally occurs.

A brief summary of the various investigations on bond in reinforced concrete follows. Some important parameters affecting bond of deformed bars are:

- (i) Concrete strength
 - (ii) Bar diameter
 - (iii) Concrete cover
 - (iv) Development length
- (i) Concrete Strength: bond strength is directly proportional to the tensile strength of the concrete, that is, the square root of the compressive strength of the concrete. Test results by various researchers support the influence of this parameter on bond (24, 33, 37, 42).
- (ii) Bar Diameter: bond strength is inversely proportional to the bar diameter, that is, an increase of bar diameter decreases bond strength. This parameter has been extensively investigated in reinforced concrete (33, 40).
- (iii) Concrete Cover: bond strength generally increases with increased cover to the reinforcement. The influence of this parameter has been well documented in reinforced concrete and formulae have been developed to predict the influence of concrete cover on ultimate bond strength (6, 33, 44).
- (iv) Development Length: bond stress generally decreases with increased development length. These findings were reported by Ferguson and Thompson (25).

Some other studies which have been carried out on bond strength are noted briefly below;

- Orangun, Jirsa and Breen (44) developed an empirical formula from test data to predict the beneficial effect of transverse (shear) reinforcement on bond.
- Kemp and Wilhem (33) developed an empirical formula, from test data, to predict the effect of dowel forces of the reinforcement in the area of shear cracks on bond capacity.
- Clark (19) in his investigations found that top bars (bars with more than 12" of concrete below them) developed 30 to 40 percent lower bond strengths than bottom bars.
- Lutz (37) developed an empirical formula, from test results, indicating the influence of the number of bars in a layer (among other parameters) on the ultimate bond stress.

2.4 PRESENT REINFORCED CONCRETE CODE REQUIREMENTS

ACI Committee 408(6) presented recommendations for code provisions for development, splice length and hooked anchorages for bars in tension. According to the recommendations, the basic development length for any cover of the bars in a particular layer where the clear cover of the bars is not less than 2" and the center-to-center spacing between bars is not less than 5" shall be taken as

$$l_d = \frac{23d_b}{\phi} \quad \text{for \# 6 and smaller}$$

and for f'_c equal to or greater than 3000 psi, shall be taken as

$$l_d = \frac{2200 A_b}{\phi \sqrt{f'_c}} \quad \text{for \# 7 and large}$$

ϕ = capacity reduction factor for development length and splices = 0.8

The basic development length shall be multiplied by $\frac{f_y}{60000}$ for reinforcement having yield strength other than 60000 psi. The recommendations also include special requirements for top bars, lightweight aggregates, over-reinforced sections, concrete cover and bar spacing, shear reinforcement, and bundled bars.

The CSA A23.3 Standard (2) Article 10.6, code provisions are presented below. The metric equivalent of the above code (16) presents similar requirements as does ACI 318-77 (3). The development length, l_d , in inches, of deformed bars and deformed wire in tension shall be computed as the product of the basic development length of item (a) and the applicable modification factor or factors of item (b), but l_d shall not be less than 12 inches except in the computation of lapped splices.

- (a) The basic development length shall be $0.04 A_b f_y / \sqrt{f'_c}$ but not less than $0.0004 d_b f_y$.
- (b) The basic development length shall be multiplied by the applicable factor or factors for:
 - (i) Top reinforcement (1.4)
 - (ii) Bars with f_y greater than 60,000 psi $(2 - \frac{f_y}{60000})$

The code also gives the modification factors for lightweight concrete, wider bar spacing, larger reinforcement area than required by flexure, and for bars enclosed with a spiral.

The development lengths recommended in the present codes (3, 16) are based on the values of ultimate bond stresses obtained from test results (44) which were present as code requirements in ACI 318-63(2). These ultimate bond stress values are derived from the use of the equation

$$u = \frac{9.5 \sqrt{f'_c}}{d_b}$$

The development lengths, in the present codes (3, 16), are based on the assumption that the bar stress must reach $1.25 f_y$ in order for the anchorage to perform satisfactorily (44). The bond stresses are $5/6$ of those recommended in ACI 318-63. The recommendations by ACI Committee 408(6) further increase the basic development length. An example of the lengths required for a No. 5 deformed bar, 60.0 ksi yield, 3000 psi concrete using the code recommendations and the formula used for the test walls are:

$$\text{by CSA A 23.3} \quad l_d = \frac{.04 A_b f_y}{\sqrt{f'_c}} = 13.6" (345\text{mm})$$

$$\text{by ACI 408 Committee} \quad l_d = \frac{23 d_b}{\phi} = \frac{14.3}{0.8} = 18.0" (457\text{mm})$$

$$\text{for the test walls} \quad l_d = \frac{(d_b)^2 f_y}{4 \times 9.5 \sqrt{f'_c}} = 11.3" (286\text{mm})$$

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1. SELECTION OF TEST SPECIMENS

3.1.1 Primary Specimens

In the selection of wall specimens there were obvious constraints of space, time, equipment, as well as the financial limitations of a large program of wall tests. Due to the generous contributions of the organizations and people listed under the Acknowledgements, the experimental program was able to proceed to completion.

The wall specimens chosen to be constructed were 200mm metric block, 600mm wide, 3.65m high. The block was a two-core hollow block, 14.5 MPa net area strength, 53% hollow. The center core of the 600mm wide wall was filled with grout or mortar and reinforcement was positioned in the required location. A total of 19 walls were tested, 17 reinforced and two without reinforcement. Block and wall details are shown in Figure 3.1 and Tables 3.1 and 3.2. The wall numbers used for designating the test walls during testing and recording of measurements taken were subsequently renumbered for presentation in this paper. As an example, Wall 9A at testing was renumbered to Wall 9 in this paper.

The walls were loaded laterally at the third points; the reinforcement was lapped at the mid point in order to have a constant moment with zero shear across the lap. A loading frame was designed and fabricated as part of the test program. Loading frame details are provided in Appendix A.

Table 3.1 Types of Full Scale Walls Tested

A	B	C	D	E	F
PLAN - NO REINFORCING STEEL AND NO GROUT	ONE CORE GROUTED	<p>NO LAP, 1 FULL LENGTH BAR.</p>	<p>STRAIN GAUGE LOCATION TYP.</p>	<p>GROUT OR MORTAR FILL</p>	<p>TOP BARS</p>

NOTE:

- ALL WALLS 3.6M x 200MM x 600MM WIDE - 2 CORE HOLLOW BLOCK.
- ALL TYPE 'S' MORTAR.
- ALL 2 POINT LOADING AT 1/3 POINTS.
- ONLY CENTER CORE FILLED.
- NO HORIZONTAL REINFORCING STEEL.
- ALL LAPS CENTERED ABOUT MID-HEIGHT OF WALL.
- REINFORCING LOCATION CENTERED ABOUT 600MM WIDTH OF WALL.
- ALL LAPS TO BE CONTACT LAPS EXCEPT WALL TYPE E.

TABLE 3.2

DESCRIPTION OF WALL SPECIMENS

WALL NUMBER	SPECIMEN DESIGNATION	TYPE (TABLE 3.1)
1	G.0 /N--/-	A
2	G.0 /G--/-	B
3	G.140/1-15/0	C
4	G.100/1-15/0	C
5	G.100/1-20/0	C
6	G.140/1-25/0	C
7	G.100/1-10/200	D
8	G.100/1-15/350	D
9	G.100/1-15/350	D
10	G.100/1-15/450	D
11	G.100/1-15/450	D
12	G.100/1-20/500	D
13	G.140/1-15/350	D
14	G.100/2-15/500	F
15	G.100/2-15/650	F
16	G.* /1-15/350	E
17	M.100/1-15/450	D
18	M.100/1-15/450	D
19	M.100/1-15/350	D

EXAMPLE OF SPECIMEN DESIGNATION:

G. 100 / 1 - 15 / 350

G - GROUT FILL
 M - MORTAR FILL
 100 - EFFECTIVE DEPTH (mm)
 1 - NUMBER OF BARS
 15 - R/S BAR DIAMETER (mm)
 350 - LAP LENGTH (mm)

N - INDICATES NO FILL AND NO R/S
 G - INDICATES GROUT FILL WITH NO R/S
 * - INDICATES MISPLACED BARS

FIGURE 3.1 PLAN: BLOCK SIZE AVERAGE DIMENSIONS(mm)

Figure 3.2 Associated Specimen Dimensions & Loading Arrangement

3.1.2 Secondary Specimens

Secondary specimens consisted of standard 6" diameter x 12" high cylinders of the grout in metal containers, 2" cube mortar prisms, and 4" x 4" x 8" high mortar and grout absorptive prisms. Block prisms constructed of 3 half blocks, that is 190mm x 190mm x 600mm high were taken using the grout and mortar for each pour in the actual wall construction. The secondary specimens were taken to evaluate the strengths of the materials used in the test walls. Tensile tests were taken on all reinforcement actually used in the walls. The types of secondary specimens are indicated in Figure 3.2.

3.2 DEVELOPMENT OF TEST PROGRAM

The types of walls tested were the following:

- (a) One wall with no reinforcement and no fill (Wall 1).
- (b) One wall with no reinforcement but with grout fill (Wall 2).
- (c) One wall with 1-15mm bar, no lap, grout fill at the maximum effective depth location (Wall 3).
- (d) One wall with 1-15mm bar, no lap, grout fill at the center line of the block (Wall 4).
- (e) One wall with 1-20mm bar, no lap, grout fill at the center line of the wall (Wall 5).

These walls were to be used for comparison of the similar walls with lapped reinforcement.

- (f) One wall with 1-25mm bar, no lap, grout fill at the maximum effective depth (Wall 6). This wall was tested in order to attempt a flexural compression failure of the block itself.
- (g) One wall with 1-10mm at the center line of the wall, grout fill, 200mm lap (Wall 7).

- (h) Two walls with 1-15mm at the center line of the wall, grout fill, 350mm lap (Wall 8 and 9).
- (i) Two walls with 1-15mm at the center line of the wall, grout fill, 450mm lap (Wall 10 and 11).
- (j) One wall with 1-20mm at the center line of the wall, grout fill, 500mm lap (Wall 12).
- (k) One wall with 1-15mm at the maximum effective depth, grout fill, 350mm lap (Wall 13).

The walls above were selected on the basis that the 15mm bar at the center line of the 200mm wall is most frequently used in actual construction. Therefore, two walls in item (h) were to be the most important of the tests. The other walls, items (g) to (k), were selected to vary several other parameters; size of reinforcement (wall 7 and 12), effective depth, (wall 13), and varying the lap lengths, (walls 8 and 9 versus walls 10 and 11.)

- (l) Two walls with 2-15mm bars at the center line of the wall in grouted cores, one with lap lengths of 500mm (Wall 14), and one with 650mm lap lengths (Wall 15). These walls were tested to compare the effect of lapping two bars versus one.
- (m) One wall with 1-15mm bar with 350mm lap, grouted core, with the reinforcement placed at either face of the wall. This wall was tested to measure the effect of misplaced bars (Wall 16).
- (n) Two walls with 1-15mm bar, 450mm lap at the center line of the wall with mortar fill (Walls 17 and 18). These walls were tested to compare mortar versus grout as fill material. The lap length was increased to compensate for the lower bond capacity of mortar versus grout fill of equivalent compressive strengths.

- (o) One wall with 1-15mm, 350mm lap at the center line of the wall (Wall 19), mortar fill for a direct comparison with the grout filled walls 8 and 9.

The bond lengths required were estimated using the basic formula:

$$u = \frac{9.5 \sqrt{f'_c}}{d_b}$$

and
$$l_d = \frac{d_b f_s}{4u}$$

i.e.
$$l_d = \frac{(d_b)^2 \times f_s}{4 \times 9.5 \sqrt{f'_c}}$$

f_s was taken to equal f_y of the reinforcement. Samples of the 15mm bar supplied was tested and the resultant yield strength of 500 MPa was used in the calculation; f'_c was taken to equal f'_m .

Using a block strength of 20.0 MPa, $l_d = 352$ mm. This lap length should have ensured a bond failure at yield of steel. The basic lap length actually used for single lap 15mm bars was 350mm as noted for Walls 8, 9, 13, 16 and 19. The same procedure was used to establish lap lengths for the 10 and 20mm single bar laps in walls 7 and 12. For double bar laps the lap lengths estimated by the formula were multiplied by a factor of 1.4 as suggested by the reinforced concrete codes. For multiple lap reinforcement for a 15mm bar the lap required is $350 \times 1.4 = 490$ mm. A lap length of 500mm was used in Wall 14.

Since the area of the grouted block core is relatively small, it was thought that this increase in lap length for multiple laps may be insufficient to develop the steel to yield and the lap was increased by a further 30 percent to 650mm as indicated in Wall 15.

Lower shear values in the range of 25 percent are indicated for mortar fill versus grout fill of the same strengths used in reinforced masonry beams (50). In concrete, ultimate shear stress values, and similarly ultimate bond stress values, are functions of the splitting tensile strength of the concrete which varies as the square root of the compressive strength of the concrete. In the mortar filled Walls 17 and 18, the lap was increased by 25 percent to account for differences in the fill material. For the 15mm single splice bars, in Walls 17 and 18, the lap was increased by 25 percent to 450mm. In Wall 19, with mortar fill, the lap was kept at 350mm in order to compare the results to the same lap used in the grouted Walls 8 and 9.

3.3 MATERIALS

3.3.1 Concrete Block Units

All the walls tested were constructed using one type of block unit of the same strength. All the block was shipped in one load to the test site from the manufacturer. The block unit (Figure 3.1) was a standard 200mm thick metric block, 2 core unit, 200mm high x 400mm long, nominal dimensions. The block strength ordered was 7.5 MPa (gross area), 14.5 MPa (net area).

Originally, in the test program it was intended to use a higher strength block, to evaluate the results of different strengths on the lap lengths required for the reinforcement, but at the time of the testing no higher strength block was available.

3.3.2 Mortar

The mortar used was Type S. The mortar was mixed at the test site by the

experienced masons, using an electrically driven mixer. The masons were instructed to use normal Portland Cement (Type 10, CSA-A5), masons hydrated lime, and mortar sand proportioned as referenced in CSA-A179M-1976 Standard (14) using approximately 1 part cement, 1/2 part lime and 4-1/2 parts sand by volume.

The lime, portland cement and mortar sand were supplied by the masons, and were stored and mixed inside the warehouse used as the test site. The masons adjusted the amounts of lime, cement and water to ensure proper workability as normally done on actual job sites. The mortar used for joints of the masonry prisms and block wall were in the order of 4" slump, the mortar fill for the block wall cavity was 10" slump.

Mortar cubes were taken of the low slump mortar used in the joints and of high slump mortar used for fill. Absorptive prisms and non-absorptive cylinders were taken of the mortar fill.

A sieve analysis of the masonry sand used was made (Table 3.3). Its gradation did not conform to the requirements of CSA-A82.56M-1976. However, this sand was used on numerous projects in actual construction, and was used here, with no apparent detrimental effects on the performance of the walls.

3.3.3 Grout

The grout fill used was a coarse grout supplied from a ready mixed truck. The volume of grout needed was approximately three cubic metres. In order to have consistency of the grout and to reduce the time of labour required to place the grout, a plant mix was used. The grout was in place approximately one hour after delivery. Absorptive prisms and non-absorptive cylinders of the grout were taken. The proportions of materials specified for the grout fill are noted in Figure 3.3.

TABLE 3.3 SIEVE ANALYSIS AND RESULTS OF MORTAR SAND

SIEVE	EMPTY WEIGHT (gm)	FULL WEIGHT (gm)	DIFF. (gm)	% PASSING
4	524.9	524.9	0	100
8	492.0	492.7	.7	99.9
16	460.0	461.1	1.1	99.7
30	394.6	406.1	11.5	98.1
50	391.2	595.1	203.9	68.4
100	351.1	685.0	333.9	19.9
200	348.8	467.9	119.1	2.6
PAN	356.8	374.9	18.1	0
			688.3	

WEIGHT OF EMPTY BOWL = 210.9gm
FULL = 901.2gm

DIFF. = 690.3gm

The Premier Group

a division of Lake Ontario Cement Limited

PRIMEAU
ARGO

July 21, 1980

Admiral Engineering & Construction
1619 Carling Avenue
Ottawa, Ontario
K2A 2G8

Attention: Mr. Dan Uniat

Dear Sir:

Re: 1590 Liverpool Court Project
Ottawa, Ontario

Listed below is a description of materials and sources, and specifically cited proportions, for the concrete on the above project. Dispatching and delivery will be made from our Bantree Street facility to your job site.

Materials:

Cement	- Normal Portland Type 10 Lake Ontario Cement Limited
Fine Aggregate	- Washed - Classified, Spratt's Sand and Gravel Limited
Coarse Aggregate	- 3/8" Crushed Limestone Ambro Materials
Water	- Municipal Supply
Water Reducing Admixture	- Construction Chemicals Limited
Air Entrainment	- Construction Chemicals Limited

Concrete Proportions:

The following proportions are for one cubic metre (1 m³), and quantities for the aggregate are saturated surface dry (S.S.D.) weights. Corrections for the moisture content in the sand and stone will be made during the batching process and will be determined on a continual basis.

Air-Entrained Concrete Proportions

1500 p.s.i. (10.4 m.p.a.), 3/8" Stone (10 m.m.)

<u>Imperial Weights</u>		<u>Metric Equivalent</u>	
Cement:	480 lbs.	Cement:	218 Kg
Sand:	2450 lbs. (S.S.D.)	Sand:	1111 Kg
Stone 3/8":	2000 lbs. (S.S.D.)	Stone 10 m.m.:	907 Kg
Water:	385 lbs.	Water:	175 Kg
W.R.D.A.:	4 oz per 100 lbs cement	W.R.D.A.:	113 grams per 45.3 Kg
A.E.A.:	5 ± 1 percent	A.E.A.:	5 ± 1 percent
Slump:	8 inches	Slump:	200 m.m.

1815 Bantree St., Ottawa, Ontario, K1B 4L6 (613) 741-1212

3.3.4 Reinforcing Steel

The reinforcing steel used for the walls were 10M, 15M, 20M and 25M metric size bars, grade 400 deformed bars. Before testing the walls, a representative sampling of the 10M, 15M and 20M bars were tested at Carleton University Laboratory on the Tinius Olsin machine. The reinforcement was inspected to be free of loose rust, scale, dirt and any deleterious material before being positioned and held in place in the walls. Subsequently, on demolition of the test walls, the reinforcement actually used in each wall was labelled, and tested. The results are indicated in Table 3.4.

TABLE 3.4 REINFORCING STEEL SPECIMEN RESULTS					
WALL NUMBER	TOP OR BOTTOM BAR	BAR SIZE	YIELD STRESS (ksi)	ULTIMATE STRESS (ksi)	% ELONGATION (in 8 inches)
3	-	15M	74.2	116.8	11.5
4	-	15M	72.9	116.5	9.3*
5	-	20M	71.3	113.8	9.5*
6	-	25M	61.2	103.6	15.5
7	TOP	10M	83.8	117.8	16.0
7	BOT	10M	84.1	117.8	16.0
8	TOP	15M	72.6	115.5	13.5
8	BOT	15M	73.9	117.1	12.0
9	BOT	15M	72.9	116.5	8.5*
11	TOP	15M	72.9	116.1	12.5
11	BOT	15M	72.6	115.5	13.5
12	TOP	20M	70.9	113.6	14.5
12	BOT	20M	70.2	113.0	12.0
13	TOP	15M	73.5	116.1	12.0
13	BOT	15M	72.3	115.8	15.0
14	TOP	15M	72.6	115.8	14.0
14	BOT	15M	73.5	116.8	14.0
14	BOT	15M	73.5	116.8	11.0
15	TOP	15M	73.2	116.5	9.5*
15	BOT	15M	72.6	115.8	13.5
15	BOT	15M	73.5	116.1	11.5
15	BOT	15M	72.9	116.1	9.0*
16	TOP	15M	72.9	116.5	15.0
16	BOT	15M	72.3	114.8	12.0
17	TOP	15M	78.0	121.9	13.0
17	BOT	15M	67.4	106.5	15.0
18	TOP	#5	72.3	116.5	11.0
18	BOT	#5	79.3	121.9	12.5
19	TOP	#5	68.3	114.8	9.5*
19	BOT	15M	68.7	107.1	15.0

TABLE 3.4 CONTINUED					
WALL NUMBER	TOP OR BOTTOM BAR	BAR SIZE	YIELD STRESS (psi)	ULTIMATE STRESS (ksi)	% ELONGATION (in 8 inches)
UNKNOWN	TOP	15M	71.9	115.5	11.0
"	TOP	15M	72.6	115.8	14.5
"	BOT	15M	72.6	115.8	13.5
AVERAGE		10M	83.9	117.8	16.0
"		#5&15M	72.8	115.8	13.0
"		20M	70.8	113.5	12.0
"		25M	61.2	103.6	15.5

NOTE:

* DENOTES FAILURE OUTSIDE OF 8 INCH GAUGE LENGTH

REFERS TO IMPERIAL BAR SIZE; # 5 INDICATES 5/8 INCH DIAMETER

3.4 CONSTRUCTION, INSTRUMENTATION AND TESTING

3.4.1 Construction of Wall Specimens

The location of the wall specimens were carefully laid-out at the warehouse test site, keeping in mind adequate room to manouever the test frame and to allow for taking of measurements. Sufficient area was also required^o for the collapse of the walls without damage to the adjacent test walls. A 1/4" thick oiled plywood sheet of the same plan dimensions as the test walls were then placed in each wall location on the floor.

The masons proceeded to construct the test walls 6 courses (1200mm high). The block was stored outside of the warehouse until required. All other materials, except for the grout were stored inside. The block walls were one and a half blocks long. Half blocks were split in the normal construction fashion by the masons using chisels as indicated in Figures 3.4, 3.5 from special three cell blocks.

The walls were built in running bond fashion to ensure no continuous vertical mortar joint. The resultant walls were 3 cores wide with the reinforcement and grout placed in the center core to achieve as symmetrical a cross section as possible. The center core was always one core of a full block, with the empty half blocks at the outside edge of the wall. There was slight stagger of approximately 20mm on the outside edges of the walls from course to course due to the use of the half blocks.

Figure 3.4 Mason at Work

Note: Special Blocks used for Half-Blocks

Figure 3.5 Wall Construction: breaking half block

The walls were constructed July 2nd and 3rd, 1980 to the 1200mm height. The nominal 1/2" thick bed joints were full mortar joints at the face and web shells, as were the head joints. This ensured that the core to be filled was fully mortared to prevent loss to grout. A small hole, approximately 100mm x 100mm minimum was broken out at the bottom course, in the centre of the wall on the tension face. The hole was used as a cleanout and to provide access to properly position the reinforcement. It was also used to visually ensure that the grout or mortar fill had fully filled the block core. The bottom block course was filled with mortar for the full width of this wall except for the center core. Figures 3.6, 3.7 illustrate the bottom cleanout and the wall construction at this stage. After cleaning out the loose or excess mortar from the center block core, the reinforcement was placed and tied in the core as shown in Figure 3.8.

The reinforcement was cut exactly to the length required to provide the required design lap lengths in each wall. Strain gauges were placed a distance of 50mm outside of the laps with the strain gauge wires from the bottom reinforcing leading down the core and out the bottom cleanout hole. The wires were installed in a 1/4 inch diameter plastic tube and were clipped to the reinforcement. Figures 3.9, 3.10 indicate the grouting procedure and strain gauge wire location. Where full length bars were used with no splice the strain gauges were placed in the mid-height section of the walls, the wires being installed in a similar fashion. Each bar had two strain gauges placed directly opposite each other on either side of the bar.

The strain gauges were prepared in the Carleton University Laboratory (Appendix B) and placed on the vertical rib deformation of the reinforcing. The reinforcement was placed in the cores with the vertical ribs perpendicular to the applied loads so that the strain gauges measured the steel stress at the faces of the bars closest and farthest from the applied loads. Figures 3.11, 3.12, 3.13 indicate details of the strain gauges.

Figure 3.6 Wall Construction : Bottom cleanout

Figure 3.7 Wall Construction to 1.2m Height

Figure 3.8 Wall Construction: bar positioned at 1.2m Height

Figure 3.9 Wall Construction to 1.2m Height:
grouting center core

Figure 3.10 Steel Strain Gauge wire leads from
bottom of wall

The steel reinforcement was positioned at the top of the 1200mm height with wire stays. The plywood sheet at the bottom of the wall included a small hole for positioning the bar. The reinforcement had a 1/4" x 3" square plate welded to the bottom end, with approximately 1/2" of the bar protruding past the plate. This plate was used to ensure no end anchorage slip of the reinforcement. (See Figures 3.6 and 3.8.) After the reinforcement was in place, the plant mix grout was placed by hand on July 4th, using a wood funnel to direct the grout into the core. The grout was in place in approximately one hour, to within 50mm from the top, leaving a recess for the next pour. (See Figures 3.9, 3.10 and 3.11.)

The next lift of 12 courses (2400mm) of wall were constructed beginning July 7th. The mason first placed the block as high as he could standing on the floor. Scaffolding was used to construct the upper portions of the walls. A hole was broken out above the 1200mm and 1800mm height block courses on the tension face of the wall on center line. The bottom hole again being used for a cleanout, and to visually inspect for grout fill. The top hole was used in order to enable the top bar to be placed and lapped in proper position and elevation. Where full length bars were used, the mason slipped the block over the reinforcement for the full 2400mm height of lift. (See figures 3.14, 3.15 and 3.16.) The strain gauge wires were again enclosed in a plastic tube, secured to the reinforcement and fed up the center core to the top of the wall. The reinforcement was tied in place to the bottom bar with wire. The construction of the block for the walls was completed on July 10th.

Figure 3.11 Steel Strain Gauge wire leads on reinforcing bar

Figure 3.12 Steel Strain Gauges in place on reinforcement

Figure 3.13 Steel Strain Gauge preparation

Figure 3.14 Wall Construction: Cleanouts at 1.2m
& 1.8m Height

Figure 3.15 Wall Construction: Single Bar Lap, Tied
in Place

Figure 3.16 Wall Construction Above 1.2m Height

The grout was poured on July 11th, 1980 in two more or less equal lifts for this 2400mm height. Each pour took about one hour, and was made in a similar manner to the first pour of July 4th. There was a fifteen minute wait for the ready mix truck to arrive after the first pour of the day. The second pour was completed in one hour. Due to limitations of the height of the warehouse, the pours were not rodded. Vibration of the grout was accomplished by wobbling the top reinforcing gently. The high slump of the grout ensured no problems with voids. No retempering of the grout joint between the two pours was attempted or required.

The reinforcement was accurately tied into place within 1/8 inch of required lap and location. On subsequent demolition of the walls after the tests were performed all the cores were shown to be fully filled and the reinforcement accurately in place.

For the walls with the mortar fill, the high slump mortar was placed by trowel by the masons in one lift, and puddled after the core was filled. The top of the reinforcement again had a 1/4" x 3" sq. plate welded to the bar to prevent end anchorage slip. The plate was grouted with "hydro-stone" to a tight bearing. The walls were laterally supported at the top of the building structure with wood struts to prevent movement and to stabilize the 3.6m high free standing walls.

On completion of the July 11, 1980 pour, the walls were left intact except for the necessary preparatory work until the start of testing on August 8, 1980, twenty-eight days later.

3.4.2 Instrumentation

The load side (compression face of the wall) was carefully marked to locate the points of measurements to be taken during testing. Figure 3.17 illustrates the locations and types of measurements taken.

The dial gauges used were attached to the dial gauge frame mounted on the test frame. The dial gauges were Starrett No. 655-5081 (2 gauges), and one No. 25-5081 with a 125mm range marked in divisions of .01mm. 1" x 1" x 1/16" metal plates were epoxy glued to the block to provide a flat surface for measurement. The dial gauge mounting frame was attached to the test frame at the top and bottom only, in order not to reflect deflections of the test frame members under load. The dial gauges were located at the center line of the wall at the 1300, 1800 and 2300mm heights above the floor.

Locating pins were positioned on the block face for measurements of strains on the compression face of the wall and were epoxy glued at the required location. The gauges used were one 2" Demec Gauge, W.H. Mayes & Sons Ltd., No. 442905 reading in divisions of .00001, one division representing a strain of 2.51×10^{-5} inches per inch. The other gauge was an 8" Demec Gauge, reading in divisions of .00001, one division representing a strain of 1.01×10^{-5} inches per inch. The gauges were checked for accuracy at the Carleton University Laboratories. Measurements were taken on the compression face of the walls with the 8" Demec across the center height mortar joint of the wall in three locations across the width of the wall, and with the 2" gauge at two locations slightly above this center height joint on the block itself, symmetrically about the width of the wall. One measurement was also taken with the 8" gauge on the rear (tension) face at the center width of the wall at the mid-height mortar course. All the measurements for the dial gauges and the strain gauges were taken and recorded manually during the tests. The measurements were taken on the wall in the lap area between load points. Figures 3.18 and 3.19 show the test frame and measuring devices in place and measurements being taken.

The load cells used were Interface Model No. 1210-AF, 10kip load cells supplied by the National Research Council for the tests. There were four cells used attached directly to the hydraulic jack pistons. The load cells rested against a 2" x 2" x .25" thick and 24" long (H.S.S.) hollow steel section. The hydraulic jacks applied the load through the load cells to the hollow steel section to the wall. A relatively stiff tube section was used to bear on the wall to equalize loads from possible rotation of the wall about its width. A one half inch tentest board was used between the block and steel tube to account for imperfections in the wall. The jacks were at 12 inches center to center about the center line of the wall width, and at the 1200mm and 2400mm height of the wall. The hydraulic jacks mounted to the test frame were 30 ton jacks with a 6" travel manufactured by Simms Eng. CO. The jacks were connected with individual hydraulic lines to a distribution chamber and then with a single line to an "Enerpac" hand operated hydraulic pump. The pump side of the hydraulic distribution chamber had a Helicoid gauge reading 0 to 2000 psi pressure. The load cells were originally calibrated in compression at the Carleton University Laboratories, using the Tinius Olsen testing frame. Maximum mv/v output was 4.137. The calibration factor of 1062 in compression was determined in accordance with the manufacturers specifications. A resolution of 1.0 lb was possible. The DC resistance of these load cells was 350 Ω .

Two strain gauges were fixed on each rebar as shown in Table 3.1. The standard three wire configuration was used to connect these gauges to the Budd SB-1 switch and balance units. Both load cells and strain gauges were connected to the switch and balance unit. It is noted here that the calibration factor for the strain gauges and the load cells were of different values. The load cell factor was 1062, the strain gauge factor was 985. The measurements were visually displayed on the connected Vishay/Ellis-20 digital strain indicator readout machine.

ITEMS

1, 3, 4	—————	6" DEMEC GAUGE - COMPRESSION FACE
6	—————	6" DEMEC GAUGE - TENSION FACE.
8, 5	—————	8" DEMEC GAUGE - COMPRESSION FACE.
7, 8, 9, 10	—————	BOTTOM REINFORCING STEEL STRAIN GAUGES - DOUBLE BAR LAPS.
17, 18	—————	BOTTOM REINFORCING STEEL STRAIN GAUGES SINGLE LAP.
15, 16	—————	TOP REINFORCING STEEL STRAIN GAUGES SINGLE LAP.
11, 12, 13, 14	—————	TOP REINFORCING STEEL STRAIN GAUGES - DOUBLE BAR LAPS.
L1, L2, L3, L4	—————	LOAD CELLS.
7, 8, 9	—————	DIAL GAUGES.

Figure 3.17 cont.

Figure 8.1.8 Test Primer Measuring
Devices in Place

Figure 3.19 Compression
Demo: Measurements

Details of these wiring details and measuring devices are noted in Appendix D. While the load was calibrated to read directly in lbs, the strain gauge readings were divided by 0.65 to obtain the strains in inches per inch for all single lap and full length bars. The wiring connections to the digital readout machine allowed for direct readings only on some of the incoming leads. In our case, the four load cell leads were connected to display the actual loads. The connections to the strain gauges were wired to display readout values that were 0.65 times the actual values. For the double bar laps, the strain gauge readings required no adjustment factor.

During the initial tests of Walls 7 and 19, no readings were available for one load cell at the 2400mm height of wall. The load cells were removed, the one malfunctioning cell replaced and then recalibrated at the university. The load cells were then remounted and the remainder of the 17 walls were tested with the four cells functioning. The load cell calibrations are shown in Tables 3.5 and 3.6.

3.4.3 Testing of Wall Specimens

The testing of wall specimens occurred from August 8, 1980 to August 23, 1980 inclusive, with a maximum of two walls tested on any day.

The procedure for testing was in general, as follows. Lines were marked on the floor parallel to the test walls to locate the rolled test frame. The test frame was placed in location centered about the test wall, wood strut shores at the test wall removed. The test frame was anchored in location into the warehouse concrete floor slab with a 5/8 diameter expansion bolt through an angle clip at the test frame legs to prevent any possible test frame movement during loading.

At the top and bottom of the walls a flat plate was placed on the tension side of the wall centered 100mm above the bottom and below the top of the wall. The plate was set in place on the wall with Hydro Stone, a high strength plaster often

TABLE 3.5 INITIAL LOAD CELL READINGS FOR WALLS 7 AND 19, TESTED ON AUG.10/80			
TINIUS OLSEN APPLIED LOAD (lbs)	LOAD CELL READINGS		
	LOAD CELL No.1	LOAD CELL No.2	LOAD CELL No.3
100	148	148	148
200	300	300	297
300	450	451	448
400	600	599	598
500	749	752	745
600	900	902	893
700	1048	1050	1041
800	1200	1202	1192
900	1346	1350	1340
1000	1486	1500	1487
1100	1643	1647	1634
1200	1788	1794	1778

TABLE 3.6 RECALIBRATED LOAD CELL READINGS FOR REMAINDER OF WALLS TESTED ON AUGUST 12, 1980				
TINIUS OLSEN APPLIED LOADS (lbs)	LOAD CELL READINGS			
	LOAD CELL No. 1	LOAD CELL No. 2	LOAD CELL No. 3	LOAD CELL No. 4
100	100	102	100	100
200	200	201	199	200
300	300	302	298	300
400	400	401	396	395
500	500	502	496	494
600	600	601	593	594
700	701	701	694	693
800	800	800	791	793
900	898	899	890	892
1000	998	996	986	994
1100	1095	1096	1084	1088
1200	1193	1192	1180	1185

used for capping test specimens. The plates were supported off the test frame with angle supports temporarily until the Hydro Stone was set. A 3" diameter solid steel round bar was then snugged in place onto the concave portion of the plate. A 3/4" diameter threaded rod attached to the main frame front legs was used to snug and secure the roller into place for the tests. Figures 3.20, 3.21 and 3.22 show some of these details. Appendix A indicates more details of the test frame. The hydraulic jack pistons were then moved into place to bear against the walls. A one inch diameter threaded rod was used for the pistons, with nuts to secure the piston in an outward position. As noted previously the jack pistons transferred the load through the load cells onto a 2" x 2" x 0.25" H.S.S. steel section at the 1200mm and 2400mm heights of the wall. The two H.S.S. sections were attached together by a similar H.S.S. section vertically between the two horizontal members to insure that the loads were always placed at the same distance vertically and to add rigidity to these bearing points. A pulley system was devised to position the pinned support angles carrying the upper roller and bearing plate as well as the 2" x 2" H.S.S. steel sections, and the dial gauge frame. The bottom support angles, again pinned at the front legs of the test frame were rotated, positioned and secured by hand.

The hydraulic pump was then pumped by hand to seat the jack pistons to bear against the block wall and initial readings were taken and recorded of the three dial gauges, the 6 Demec gauge readings, the four load cell readings, the hydraulic pressure gauge and the reinforcing bar steel strain gauge readings. There were four steel strain gauge readings for single lapped bars, two for the single non-lapped bars, and eight for the double lapped bars. The initial readings were taken when load cell readings were at a very low load level, that is, in the range of 5 to 10 lbs.

An increment of 15 psi on the hydraulic pressure gauge was found to correspond to an increment of approximately 200 lbs force on the load cell

Figure 3.20 Test Frame: Wall Bearing Plate and
3" Diameter Bar

Figure 3.21 Top of Wall Under Load (NOTE: Rotation of Bearing Plate and Crushing of Hydro Stone)

Figure 3.22 Bottom of Wall Under Load (NOTE:Rotation
at Bottom of the Wall)

readings. The estimated load to produce a yield stress on the reinforcement bar was calculated. A minimum of five, to a maximum of ten load increments of approximately equal magnitude were used to load the walls to yield of the reinforcement, using this estimated load as a guide. It was known when the actual yield level of the reinforcement was reached by reading the visual display of the Ellis-Vishay digital readout machine. A record was kept of the crack occurrences in the wall at load increments and photographs were taken during the tests.

After yield of the reinforcement was reached the walls were further loaded to the maximum 6" travel capacity of the jacks. At this time, the load on the wall was maintained by threading the nut on the jack piston rod to bear onto the jack body, releasing the hydraulic pressure on the jacks, resetting the jack internal piston and loading the jack pistons again for a further 6" travel. When the dial gauge capacity was exceeded measurements were taken by hand with a rule. It was found that after yield of the reinforcement the force required to further displace the wall was nominal, that is in the range of 20 percent of the force applied to achieve full yield of the bar.

After the walls were tested, and the jack pressures released, the steel angle support brackets, measuring devices, load cells were removed. On removal of the loads, the walls which had been deflected to a maximum of 12", would return to almost their unloaded position, displaying only a very slight bend.

The test frame was then moved to the next wall, the previously tested wall was demolished, and removed. The demolished walls were inspected visually for necking of reinforcement in the lapped areas, quality of grout filling. The reinforcement generally displayed yield at the strain gauge locations where the vertical deformations of the reinforcement had been ground smooth to mount the gauges, slightly reducing the net areas of the bars. The embedment of the bars

were visually inspected where possible, in the lapped areas. In all cases inspected, the cavities were well filled. There was no indication of the crushing of the grout or mortar at the bar deformations to indicate slip or bond failure. The effective depth of the bars was also measured and recorded. The bars were placed within 1/8" of their required location. The walls in stages of demolition are shown in Figures 3.23 and 3.24. The procedure was repeated until all the walls were tested.

3.4.4 Construction of Associated Specimens

(i) Masonry Prisms

The block prisms were constructed of half blocks, three blocks high, by the masons. The block was cut at Carleton University with their water cooled saw, to include the full center web of the half block, and then transported to the test site warehouse. An area was set aside in the warehouse for the storage of all the associated specimens, clear of the test walls. The masonry prisms were constructed on 1/4" oiled plywood sheets, early July 11, 1980 approximately 2 hours before being filled. All grout and mortar filled block prisms were taken with the material used in the test wall pours of July 11, 1980 in the critical 1200mm to 2400mm height.

The block prisms were puddled with a rod, and filled in three lifts. The block prisms were then left covered with polyethelene for ten days to properly cure. Figure 3.25 shows the constructed masonry prisms.

(ii) Standard 6" Diameter Concrete Cylinders

The standard 6" nominal diameter by 12" high cylinders commonly used for concrete strength testing were used here. One cylinder was taken of the first grout pour of July 4, 1980, no mortar cylinder was taken. The cylinder was tested at 7 days.

Figure 3.23 Tested Wall: In Process
of Demolition and Removal

Figure 3.24 Reinforcement at Demolished
Wall

Figure 3.25 Block Prism Construction

Eight cylinders were taken from the first grout pour of July 11, 1980 for fill of the wall from the 1200mm to 2400mm height. Four of the cylinders were tested in axial compression to determine failure load in this mode and four were tested by the split cylinder test to give an indication of the tensile strength of the grout. Five cylinders were taken from the second grout pour on July 11, 1980, 5 cylinders were taken of the mortar fill.

The cylinders were manually vibrated with a rod and filled in 2 lifts. The cylinders were covered with polyethelene sheet for 10 days to allow for proper curing.

(iii) 2" Mortar Cubes

A representative sample of three 2" mortar cubes were taken of the low slump mortar used in the wall joints for each 1200mm height of walls constructed. Three mortar cubes were also taken of the mortar fill used, taken from the mortar used to fill the block prisms.

The cubes were covered with polyethelene for ten days.

(iv) 4" x 4" x 8" Absorptive Prisms

5 specimens each were made of the grout and mortar fill used in the 1200mm to 2400mm heights of wall. These specimens were built as indicated in Figure 3.26 on July 11, 1980, approximately 2 hours before being filled. The prisms were set on an oiled plywood sheet, the block used was from the same shipment received for the wall construction. Absorptive paper towelling was taped to the form face of the blocks to prevent bond but allow absorption of water into the block. The fill material was poured in two lifts and

Figure 3.26 4" x 4" x 8" Absorptive Prism
Construction

manually rodded. After the top was trowelled level, the prisms were covered with wet towelling and left to cure for two days, after which time they were covered for a further eight days with polyethelene sheet.

(v) Reinforcing Steel

As noted previously, representative samples of the 10M, 15M and 20M bars were taken prior to construction of the walls. On demolition of the walls, samples were again taken of each reinforcing bar for tensile testing.

3.4.5 Instrumentation and Testing of Associated Specimens

All the associated specimens were tested at Carleton University in the axial compression mode except for the 4 split cylinder tests and the tensile testing of the reinforcement. The grout filled block prisms were tested on August 11, 1980, the mortar filled and hollow block prisms on August 28, 1980, the mortar cubes and absorptive prisms and concrete cylinders on August 7, 1980. The tensile testing of the steel reinforcing taken from the test walls occurred on August 5, 1980.

The block prisms were capped with a flexible test material, and loaded in compression to failure. 8" Demec strain gauges were used to measure the strains at the front and back of the prisms across the mortar joint. The actual dimensions of the block were accurately measured and the axial stress calculated. The block prisms were loaded in increments of 10 kips, to failure (Table 4.3, 4.4, 4.5 and Figures 4.1, 4.2, 4.3 show the results).

The 2" mortar cubes, 4" x 4" x 8" absorptive prisms, and fifteen cylinders were loaded to failure in compression, the results are shown in Tables 4.1 and 4.2. Load application was generally at a rate of 10,000 lbs per minute for the specimens. The results of 4 cylinders loaded to failure by the split cylinder test are shown in Table 4.2.

The results of the tensile tests of the reinforcement removed from the walls
is shown in Table 3.4.

CHAPTER 4

TEST RESULTS AND DISCUSSIONS

4.1 ASSOCIATED SPECIMENS

A summary of ultimate strengths of the block prisms, the 6" diameter cylinders, the 4" x 4" x 8" absorptive prisms, the 2" mortar cubes, the reinforcing steel are given in Tables 3.4, 4.1 to 4.5 and Figures 4.1, 4.2 and 4.3.

4.1.1 Mortar Cubes

The average value of 9 mortar cubes taken of the joint mortar was 2952 psi. The average value of the joint mortar taken in the critical 1.2m to 2.4m height was 1944 psi. The average value of the mortar fill was 2759 psi. These values are above the specified strength for type S mortar of 1800 psi. The differences in the ultimate strengths of these specimens result from the use of small batches of the material mixed at the site by the masons.

4.1.2 Cylinders - 6" Diameter in Compression

The average value of all 9 grout cylinders tested in compression at 25 days or older was 2663 psi. The average value of the grout fill in 1.2m to 2.4m height was 2540 psi, the average value of the 5 mortar fill cylinders was 3107 psi. All cylinder strengths are in excess of the 1500 psi strength that was specified.

The one cylinder tested in axial compression at 7 days from the first grout pour was to provide a degree of verification of the plant mix supplied and is estimated to have a 28 day strength of 1400 psi since the seven day strength was 1043 psi. On subsequent discussions with the supplier, it was found that the plant mix supplied had unfortunately been altered from that specified and delivered for the first batch by adding more Portland Cement at the plant.

4.1.3 Cylinders - 6" Diameter in Tension

Split cylinders of the grout were also investigated in order to get some indication of the material strength in tension. The strength can be computed using the following equation:

$$f_{ct} = \frac{2P}{\pi DL}$$

where f_{ct} = splitting tensile strength of the material

P = ultimate load

D = diameter of cylinder

L = length of cylinder

f_{ct} (or f_r the modulus of rupture of concrete) is equal to $k \sqrt{f'_c}$ where f'_c is the ultimate axial compression stress and k is equal to 7.5 for concrete.

Using the data obtained from the split cylinder and the compressive specimens in the 1.2m to 2.4m height of the wall where $f_{ct} = 395$ psi and $f'_c = 2540$ psi, we obtain k equal to 7.84. This value for k approaches the values obtained by Baynit (10).

4.1.4 4" x 4" x 8" Absorptive Prisms (Compression)

The average ultimate strength of the 5 grout fill prisms was 3792 psi. The average ultimate strength of the 5 mortar fill prisms was 3690 psi.

The ratio of ultimate strength of the average value of 2663 psi for the 9 cylinders to the average value for the absorptive prisms of 3792 psi is 0.70. The

specified test value of grout fill of 1500 psi was intended to achieve an in wall strength of 2100 psi (14.5 MPa) to correspond to the block net area strength.

4.1.5 Masonry Prisms

The average ultimate strength compressive stress for the 5 grout filled block prisms was 1367 psi and the average ultimate strain was 79.1×10^{-5} inches per inch.

The average ultimate compressive stress for the 5 mortar filled block prisms was 2091 psi and the average ultimate strain was 123×10^{-5} .

The average ultimate compressive strain for the 5 unfilled block prisms was 2280 psi and the average ultimate strain was 148×10^{-5} .

The present masonry codes indicate that the compressive strength of a block wall varies as the block strength itself for mortar joint strengths of 1800 psi (type S) and 2500 psi (type M) mortars.

No parameters to vary the ultimate compressive strength of the wall construction as a function of the fill material are indicated in the codes. The codes (6, 18) indicate that the ratio of ultimate compressive strength of the filled block construction (f'_m) to the ratio of block ultimate strength, is approximately 0.65.

Using the average grout filled prism strength of 1367 psi, and the average unfilled prisms strength of 2280 psi, we obtain a ratio of 0.60. Similarly with the mortar filled prisms we obtain a ratio of 0.92. Using the specified strength of the block of 2100 psi, the ratios are 0.65 and 1.0 for the grout and mortar filled cores respectively.

TABLE 4.1 MORTAR CUBES AND ABSORPTIVE SPECIMEN RESULTS					
SPECIMEN NO. AND TYPE	LOCATION TAKEN	TYPE OF MATERIAL	AGE AT TEST (days)	FAILURE STRESS (psi)	AVERAGE STRESS (psi)
2" MORTAR CUBES SPECIMENS					
1 2 3	WALLS to 1.2m HEIGHT	JOINT MORTAR " "	25 " "	2525 2313 2433	2424
1 2 3	WALLS 1.2 to 2.4m HEIGHT	JOINT MORTAR " "	25 " "	1675 2013 2145	1944
1 2 3	WALLS 2.4 to 3.6m HEIGHT	JOINT MORTAR " "	25 " "	4575 4413 4475	4488
1 2 3	WALLS	MORTAR FILL	25	2588 3088 2600	2759
4" x 4" x 8" ABSORPTIVE PRISM SPECIMENS					
1 2 3 4 5	WALLS	MORTAR FILL " " " "	25 " " " "	3479 3673 3738 3840 3721	3690
1 2 3 4 5	WALLS	GROUT FILL " " " "	25 " " " "	3538 3832 3723 4000 3866	3792

TABLE 4.2 6" DIAMETER CYLINDER SPECIMENS

CYL. No.	LOCATION TAKEN	TYPE OF MATERIAL	AGE AT TEST	TYPE OF TEST	FAILURE STRESS (psi)	AVERAGE STRESS (psi)
1	WALL	GROUT FILL	25	AXIAL COMPR.	2766	2540
2	1.2 to 2.4	"	"	"	2320	
3	HEIGHT	"	"	"	2603	
4	July 11, 1980	"	"	"	2469	
1	WALL	GROUT FILL	25	AXIAL COMPR.	2815	2761
2	2.4 to 3.6	"	"	"	2801	
3	HEIGHT	"	"	"	2596	
4	July 11, 1980	"	"	"	2808	
5		"	"	"	2787	
1	WALL	MORTAR FILL	25	AXIAL COMPR.	2957	3107
2	1.2 to 2.4	"	"	"	3057	
3	HEIGHT	"	"	"	3176	
4	July 11, 1980	"	"	"	3105	
5		"	"	"	3240	
1	WALL to 1.2	GROUT FILL	7	AXIAL COMPR.	1043	1043 (1)
	HEIGHT					
	July 4, 1980					
1	WALL	GROUT FILL	25	SPLIT CYLINDER TEST	395	395
2	1.2 to 2.4	"	"		425	
3	HEIGHT	"	"		385	
4	July 11, 1980	"	"		365	

(1) = ESTIMATED 28 DAY STRENGTH OF THIS TEST IS 1400psi

TABLE 4.3 BLOCK PRISM SPECIMENS: GROUT FILLED CORFS

SPECIMEN No.	1		2		3		4		5	
	σ	ϵ	σ	ϵ	b	ϵ	σ	ϵ	σ	ϵ
0	0	0	0	0	0	0	0	0	0	0
10	-	-	184	5.6	181	5.1	181	4.5	184	6.1
20	362	9.6	368	11.6	362	10.1	362	11.1	368	12.1
30	543	16.7	552	17.7	543	14.6	543	17.2	552	18.2
40	724	25.8	736	24.2	724	22.2	724	25.3	736	23.7
50	904	41.9	920	28.3	904	27.3	904	32.8	920	29.8
60	1085	56.1	1104	37.9	1085	36.4	1085	38.9	1104	35.9
70	1266	86.9	1288	62.1	1266	77.3	1266	51.5	1288	46.0
75	-	-	1381	95.4	-	-	1357	70.2	1381	50.5
80	-	5	-	-	-	-	-	-	1473	57.1
85	-	-	-	-	-	-	-	-	1565	65.7
									-	-

σ = AXIAL STRESS IN psi
 ϵ = AVERAGE OF FRONT AND BACK STRAIN GAUGES $\times 10^{-5}$ in/in
 AVERAGE ULTIMATE STRESS: 1367 psi
 AVERAGE ULTIMATE STRAIN: 79.1×10^{-5} in/in

TABLE 4.4
BLOCK PRISM SPECIMENS: MORTAR FILLED CORES

SPECIMEN No.	1		2		3		4		5	
	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ
0	0	0	0	0	0	0	0	0	0	0
20	362	14.6	368	11.6	362	11.6	362	13.1	365	12.6
40	724	25.8	724	26.3	724	27.8	724	27.8	730	29.8
60	1085	40.4	1085	41.9	1085	44.4	1085	45.5	1095	52.0
80	1447	60.1	1473	59.6	1447	65.1	1447	65.7	1460	78.3
90	1628	71.2	1657	70.2	1628	80.3	1628	76.3	1642	101.0
100	1809	83.8	1841	82.3	1809	95.4	1809	91.9	1825	140.9
105	1899	90.4	1933	88.4	1899	105.0	1899	102.5	1916	-
110	1990	96.5	2025	99.0	1990	125.7	1990	118.7	-	-
115	2080	102.5	-	-	2080	-	2080	128.9	-	-
120	2171	110.1	-	-	-	-	2171	-	-	-
125	2261	121.7	-	-	-	-	-	-	-	-

σ = AXIAL STRESS IN psi AVERAGE ULTIMATE STRESS: 2091 psi
 ϵ = AVERAGE OF FRONT AND BACK AVERAGE ULTIMATE STRAIN: 123×10^{-5} in/in
 STRAIN GAUGES $\times 10^{-5}$ in/in

TABLE 4.5 BLOCK PRISM SPECIMENS: UNFILLED CORES

SPECIMEN No. APPLIED LOAD (KIPS)	1		2		3		4		5	
	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ	σ	ϵ
0	0	0	0	0	0	0	0	0	0	0
20	678	31.3	695	29.8	620	29.8	605	14.1	673	34.3
40	1356	72.2	1390	65.1	1239	64.1	1210	44.9	1345	70.7
50	-	-	1738	88.7	1549	88.4	1513	65.1	1682	89.9
55	-	-	1912	102.0	1704	100.5	1664	75.8	1850	110.6
60	2034	131.8	2086	116.7	1859	113.6	1815	91.4	2018	122.2
65	2221	-	2259	142.4	2014	129.3	1966	110.1	2186	143.9
70	-	-	2433	177.3	2122	-	2118	126.8	2354	173.7
75	-	-	-	-	-	-	2269	-	-	-

σ = AXIAL STRESS IN psi
 ϵ = AVERAGE OF FRONT AND BACK STRAIN GAUGES $\times 10^{-5}$ in/in
 AVERAGE ULTIMATE STRESS: 2280 psi
 AVERAGE ULTIMATE STRAIN: 148×10^{-5} in/in

Figure 3.1 Grout Filled Block Prisms: Stress Strain Curve

AVERAGE OF FRONT & BACK STRAIN GAUGES $\times 10^{-3}$

Figure 4.2 Mortar Filled Block Prisms. Stress - Strain

It can be seen from these block prism results that based on ultimate compressive stress, the mortar filled prisms are the more desirable material to use, since the strength of the block prism approaches the strength of the block itself. Similar results can be seen for the ultimate strain of the unfilled block prisms, and that the grout filled prisms are less ductile than the mortar filled block prisms.

ISSACS (34) proposed a formula for determining the ultimate compressive strength of grout filled masonry as follows.

$$f'_m = 0.63 (K_1 f'_b + K_2 f'_c)$$

where f'_m = ultimate compressive strength of the filled block

0.63 = a parameter based on the geometry of the block

f'_b = compressive strength of the hollow blocks, net area

f'_c = compressive strength of the grout

$K_1 = A_c/A_n$

$K_2 = A_b/A_n$

A_b = total net area of the block

A_c = total net area of the grout

$A_n = A_b + A_c$

For our case of grout filled cores, $f'_b = 2280$ psi (unfilled block prism specimens), $f'_c = 2663$ psi (cylinder tests), $A_b = 30.7$ in² and $A_c = 24.3$ in². A_b and A_c are average values of the block prisms tested.

$$\begin{aligned} f'_m &= 0.63 \left(\frac{30.7}{55.0} \times 2280 + \frac{24.3}{55.0} \times 2663 \right) \\ &= 0.63 (1272.7 + 1176.6) \\ &= 1543 \text{ psi} \end{aligned}$$

For the mortar filled cores using $f'_c = 3107$ psi (cylinder)

$$f'_m = 1667 \text{ psi}$$

Using the 4" x 4" x 8" absorptive specimen average ultimate strengths for grout and mortar and the same f'_b as above we obtain

$$f'_m = 1857 \text{ psi} \quad \text{for grout}$$

$$\text{and} \quad f'_m = 1828 \text{ psi} \quad \text{for mortar}$$

Hamid & Drysdale (23) (29) tested 43 half block, 3 course high prisms and similarly proposed a formula for the prediction of grout filled block strengths. Based on their formula, using the concrete cylinder strengths for the fill, and the block strength of 2280 psi, the prism strengths calculated are

$$f'_m \approx 1700 \text{ psi for the grout filled}$$

$$\text{and} \quad f'_m \approx 1900 \text{ psi for the mortar filled prisms.}$$

Using the 4" x 4" x 8" absorptive strengths for fill material, the prism strengths calculated using the Hamid and Drysdale formula are

$$f'_m \approx 1950 \text{ psi for the grout and mortar filled prisms.}$$

It can be seen that for both the Issacs and Drysdale, Hamid proposals which are similar, the ultimate strength of the grout filled prisms is approached by using the grout cylinder strengths and for the mortar filled prisms by using the absorptive prism strengths. In all cases, however, the results could not be predicted accurately.

Therefore, the ultimate compression strengths of the walls (f'_m) are assumed to be as observed from the block prism specimen test results noted in Tables 4.3, 4.4, 4.5

9.4 MPa (1365) psi for grout filled walls,
14.4 MPa (2090) psi for mortar filled walls,
and 15.7 MPa (2280) psi for the unfilled walls.

The average ultimate strains indicated at failure for these block prisms were 0.00079 in/in for the grout filled, 0.00123 in/in for the mortar filled and 0.00148 for the unfilled cores. The maximum strains recorded on any face of these prisms were 0.0011 for the grout filled, 0.00175 for the mortar filled and 0.00212 for the unfilled block prisms. The maximum strain at failure for reinforced concrete sections is assumed to be 0.003 in/in.

Drysdale (23) points out that if the fill material is not of the same relative strength range as the block itself, the modulus of elasticity and rigidity (which vary as the square root strength of the material) will have different values for the block and fill. If the grout strength is much higher than the block strength, the resultant lateral strains will be incompatible, creating tension stresses in the weaker block shells, leaving the smaller fill core to take the load, and the filled prisms will fail at a lower stress level than the ungrouted prisms. In our case, the grout fill in place as indicated by the absorptive prisms was much stronger than the block itself and the filled prisms failed at a lower stress level than the unfilled block prisms. In fact, the average failure loads for the grout filled prism was 76 kips on an area of 55 square inches (1367 psi), with a maximum load value of 88.5 kips, a minimum of 70 kips. The average maximum load on the unfilled prism was 70 kips on an area of 30.7 square inches (2280 psi), with a maximum value of 75 kips, a minimum of 65

2

kips. It can be seen that the differences of average axial compressive load capacity is within 10 percent for these prisms whereas the ratio of net areas of unfilled to filled block prisms is 55.8 percent.

The mortar fill again was of a much higher strength than the block. However, the modulus of elasticity and rigidity of mortar versus grout of similar strengths is lower, by approximately 25 percent (50). It would seem that in our case, there was appreciable strain compatibility in the mortar filled prisms, since the ultimate stress of these prisms approached the unfilled block prisms. For the mortar filled prisms, the average maximum load was 115 kips on a net area of 55 square inches (2091 psi), with a maximum of 125 kips, a minimum of 105 kips. The observations of the maximum average stress values show that the grout filled prisms failed at much lower stress levels than the unfilled block prisms. This is probably due to incompatibility of strains between the higher strength grout fill and the block as predicted by the Drysdale (23) paper. The unfilled block prisms failed in the range of the specified block strength.

The failure of the filled block prisms were generally initiated by vertical splitting of the block face. In the mortar filled prisms failure was initiated in 2 of the specimens by diagonal splitting of the block face and only one by crushing of the top block corner. The vertical splitting indicated the differences of the strains of the fill and block materials, especially in the grout filled prisms.

In the unfilled block prisms, failure was initiated by crushing of the top block in two specimens and by total collapse in the three others. This failure pattern is an indication of the strain compatibility of the joint mortar and the block.

4.1.6 Reinforcing Steel Specimens

Table 3.4 indicates the yield, ultimate stress levels of the reinforcing bars taken from the actual walls on demolition, as well as the percentage elongation on

failure. The tests were made using two foot long coupons cut from the bars. The strains were measured with 8" Deined gauges, and tested in tension on the Tinius Olsen machine at Carleton University.

The results indicate that the reinforcement strains were linear to yield with a definite flat yield plateau prior to strain hardening, and the percentage elongation was as expected for the reinforcement.

4.2 WALL SPECIMENS

4.2.1 General Observations

The following results were obtained from the wall specimens at incremental load levels:

- (i) Deflections by 3 dial gauge readings
- (ii) Steel strains in the reinforcement
- (iii) Compressive strain readings on the block
- (iv) Tensile strain readings on the block.

Photographs were also taken of the various walls at different stages of loading. A record was kept of crack locations in the walls. Indications of mortar joint crushing were recorded where observed. An example of the records taken during wall testing are shown in Table 4.6 and Figure 4.4.

From the data recorded, the following were obtained:

- (i) Steel stress in the bars
- (ii) Concrete strains on the compression face of the walls
- (iii) Wall deflections
- (iv) Crack widths on the tension face of the walls.

This data was used in combination with the results of the tests of the associated specimens to determine:

TABLE: 4.6 EXAMPLE OF LOADS AND MEASUREMENTS TAKEN DURING TESTS WALL No. 9 DATE AUG. 12/1980 START 4:00 P.M. FINISH 5:00 P.M.																
LOAD NUMBER	DEMEC GAUGE No.						DIAL GAUGE No.						LOAD CELL No.			
	1	2	3	4	5	6	1	2	3	4	5	6	1	2	3	4
1	1018	900	1022	1017	895	1017	11956	12285	12207				0	0	0	0
2	-	-	-	-	-	-	11930	12259	12186				151	102	-31	42
3	1010	898	1018	1010	896	1020	11722	12044	12005				315	266	152	209
4	-	-	-	-	-	-	11020	11280	11332				452	400	309	349
5	980	889	986	989	894	1140	10296	10467	10623				600	543	464	503
6	-	-	-	-	-	-	9691	9803	10037				750	668	573	630
7	954	888	958	960	881	1240	8902	8918	9245				910	803	773	773
8	-	-	-	-	-	-	8360	8312	8712				1000	890	878	855
9	925	883	929	937	873	1296	6109	5905	6819				1030	964	1027	924
10	-	-	-	-	-	-	5692	5401	6371				1020	826	934	971
11	-	-	-	-	-	-	3240	2544	3932				1050	898	1086	965
12	-	-	-	-	-	-	0978	0007	1823				1130	927	1115	-
13	-	-	-	-	-	-	-	-	-				1140	890	1110	-
ADJUST- MENT	MULTIPLY BY						NIL						NIL			
UNITS OF MEASURE- MENT	1.01	2.51	1.01	1.01	2.51	1.01	10 ⁻⁵ in/in						10 ⁻² mm			
													lbs			

TABLE 4.6 cont.

LOAD NUMBER	STRAIN GAUGE No.				HYDRAULIC PRESSURE GAUGE	REMARKS
	1	2	3	4		
1	0	0	0	0	0	HORIZONTAL CRACK: JOINTS 8 & 12 CRACKS AT JOINTS 6,7,8,10,11,12,13 CRACKS AT JOINTS 4 to 14 INCLUSIVE VERTICAL CRACK BETWEEN JOINTS 9, 10. HORIZONTAL CRACK AT JOINT 15 HORIZONTAL CRACK AT JOINTS 3 to 15
2	0	4	0	4	25	
3	14	29	86	93	52	
4	521	736	405	395	65	
5	796	1113	743	789	93	
6	1026	1427	999	1110	110	LARGE BANG. RAPID CHANGE IN BAR STRAINS, BEARING PLATES GOOD LARGEST JOINT CRACKS AT JOINT 8 & 10
7	1318	1804	1330	1527	128	
8	1620	1909	1551	1754	140	
9	3102	1979	1921	1944	152	
10	2860	1963	6582	3146	137	
11	4040	2025	8593	784	160	MORTAR CRUSHING COMPRESSION FACE SUDDEN FAILURE, POSSIBLE BOND FAILURE
12	45420	2056	10102	737	-	
13	45420	2482	6582	377	-	
	DIVIDE BY					ADJUSTMENT FACTOR
	0.653	0.656	0.652	0.652		
	10^{-6} in/in				psi	UNITS OF MEASUREMENT OBTAINED

NOTES: RE: TABLE 4.6

See Fig. 3.17 for location of measurements on wall:
initial load number 1 represents zero loads; use adjustment factor to convert to true units of measure; subtract Demec and dial gauge readings from initial readings to obtain strains and deflections.

NOTES: RE: WALL No. 9

- some rotation of block wall at base, beginning at load level 4; measured maximum deflection $6 \frac{1}{2}$ ' (165mm) total at mid-height with no appreciable increase in load above load number 13
- Appears to be a brittle failure, possibly a bond failure but at stress levels beyond yield of reinforcement
- On demolition of wall, bars in lap area straight, reinforcement lug marks slightly crushed, reinforcement loose in middle joint area. See Fig. 4.4 for block crack pattern in this area at complete failure load; photographs taken; strain gauge locations perpendicular to wall width; measured effective depth $5 \frac{3}{4}$ " (95mm).

Figure 4.4 Crack Pattern at Tension Face Wall 9

- (i) Flexural moments of the walls
- (ii) Curvature of the walls,

and to plot moment-deflection and moment-curvature graphs for these walls.

Analysis of these curves and observations during testing indicate that for the bond lengths used, no bars failed in bond prior to initial yield of the steel.

4.2.2 Calculations and Classification of Test Data

The results of the test data were assimilated and a computer program developed to obtain calculated moments, steel forces, deflections and curvatures, as well as to estimate the neutral axis and compression stresses in the walls. For a brief summary of the program development and results see Appendix C.

On review of the results of the test data and the developed computer program, the relevant data to be used in this paper was extracted and condensed in Tables 4.7 to 4.24 for each level of applied load increment at each of the nineteen walls.

The "Moments" listed in the tables were calculated at mid-height of the walls using the resultant applied forces derived from the load cell readings taken during the tests.

The "Deflections" listed in the tables were derived from the readings taken from the dial gauge placed at the centerline of the wall at mid-height of the wall.

The "Steel Forces" listed in the tables were calculated from the steel strain measurements recorded during testing to obtain the stress levels in the reinforcement at the applied load level. There were two strain gauges on each reinforcing bar. The stresses calculated at each strain gauge were limited to a maximum value equal to the yield stress of the reinforcing bar. The two calculated stresses at each bar were averaged and multiplied by the bar area to obtain the force in each reinforcing bar. These calculated forces at the strain gauge locations are listed in the

tables for the top and bottom lapped bars. Where double lapped bar splices were used, the calculated forces are averaged for the two bars at the top and the bottom. Where single unlapped bars were placed in the walls, only two strain gauges were placed on the reinforcing at mid-height of the wall. The steel forces listed in the tables refer to top and bottom bars similarly to the data noted for lapped splices, but the forces are equal in magnitude for the two listed locations.

The "Curvatures" listed in the tables were derived from dial gauge readings as well as from block joint strain readings. The curvatures listed for the dial gauge readings were calculated at mid-height of the walls using the three dial gauge readings and the formula $z = ay^2 + by + c$ to approximate a parabolic curve for the deflected shape of the wall. a , b and c are derived from the dial gauge readings, z is the vertical distance from mid-height of the wall to a point on the curve, y is the horizontal distance to the same point. Differentiating z with respect to y twice, we obtain ϕ , the curvature in terms of the dial gauge measurements. The curvatures listed for the block joint strain readings were derived using the joint strain calculated from the average of the three 8" Demec readings taken on the compression face of the wall and the average of all the steel strain readings. Since the effective depth to the reinforcement was known, this curvature could be calculated. Demec strain readings were not taken at every applied load level during the tests.

The "Neutral Axis" values listed in the tables were derived using the average of the three 8" Demec readings taken on the compression side of the wall, and the average of the steel strain gauge readings at the top and bottom reinforcing bars. Since the effective depth is known the location of the neutral axis can be calculated.

The "Conc. Stress", that is, Calculated Concrete Stresses listed in the tables were derived from the values of the steel bar forces and the neutral axis described

above. For load levels below which the average stress in any reinforcing bar had not reached yield stress, a triangular stress block was used to determine the moment lever arm. Once yield in any bar was reached, the Whitney stress block (54) (distance from compressive face to neutral axis times .85) was used to determine the moment lever arm. The steel force multiplied by the lever arm is the resisting moment. No capacity reduction factor was included in the calculations.

Table 4.7, for Wall 1 and 2, does not include values for "Steel Force", "Curvature", "Neutral Axis" and "Conc. Stress" since these walls were not reinforced. The maximum "Concrete Stress" noted in this table was derived from the maximum value noted for "Moment" for these walls divided by the section modulus of the masonry wall section.

TABLE 4.7 CALCULATED DATA FOR WALLS 1 & 2

WALL 1		WALL 2	
MOMENT (kN.m) MID-HEIGHT	DEFLECTION (cm) MID-HEIGHT	MOMENT (kN.m) MID-HEIGHT	DEFLECTION (cm) MID-HEIGHT
0	0	0	0
0.59	0.27	0.67	0.13
0.72	0.84	1.43	0.58
0.74	2.89	-	1.64
0.73	4.36	0.52	5.29
NEUTRAL AXIS = 95mm MAXIMUM CONCRETE STRESS = 0.07 MPa		NEUTRAL AXIS = 95mm MAXIMUM CONCRETE STRESS = 0.12 MPa	

TABLE 4.8 CALCULATED DATA FOR WALL 3

MOMENT (KN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0.0	0.0	0	0	0	-	-	-	-	-
2.08	0.09	1.9	1.9	0.0007	-	-	-	-	-
4.12	0.48	21.3	21.3	0.0045	-	-	-	-	-
6.17	0.95	39.1	39.1	0.0086	0.0010	41.6	41.6	3.08	3.08
8.31	1.46	57.4	57.4	0.0126	-	-	-	-	-
10.51	2.00	72.1	72.1	0.0171	-	-	-	-	-
12.51	2.54	89.7	89.7	0.0213	0.0222	39.1	39.1	7.52	7.52
14.51	4.41	100.0	100.0	0.0375	0.0288	43.8	43.8	4.40	4.40
15.08	7.00	100.0	100.0	0.0682	-	-	-	-	-
15.84	8.77	100.0	100.0	0.0855	-	-	-	-	-
15.99	10.28	100.0	100.0	0.1014	-	-	-	-	-
15.99	11.39	100.0	100.0	0.1108	-	-	-	-	-
18.22	11.99	100.0	100.0	-	-	-	-	-	-
	15.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.9 CALCULATED DATA FOR WALL 4

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONS. STRESS (MPa)	
		BOT	TOP	DG	BL.JT.	BOT	TOP	BOT	TOP
0.0	0	0	0	0	-	-	-	-	-
2.00	0.08	0.9	0.9	0.0005	-	-	-	-	-
3.33	0.64	27.0	27.0	0.0065	0.0095	24.0	24.0	3.69	3.69
4.89	1.33	45.2	45.2	0.0119	0.0223	23.6	23.6	8.93	8.93
6.52	2.04	64.1	64.1	0.0176	-	-	-	-	-
8.00	2.71	80.8	80.8	0.0227	0.0342	22.8	22.8	8.41	8.41
9.83	3.46	99.2	99.2	0.0283	-	-	-	-	-
11.27	5.95	100.0	100.0	0.0546	-	-	-	-	-
12.06	10.99	100.0	100.0	0.1047	-	-	-	-	-
11.72	11.99	100.0	100.0	-	-	-	-	-	-
11.93	11.99	100.0	100.0	-	-	-	-	-	-
	32.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
 TOP - TOP REINFORCEMENT
 DG - DERIVED FROM DIAL GAUGE READINGS
 BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
 CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.10 CALCULATED DATA FOR WALL 5

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)	NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP		BL.JT.	BOT	TOP	BOT
0	0	0	0	0	-	-	-	-
2.81	.28	21.0	21.0	0.0031	-	-	-	-
6.30	1.64	64.9	64.9	0.0145	0.0164	29.5	29.5	7.2
8.04	2.19	84.0	84.0	0.0188	-	-	-	-
10.53	3.09	112.4	112.4	0.0260	0.0284	29.6	29.6	12.5
12.71	3.96	140.3	140.3	0.0329	-	-	-	-
13.78	4.44	147.0	147.0	0.0368	0.0401	29.8	29.8	9.5
14.87	5.38	147.0	147.0	0.0477	-	-	-	-
15.84	-	147.0	147.0	-	-	-	-	-
16.04	12.55	-	-	-	-	-	-	-
	15.0 max							

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DG - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.11 CALCULATED DATA FOR WALL 6

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
3.20	.24	19.1	19.1	0.0021	-	-	-	-	-
6.35	.70	46.8	46.8	0.0061	0.0055	55.4	55.4	2.77	2.77
9.60	1.15	76.9	76.9	0.0100	-	-	-	-	-
12.83	1.62	106.8	106.8	0.0139	0.0122	52.5	52.5	6.68	6.68
15.95	2.10	137.0	137.0	0.0179	-	-	-	-	-
18.98	2.59	166.2	166.2	0.0218	0.0190	52.3	52.3	10.43	10.43
22.13	3.12	198.4	198.4	0.0262	-	-	-	-	-
25.00	4.58	200.0	200.0	0.0413	0.0285	58.3	58.3	6.63	6.63
26.32	7.54	200.0	200.0	0.0769	-	-	-	-	-
27.17	10.43	200.0	200.0	0.0985	-	-	-	-	-
28.02	11.81	200.0	200.0	0.1104	-	-	-	-	-
31.94	11.81	200.0	200.0	-	-	-	-	-	-
	16.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.12 CALCULATED DATA FOR WALL 7

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)	NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP		BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-
0.92	.03	.05	.04	0.0002	-	-	-	-
1.81	.08	.29	.09	0.0004	-	-	-	-
2.91	.21	.87	.33	0.0015	68.3	82.7	.04	.01
3.23	.65	18.1	.71	0.0053	-	-	-	-
3.80	1.24	28.9	24.4	0.0117	17.9	17.5	4.4	4.6
4.51	1.71	29.1	30.9	0.0149	-	-	-	-
5.23	2.08	34.4	36.6	0.0182	16.7	15.9	6.8	7.6
5.94	2.35	38.4	40.6	0.0206	-	-	-	-
6.75	2.81	44.9	46.8	0.0247	16.6	16.0	8.9	9.6
7.45	3.31	51.6	53.7	0.0291	-	-	-	-
7.79	3.87	56.5	56.5	0.0319	-	-	-	-
7.44	4.76	56.5	56.5	0.0382	-	-	-	-
7.48	8.45	56.5	56.5	0.0834	-	-	-	-
8.53	11.74	56.5	56.5	0.1171	-	-	-	-
	250 max.							

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE: 4.13 CALCULATED DATA FOR WALL 8

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
1.12	0.04	0.4	0.3	0.0001	-	-	-	-	-
2.70	0.30	2.8	21.1	0.0027	0.0047	64.2	20.4	0.1	3.4
4.37	1.05	24.7	39.8	0.0080	-	-	-	-	-
5.96	1.79	45.5	58.3	0.0139	0.0181	25.7	21.3	5.8	9.0
7.57	2.59	66.6	77.6	0.0201	-	-	-	-	-
9.18	3.40	87.7	97.3	0.0261	0.0324	24.6	22.8	6.9	8.2
10.29	4.33	100.0	100.0	0.0347	0.0620	22.6	9.4	8.5	20.5
10.90	5.81	100.0	100.0	0.0491	0.0634	23.2	10.9	8.3	17.7
11.16	7.18	100.0	100.0	0.0625	0.0993	9.7	11.0	19.9	17.5
11.92	12.31	100.0	100.0	-	-	-	-	-	-
	18.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.14 CALCULATED DATA FOR WALL 9

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0.0	-	-	-	-	-
0.72	0.03	0.1	0.1	0.0002	-	-	-	-	-
2.55	0.24	5.5	1.3	0.0018	0.0016	30.2	62.4	0.6	0.1
4.10	1.01	24.6	38.7	0.0077	-	-	-	-	-
5.72	1.82	47.1	58.7	0.0152	0.0175	21.4	18.0	7.2	10.7
7.11	2.48	64.9	75.5	0.0205	-	-	-	-	-
8.84	3.37	87.9	96.1	0.0278	0.0308	21.0	19.6	8.1	9.5
9.82	3.97	100.0	100.0	0.0331	-	-	-	-	-
10.70	6.38	100.0	100.0	0.0591	0.0463	22.0	17.7	8.8	10.9
10.17	6.88	100.0	100.0	0.0646	-	-	-	-	-
10.84	9.74	100.0	100.0	0.0964	-	-	-	-	-
11.29	12.28	100.0	100.0	0.1238	-	-	-	-	-
11.23	12.29	100.0	100.0	-	-	-	-	-	-
	16.5 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.15 CALCULATED DATA FOR WALL 10

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m) DC BL.JT.	NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP		BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-
1.13	0.04	.2	.2	Q0002	-	-	-	-
2.72	0.28	16.4	16.6	Q0018	17.8	17.8	3.1	3.1
4.62	0.96	36.3	38.9	Q0064	-	-	-	-
6.06	1.48	51.0	53.9	Q0104	21.1	20.2	8.0	8.8
7.65	2.11	68.1	71.3	Q0143	-	-	-	-
9.04	2.67	83.0	86.5	Q0181	20.8	20.2	13.1	14.1
10.10	3.15	95.6	98.5	Q0212	-	-	-	-
11.31	3.81	100.0	100.0	Q0235	20.3	20.5	9.5	9.4
11.97	7.16	100.0	80.7	Q0909	-	-	-	-
12.26	8.82	100.0	89.0	Q1523	14.5	27.3	13.3	6.3
12.54	11.53	100.0	100.0	Q0767	-	-	-	-
12.77	11.77	100.0	100.0	-	-	-	-	-
12.94	11.77	100.0	100.0	-	-	-	-	-
12.93	11.77	100.0	100.0	-	-	-	-	-
	29.0 max							

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.16 CALCULATED DATA FOR WALL 11

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL-JT	BOT	TOP	BOT	TOP
0.0	10	0	0	0	0			-	-
1.15	0.04	0.3	0.3	0.0003	-	-	-	-	-
2.73	0.19	0.9	1.7	0.0016	-	-	-	-	-
4.26	0.94	30.7	38.3	0.0067	0.0007	55.0	40.0	0.05	0.14
5.78	1.63	48.2	55.6	0.0111	-	-	-	-	-
7.40	2.40	67.1	75.6	0.0164	0.0176	22.5	20.1	7.0	9.1
9.09	3.24	87.4	96.0	0.0219	-	-	-	-	-
10.28	3.90	100.0	100.0	0.0260	0.0306	20.8	19.3	8.1	9.6
10.86	5.53	100.0	100.0	0.0350	-	-	-	-	-
11.45	10.77	100.0	100.0	0.0568	-	-	-	-	-
11.57	11.82	100.0	100.0	-	-	-	-	-	-
11.61	11.82	-	-	-	-	-	-	-	-
	15.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL-JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.17 CALCULATED DATA FOR WALL 12

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
2.24	.09	0.4	0.9	0.0008	-	-	-	-	-
4.35	.63	29.8	37.1	0.0040	0.0071	17.4	14.5	5.6	8.4
7.19	1.40	62.7	66.8	0.0088	-	-	-	-	-
9.86	2.14	90.9	94.3	0.0061	0.0206	20.4	19.8	14.6	15.6
13.01	3.12	125.6	127.0	0.0197	-	-	-	-	-
15.45	4.03	147.0	147.0	0.0257	0.0357	20.9	21.2	13.6	13.4
16.26	5.13	147.0	147.0	0.0320	-	-	-	-	-
17.00	6.81	147.0	161.1	0.0411	0.0368	23.9	32.2	11.9	7.0
17.53	9.37	147.0	125.1	0.0620	-	-	-	-	-
18.22	-	86.4	87.6	-	-	-	-	-	-
	11.5 max								

LEGEND: BOT - BOTTOM REINFORCEMENT

TOP - TOP REINFORCEMENT

DC - DERIVED FROM DIAL GAUGE READINGS

BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS

CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE: 4.18 CALCULATED DATA FOR WALL 13

MOMENT (KN-M)	DEFLECTION (CM)	STEEL FORCE (KN)		CURVATURE (RAD/M)			NEUTRAL AXIS (MM)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.		BOT	TOP	BOT	TOP
0	0	0	0	0	-		-	-	-	-
1.94	0.07	1.6	1.4	0.0005	-		-	-	-	-
4.70	0.54	26.1	32.4	0.0005	0.0066		32.3	27.2	2.7	3.9
7.18	1.06	46.3	53.4	0.0090	-		-	-	-	-
9.96	1.66	68.5	75.8	0.0132	0.0159		28.0	25.8	8.0	9.7
12.61	2.39	89.3	97.6	0.0256	-		-	-	-	-
14.62	3.50	100.0	100.0	0.0343	0.0374		25.8	11.5	7.5	16.8
15.84	5.97	10.7	100.0	0.0896	-		-	-	-	-

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.19 CALCULATED DATA FOR WALL 14

MOMENT (kN-m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
3.85	.79	59.6	43.8	0.0010	-	-	-	-	-
7.77	1.54	100.2	78.5	0.0057	0.0162	23.8	28.4	13.8	9.1
10.80	2.32	136.6	116.7	0.0107	0.0230	24.6	27.6	18.2	13.9
13.89	3.12	172.5	147.1	0.0196	0.0296	25.8	28.9	21.9	16.7
16.21	3.72	199.9	171.3	0.0195	0.0147	26.4	29.4	14.6	11.2
18.27	4.39	200.0	197.0	0.0275	0.0402	26.4	29.7	14.6	12.8
18.92	6.12	200.0	200.0	0.0362	-	-	-	-	-
9.18	7.94	186.3	200.0	0.0709	-	-	-	-	-
	10.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT

TOP - TOP REINFORCEMENT

DC - DERIVED FROM DIAL GAUGE READINGS

BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS

CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.20 CALCULATED DATA FOR WALL 15

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
3.56	0.57	20.5	27.8	0.0069	-	-	-	-	-
6.65	1.14	62.2	61.9	0.0071	0.0129	34.0	34.2	6.0	5.9
9.74	1.84	101.2	92.9	0.0118	-	-	-	-	-
13.13	2.67	145.5	123.5	0.0167	0.0274	30.9	34.4	15.5	11.8
15.11	3.16	169.0	146.3	0.0198	0.0323	31.6	34.7	17.5	13.8
17.18	3.81	198.4	170.2	0.0198	0.0379	31.7	35.0	12.1	9.4
19.37	4.96	200.0	200.0	0.0295	-	-	-	-	-
20.46	7.71	200.0	161.5	0.0584	-	-	-	-	-
8.70	8.42	200.0	143.2	0.0848	-	-	-	-	-

LEGEND: BOT - BOTTOM REINFORCEMENT

TOP - TOP REINFORCEMENT

DC - DERIVED FROM DIAL GAUGE READINGS

BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS

CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.21 CALCULATED DATA FOR WALL 16

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
0.78	0.06	0.6	0.3	-0.0004	-	-	-	-	-
1.61	0.11	2.4	0.1	0.0004	0.0007	47.0	45.5	0.2	0.1
2.71	0.43	5.1	12.4	0.0020	-	-	-	-	-
3.41	1.06	6.5	25.9	0.0023	0.0093	67.3	9.5	0.3	9.0
4.47	1.96	18.7	59.0	0.0115	-	-	-	-	-
5.98	3.48	34.7	89.1	0.0224	0.0378	70.1	14.1	1.6	20.8
6.74	5.43	41.9	100.0	0.0327	-	-	-	-	-
6.74	11.28	10.6	100.0	0.1453	-	-	-	-	-

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.22 CALCULATED DATA FOR WALL 17

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0.0	0	0	0.0013	-	-	-	-	-
2.55	0.21	2.9	0.8	0.0027	0.0100	-	-	-	-
3.81	0.94	32.3	32.1	0.0127	-	14.5	14.5	7.3	7.3
5.49	1.81	51.9	49.0	0.0182	0.0237	-	-	-	-
7.02	2.58	69.9	68.6	0.0240	-	21.3	21.6	10.8	10.4
8.54	3.42	88.2	88.4	0.0329	0.0366	-	-	-	-
10.05	4.76	100.0	100.0	0.0805	-	20.0	19.6	9.7	9.9
10.72	10.50	100.0	100.0	-	-	-	-	-	-
10.49	-	100.0	100.0	-	-	-	-	-	-
	15.0 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.23 CALCULATED DATA FOR WALL 18

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0	-	-	-	-	-
1.62	0.04	0.24	0.16	0.0003	-	-	-	-	-
2.89	0.34	0.9	20.5	0.0019	-	-	-	-	-
4.67	1.23	34.5	40.7	0.0086	-	-	-	-	-
6.34	2.02	55.9	63.7	0.0133	0.0119	16.5	14.4	6.8	9.3
7.81	2.73	75.5	81.8	0.0183	-	17.2	16.1	14.4	16.7
9.43	3.62	96.2	100.0	0.0239	0.0252	18.3	11.6	10.6	16.7
10.77	4.98	100.0	100.0	0.0321	-	-	-	-	-
11.68	10.13	100.0	100.0	0.0675	-	-	-	-	-
11.53	12.10	100.0	99.8	-	-	-	-	-	-
	16.5 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

TABLE 4.24 CALCULATED DATA FOR WALL 19

MOMENT (kN.m)	DEFLECTION (cm)	STEEL FORCE (kN)		CURVATURE (rad/m)		NEUTRAL AXIS (mm)		CONC. STRESS (MPa)	
		BOT	TOP	DC	BL.JT.	BOT	TOP	BOT	TOP
0	0	0	0	0.0	-	-	-	-	-
1.13	0.04	0	0	0.0002	0.0004	95.0	95.0	0	0
2.62	0.16	0	4.2	0.0009	0.0016	95.0	46.5	0	0.3
4.06	0.68	23.9	12.9	0.0069	0.0079	31.0	45.0	2.5	0.9
5.49	1.26	37.3	43.5	0.0134	0.0148	28.4	25.4	4.3	5.6
6.89	1.90	51.6	57.5	0.0143	0.0196	26.4	24.6	6.4	7.7
8.36	2.52	66.4	72.5	0.0243	0.0248	25.9	24.3	8.4	9.8
9.90	3.25	85.5	90.1	0.0314	0.0312	25.3	24.3	6.4	7.2
11.83	12.10	88.1	94.9	0.0000	0.0323	25.9	24.1	6.4	7.6
11.55	4.18	100.0	100.0	0.0474	0.0508	25.8	14.9	7.5	13.0
11.83	5.47	100.0	100.0	0.0626	-	-	-	-	-
12.29	6.84	100.0	100.0	0.0800	-	-	-	-	-
12.29	9.52	-	-	0.1736	-	-	-	-	-
	25.5 max								

LEGEND: BOT - BOTTOM REINFORCEMENT
TOP - TOP REINFORCEMENT
DC - DERIVED FROM DIAL GAUGE READINGS
BL.JT - DERIVED FROM BLOCK JOINT STRAIN READINGS
CONC. STRESS - CALCULATED CONCRETE STRESS

A review of the test data readings and the values calculated by the computer program led to several conclusions. The readings taken most consistently during testing were the dial gauge readings, the steel strain gauge readings and the load cell readings at each incremental load application at each of the nineteen test walls. Readings during testing were not taken as frequently on the Demec strain gauges because of the difficulty involved in taking these readings on the walls especially at high levels of load application when the walls were in a precarious state.

The center dial gauge readings and the moment calculated from the load cell readings would most accurately depict the performance of the test walls. The walls deflected in segmented fashion rather than smooth curves due to cracking at the mortar joints. The center dial gauge reading at mid-height most accurately represented the center deflection of the walls at different load levels. The steel strain readings predict the levels at which the steel reinforcement reached yield stress levels. These readings would also indicate the steel stress level at which bond failures occurred.

The steel forces were derived from the steel strain gauge readings. At strain levels above the yield stress of the reinforcement the steel force in the bars is constant. The values listed in Tables 4.7 to 4.24 indicate that the "Moment" due to externally applied forces, the "Deflection" derived from the center dial gauge readings, and the "Steel Force" derived from the steel strain gauge readings vary linearly until steel yield levels are approached.

The "Steel Force" values listed in the tables indicate that the top and bottom reinforcing bars did not reach yield stress levels simultaneously. Theoretically, this should have occurred. However, the reinforced concrete walls were not a homogeneous material and there were some differences in the actual values of externally applied forces at the top and bottom wall locations.

It was also observed that the two steel strain readings at each reinforcing bar did not reach yield stress levels simultaneously. As noted previously, these strain gauges were located at the front and rear of each bar. At load levels approaching yield stress levels in the reinforcing, the steel gauge readings towards the front (compression) face of the wall were lower than that of the gauge at the rear of the bar, indicating a "strain gradient" across the diameter of the bar itself. The effective depth of the walls are small (95mm) compared to the bar size (15mm), as an example, and therefore a "strain gradient" and resultant stress variation occurs across the diameter of the bar.

Since the yield stress in the bar did not occur simultaneously in the top and bottom bars, and in fact, across the area of the bar, a discreet point at which average yield stress in any reinforcing bars was reached is defined as follows.

For our purposes in this paper, initial yield indicates the level at which any one bar achieved an average stress across the bar which was at or above the yield stress of the reinforcement. Full yield indicated the stress level at which any bar reached yield stress levels across the entire area of the bar.

In wall 16, the reinforcing bars were lapped to simulate misplaced reinforcement with the bars placed at two different effective depths in the wall. The top bar at the smaller effective depth reached yield stress levels, whereas the bottom bar at the larger effective depth never reached yield levels.

There were no bond failures prior to initial yield of the steel. The values listed in Tables 4.7 to 4.2.4 for "Moment", "Deflection" and "Steel Force" indicate that before the stress in any bar reached initial yield levels, the values for Moment and Deflection varied linearly. Deflections increased rapidly after full yield levels were reached.

The maximum deflections reached before unloading the walls are also noted in these tables. The deflections noted for the reinforced walls are generally in the

order of 10 cms or greater indicating the relatively large deflections of these walls with no bond or ultimate wall failures occurring. The unreinforced Walls 1 and 2 had relatively small values for deflection of approximately 5 cm at a point where these walls were on the verge of complete failure.

The computer printout data indicated that generally the resisting moments derived from the steel stresses and curvatures from either the dial gauge readings or block joint readings were of the same order as the externally applied moments calculated from the load cells. The average of the three 8" Demec readings across the block joint on the compressive side of the wall were taken as most representative of the masonry wall compressive strains. The neutral axis and the maximum compressive stresses calculated from these values are also listed in Tables 4.7 to 4.24. The "Curvatures" are listed in tabular form in these tables for later use and discussion in Figures 4.11 to 4.16.

The calculated values for the neutral axis and concrete stresses, indicate generally, from the limited number of values tabulated, that as the moments increased, the neutral axis depth decreased and the concrete stresses increased until yield stress levels were reached in the steel, as would be expected. At load levels where yield stresses were reached in the reinforcement, there were some inconsistencies in the values listed with the neutral axis increasing and the concrete stresses decreasing. This is due to the fact that the block joint strains would increase at yield of the reinforcement whereas the steel force value would remain at the yield stress value of the bar, increasing the neutral axis depth and resulting in lower concrete stresses.

The maximum tabulated concrete stresses are 20.5, 20.8, 21.9 MPa (Walls 8, 14 and 16) for grout filled walls and 16.7 MPa (Wall 18) for mortar filled walls. The comparable average values of the block prism specimens were 9.4 MPa (1367 psi) for the grout filled and 14.4 MPa (2091 psi) for the mortar filled specimens. The

comparable average values of the 6" diameter cylinder specimens taken of the grout and fill material in the 1.2 to 2.4m height of wall were 19.0 MPa (2761 psi) and 21.4 MPa (3107 psi) respectively. Although it is known that ultimate flexural compressive concrete stress values are greater than ultimate axial compressive stresses (54) due to the fact that the flexural stresses vary across the applied section, it appears that these block prism values greatly underestimate the capacity of the reinforced masonry walls, and that the cylinder strengths more closely approximate the maximum tabulated values.

Table 4.25 is a summary of wall moments and types of failures observed. The table lists the "Initial Crack Moment", when the initial crack at the mortar joint on the tension face of the wall occurred. "Moment Where Load Drops Off" values are at load levels where no increase of externally applied load was possible, and deflections at these levels are listed under "Centre Dial Gauge Deflection". "Descriptions of Crack Patterns and Failure" are also tabulated; a legend of the listed descriptions follows on the next page.

WALL NUMBER	INITIAL CRACK MOMENT (KN.M)	MOMENT WHERE LOAD DROPS OFF (KN.M)	FAILURES SUDDEN(1) OR SLOW	CENTRE DIAL GAUGE DEFLECTION (CM)	DESCRIPTIONS OF CRACK PATTERNS AT FAILURE
1	0.72	0.72	SUDDEN(1)	4.36	A * UNSTABLE WALL CONDITION
2	1.43	1.43 +	SUDDEN(1)	5.29	A * UNSTABLE WALL CONDITION
3	-	-	SLOW	-	A *
4	-	11.27	SLOW	6.0	A *
5	2.81	15.84	SLOW	-	A (1) *
6	-	28.02	SUDDEN(1)	11.81	C *
7	2.91	7.79	SLOW	-	A
8	2.70	10.90	SUDDEN(1)	7.18	B COMPLETE LOAD DROP OFF AT 12.31 cm DEFLECTION
9	2.55	10.17	SUDDEN(1)	12.29	B
10	2.72	10.97	SLOW	-	C COMP. FACE CRACKING AT 16.5 cm DEFLECTION
11	-	10.86	SLOW	11.82	L
12	-	17.53	SUDDEN(1)	9.37	B
13	-	15.84	SUDDEN(1)	6.0	B
14	-	18.92	SLOW	-	A *
15	-	20.46	SLOW	-	B * FULL COMPRESSION FAILURE MORTAR JOINT
16	0.78	6.74	SUDDEN(1)	11.28	B *
17	2.55	10.72	SLOW	-	A *
18	-	10.77	SLOW	-	A *
19	2.62	11.55	SUDDEN(1)	9.52	A * UNIFORM HORIZONTAL CRACKING TENSION FACE B HORIZONTAL & VERTICAL CRACKING TENSION FACE

(1) Sudden failure due to block failure at large deflections and not to steel yielding or bond failures.

TABLE 4.25 cont'd

LEGEND: RE: FAILURE TYPES	
A	UNIFORM HORIZONTAL CRACKING TENSION FACE
B	HORIZONTAL AND VERTICAL CRACKING TENSION FACE
C	VERTICAL AND HORIZONTAL CRACKS BOTH FACES
*	WALLS NOT RELOADED AFTER INITIAL 15cm JACK TRAVEL
(1)	COMPRESSION FAILURE IMMINENT AT 15cm

At applied loads resulting in full yield of the bars at deflections in excess of 5 cm, it was thought during testing there were bond failures of the bars in Walls 7, 8, 9 and 19. There were very large increases in deflection with very small increases of applied load. On demolition of the walls, the reinforcing bar in Wall 7 exhibited a very large reduction in area (necking) which indicated the bar was approaching tensile failure, with no resultant bond failure. Walls 8, 9, 19 displayed vertical cracking on the tension face at the wall center line which may have indicated a splitting failure. There was also some very minor indications of possible concrete crushing at the bar deformations. However, this indication was questionable because the demolition procedure did not permit undisturbed concrete to be viewed in these instances. Some of the walls had vertical cracking patterns on the tension face of the block, indicating a possible bond splitting failure at deflections in excess of 6 cm. This occurred at Walls 8, 9, 11, 12, 13, 15, 16, 19 at deflections in excess of 4 cm and at full yield of the reinforcement. The splitting is visually depicted in Figure 4.20. Relatively sudden failures occurred in Walls 6, 8, 9, 12, 13, 16, 19, with large bangs occurring in some walls on failure due to sudden block cracking. Wall 6 had an unlapped reinforcing bar, and failure occurred at the compression face of the wall.

The strain level of the reinforcement in the walls, as noted above, was far in excess of full yield strains and it is doubtful that any bond failure occurred. It is felt that the large deflections and crack deformations of the block itself resulted in splitting about the reinforcement and eventual release of the reinforcement in the unconfined grout or mortar fill.

In some cases, for example, Walls 19, 10, 13, 12, the ultimate moment capacity of the walls with lapped reinforcement was above that of the walls with single unlapped bars. This is an indication of the variability of the materials used, methods of construction, and measurements taken.

4.2.3 Moment Versus Deflection

From the applied jack loads, applied moments were calculated. These results were plotted for each reinforced wall against the mid-height wall deflection obtained from the mid-height wall dial gauge reading. The results are shown in Figures 4.5 to 4.16 and are taken from the tabulated values in Tables 4.5 to 4.24. These curves would indicate if the walls performed similarly for similar magnitudes of external applied loads. The curves should be linear to the point of reinforcing steel yield or bond failure, whichever came first.

These moment deflection curves are shown in broken line only from the point where readings were taken by a rule, to the maximum deflection measured. The estimated initial yield stress level of the reinforcement is indicated on the moment deflection curves. The initial yield is indicated rather than full yield. With the relatively small effective depths used, it was felt that the critical stress would be the level at which any reinforcing bar reached an average yield stress on the bar. The stress gradient across the diameter of the reinforcement bar varied significantly in the test data accumulated, as indicated by the steel strain gauge readings recorded during testing.

The dark line curves depicted in these figures are for walls with no laps on the reinforcement. Figure 4.5 compares the results of four equally reinforced (1-15M bar) walls with and two different lap lengths. Figure 4.6 compares the results of a grouted unlapped wall versus the mortar filled walls with two different lap lengths, as well as the wall with misplaced bars, all equally reinforced (1-15M bar). Figure 4.7 indicates the range of the walls with the least (1-15M bar) and most (1-25M bar) reinforcement, against Wall 4 (1-15M bar). Figure 4.8 compares the results of two equally reinforced (1-15M bar) grouted walls, one lapped (Wall 13), the other a single bar (Wall 3), with the least cover to reinforcement. The misplaced bars wall and the wall with no reinforcement are also shown on this

figure for comparison. Figure 4.9 compares the results of the ungrouted, unlapped Wall 4 with 1-15M bar at the center line of the wall versus grouted Wall 5 with the larger unlapped 20M bar, as well as Walls 14 and 15 with two bars lapped with different lap lengths. Figure 4.10 compares the results of the larger 20M bars in grouted walls, Wall 5 with the unlapped bar, Wall 12 with the lapped bar.

Figure 4.5 Moment vs Deflection: Walls 4, 8, 9, 10, 11

Figure 4.6 Moment vs Deflection: Walls 4, 16, 17, 18, 19

Figure 4.7 Moment vs deflection: Walls 4, 6, 7

Figure 4.8 Moment vs Deflection: Walls 3, 13, 16

Figure 4.9 Moment vs Deflection: Walls 4, 5, 14, 15

Figure 4.10 Moment vs Deflection: Walls 5, 12

The moment-deflection curves indicate a relatively straight portion, from the point after initial tension cracking of the block joint to slightly above initial yield, indicating linearity between moment and deflection, and then a flattening of the curve towards ultimate moment capacity with larger increases in deflection.

It is interesting to note that the test walls did not deflect in smooth curves but rather in segmented fashion, depending on the amount of load applied and the size of the reinforcement. Figures 4.17, 4.18, 4.19 indicate this behaviour. In general, the smaller bars resulted in a smoother deflected wall; the curvature of the wall was also smoother on the application of more load, with the crack patterns more evenly distributed on the wall. The walls with the larger bars appeared segmented, with the tension cracks varying in degree of separation at the block joints. A further comparison of Figures 4.5 to 4.10 indicates that the yield moments were generally reached at locations on the moment deflection curves where the shape of the curve flattened out. It is also seen that the walls with lapped reinforcement generally performed as well as those with the unlapped bars for the entire length of the curve and from observations during testing. This can be observed for example in Figure 4.5 by comparing Wall No. 4 (unlapped splice) to the other curves on this figure.

There were no bond failures prior to yield of the steel. Review of the test data of the steel strain gauge readings indicated that before any bar reached an initial stress (average stress equal to the yield stress), the curves were linear. The curves started to flatten out when the stress level in any bar was above yield across the full bar area. The one wall where average yield is in the flat portion of the curves is Wall 3, in Figure 4.8. Wall 3 had no lapped splice of the reinforcement, and therefore bond failure of the lapped splice could not occur. The calculated data for Wall 3 in Table 4.8 indicates that the initial yield is very close

to the point of full yield on the bar. The initial yield across the bar is also very close to the next lowest point on the curve.

In the design of prestressed reinforced concrete members, loss of applied force in the tendons is taken into account. The frictional losses, which are generally small, are due in part, to the friction between the tendons and the surrounding material resulting from the curvature in the tendons. The walls tested displayed large local curvatures due to cracking at the block joint. It is felt that the high local frictional forces at these large joint curvatures had a beneficial effect on the bond strength of the reinforcement.

4.2.4 Moment Versus Curvature

The curvature was calculated from the average of three 8" Demec gauges across the block joint, and the average steel strains. The applied moments calculated from the load cell readings were plotted against the block joint curvature calculated from the block joint strains and steel stresses. These moment-curvature curves were plotted to provide a comparison with the dial gauge data of the moment-deflection curves. The measurements of block joint strains were fewer but more accurate than the dial gauge measurements.

These curves were grouped in similar comparison of walls as noted for the moment deflection curves. The results were also similar. Initial yield level is indicated on these graphs also.

Figure 4.11 Moment vs Curvature: Walls 4, 8, 9, 10, 11

Figure 4.12 Moment vs Curvature: Walls 4, 16, 17, 18, 19

Figure 4.14 Moment vs Curvature: Walls 3, 13, 16

Figure 4.15 Moment vs Curvature: Walls 4, 5, 14, 15

Figure 4.16 Moment vs Curvature: Walls 5, 12

Figure 4.17 Deflected Shape of a
Wall Prior to Yield of Steel

Figure 4.18 Deflected Shape of a Wall
After Yield of Steel

Figure 4.19 Deflected Shape of a Wall
at Point of Collapse

Figure 4.20 Typical Vertical and Horizontal
Crack Patterns for Tension Face of Wall 12

The graphs indicate that curves for the walls with the lapped reinforcement generally follow the slope of the curves the walls with the single bars and are of the same order magnitude. The larger the amount of reinforcement provided, the steeper the slope of the curve, as expected.

Figure 4.13 also indicates the typical curvature from the dial gauge measurements for Wall 4 in dashed line. In comparing this "dial gauge" moment-curvature curve to the "block-joint" moment curvature curve, it is seen that the slopes of the curves are similar.

4.2.5 Bond

(i) General

The moment-deflection and moment-curvature curves indicate that initial yield of the reinforcement in all the walls is in the linear portion of the curves prior to full yield (or ultimate moment capacity of the walls) except for Walls 3 and 16. Wall 3 has a single unlapped bar. No bond failure of the lap could have occurred. The incrementally applied loads during test did not occur at the exact point of initial yield, although an attempt was made to this effect. Review of the raw test data indicates that the maximum stress in the bar was at least 96 percent of yield at the load level prior to that indicated as initial yield on the moment deflection curve. As noted earlier, the initial yield points indicated on the moment deflection curves are taken at the load increment level that indicated average yield stress in the reinforcement and therefore are at a point above which yield stress first occurs on any section of the bar.

The length of embedment or lap of the reinforcement required to develop the yield stress in tension of the steel is referred to as the development length. Bond stress is calculated assuming the bond stress will

be uniformly distributed over the development length. Bond stress distribution is not, however, uniform and in fact will vary from zero at a tension crack in the concrete to a maximum peak just beyond the crack. If the average of the bond stress over the full development length of the bar does not exceed the ultimate capacity of the encasing material, the anchorage force will be greater than the tension force and the reinforcement will not fail in bond.

The average bond stress $u = \frac{d_b \cdot f_y}{4l_d}$

The primary goal of this program was to determine the development length for deformed reinforcing bars of 400 MPa yield strength in masonry block walls loaded in the flexural mode. The development lengths used in the tests were computed as noted previously from the equation:

$$l_d = \frac{(d_b)^2 f_s}{4 \times 9.5 \sqrt{f'_c}}$$

On review of the test data, calculations, the conclusion is that there were no bond failures at yield of the reinforcement for the lap lengths used in all the walls, and it appears likely that this relationship is overconservative.

(ii) Single Bar Lap

Walls 8 and 9 (350mm lap) and Walls 10 and 11 (450mm lap) were used to compare the performance of single lapped bars against Wall 4 (single unlapped bar). Figure 4.5 indicates that the development length used of 350mm was sufficient to develop the yield stress of the reinforcement. There

appears to be no significant differences except for the variations due to materials, construction and measurement noted previously on the performance of these walls. The calculated bond stress u was 5.7 MPa (827 psi) at full yield of the reinforcement for a lap length of 350mm.

(iii) Multiple Bar Laps

Walls 14 and 15 were tested to compare the effects of multiple laps against the singly lapped bars of Walls 8 and 9 and the single unlapped bar of Wall 4. For the development lengths used the bars reached yield stress and there were no bond failures. Except for the differences in levels of moment capacity, the performances of these walls appear to be similar to those of the single lap bars. The calculated bond stress u was 3.9 MPa (560 psi) at full yield of the reinforcement.

(iv) Grout Versus Mortar Fill

Walls 17, 18, 19 were tested to compare the effects of grout versus mortar fill (Figure 4.6). There is no significant difference between the performance of the grout and mortar fill for the development lengths used. In fact Wall 19 (mortar filled) achieved a higher moment capacity with 350mm lap than Wall 4 (grout filled). It is noted that although the block prism strengths for grout and mortar were 9.43 MPa and 14.4 MPa respectively, there was no bond failure prior to full yield of steel in either case.

(v) Bar Size

Wall 7 (10M bar, 200mm lap) and Wall 12 (20M bar, 500mm lap) were tested to compare the effects of bar diameter. As noted previously, the bond stress in Wall 8 and 9 (15M bar, 350mm lap) was 5.7 MPa. For the 10M bar,

the bond stress u was (7.9 MPa) 1146 psi. For the 20M u was 4.9 MPa (708 psi). These values are calculated using the actual yield stresses of the bars used.

Since the actual performance of the walls were similar, and no bond failure occurred at yield levels of the tension reinforcement, no comment can be made as to how ultimate bond strength varies with bar diameter.

(vi) Cover and Spacing

Wall 13 (15M, 350mm lap) was tested to compare the results of minimum cover to reinforcement. Comparing the results of Wall 13 to those of Walls 4, 8 and 9, (except for the expected moment capacities), the walls performed similarly and there were no bond failures at yield.

ACI Committee 408(6) recommends a minimum clear cover of the reinforcement of 50mm for 20 MPa concrete. The cover to the face of the block for Wall 13 is 42mm.

Walls 14, 15 were tested to compare the results of multiple contact lap splices. The center to center spacing of the lapped bars was approximately 40mm. ACI Committee 408 (6) recommends the following formula for the required development length:

$$l_d = \frac{5500 A_b}{\phi K \sqrt{f'_c}}$$

where A_b = area of the bar

ϕ = capacity reduction factor = 0.8

f'_c = strength of the concrete

and K = is a parameter included for concrete cover, in this case equal to C_s , one half the center to center spacing, or cover to the center line of the reinforcement. K need not be more than $3d_b$.

For the required cover of 50mm to the face of the bar, the proposed ACI formula for l_d is:

$$l_d = \frac{23}{\phi} d_b$$

and $l_d = 460\text{mm}$ for a 15mm bar.

The required l_d for the reduced center to center spacing using $K = 1/2$ (40mm) in the noted ACI formula for reduced "cover" is 1269mm. The ratio of l_d calculated by both these ACI formulae is 2.76 for 20 MPa concrete. The development length chosen for our particular test were 1.4 and 1.85 times the basic l_d of 350mm for single lap bars.

The maximum calculated bond stress for these doubly lapped bars was 4.0 MPa (575 psi) for a lapped length of 350mm.

(vii) Comparison: Test Results Versus Code Requirements

Table 4.26 compares the results of the development length used in the test walls with the values proposed by Baynit (10), CSA A 23.3 (12), ACI Committee 408 (6) and includes the author's proposed development lengths.

The above table lists the walls and the number and size of bars. The development length used for the test walls is reduced to 400 MPa by direct ratio to the actual yield stress of the reinforcement used. The development lengths observed by Baynit (10) listed in the tables were interpolated by Baynit (10) for the 10, 15, and 20 M bars from his observed test results for numbers 4, 6, and 8 bars for 400 MPa yield steel. Baynit's (10) proposed values are based on his formula and are presented below.

The formulas developed by Baynit (10) approximate the proposed ACI 408 Committee (6) recommendations for similar strengths of concrete. Baynit's proposed formula are as follows:

$$(i) \quad l_d = 300, \quad \text{where} \quad d_b \leq 11.3$$

$$(ii) \quad l_d = \frac{22 d_b + 50}{\phi}; \quad 11.3 \leq d_b \leq 16$$

$$(iii) \quad l_d = \frac{76 d_b - 800}{\phi}; \quad 16 \leq d_b \leq 25.2$$

where $\phi = 0.8$, l_d and d_b are in mm. The ϕ factor is not included in the listed values in the Table.

The development lengths listed for the CSA A 23.3 (12) values are based on the CSA formulae of

$$l_d = \frac{.04 A_b f_y}{\sqrt{f'_c}}$$

with the appropriate adjustments for cover to reinforcement as noted previously in this paper, with f'_c equal to 20 MPa and f_y equal to 400 MPa.

The development lengths listed for ACI Committee 408 (6) values are based on the ACI formula

$$l_d = \frac{23 d_b}{\phi}$$

The ϕ factor was not included.

The ACI formula is based on f'_c equal to 20 MPa, with 400 MPa yield steel. For reduced "cover" at the lapped splices, the ACI formula used was as noted in this paper in the section "Cover and Spacing".

Figure 4.21 presents the Baynit (10) proposed l_d formula and the ACI 408 Committee (6) proposed formula in graphic form with the ϕ factors included for 2000 psi (13.8 MPa) strength of masonry or concrete.

The author's proposed values are based on the values used in the test walls with the steel yield at 400 MPa.

Figure 4.21 Basic Development Length: Reinforced Concrete ACI 408 Committee Proposal and Block Masonry Baynit Proposal

WALL NO.	REINFORCEMENT NO. & SIZE BAR	DEVELOPMENT LENGTHS l_d (mm)					
		TEST VALUES	BAYNIT (10)		CSA A 23.3 (12) (c)	ACI COMM. 408(b)(2)(6)	AUTHOR'S PROPOSED (c)
			OBSERVED (a)	PROPOSED (b)			
7	1-10M	138	175	300	262	260	150
8	1-15M	280	275	400	368	368	300
9	1-15M	280	275	400	368	368	300
10	1-15M	360	275	400	368	368	300
11	1-15M	360	275	400	368	368	300
12	1-20M	400	440	680	505	450	425
13	1-15M	280	275	400	460	420	300
14	2-15M	412	420	800	460	1012	-
15	2-15M	520	420	800	460	1012	-
16	1-15M	280	275	400	460	420	300
17	1-15M	360	300	440	368	365	300
18	1-15M	360	300	440	368	365	300
19	1-15M	280	300	440	368	365	300

l_d is based on $f_y = 400$ MPa for all values noted

a) mean development lengths interpolated by BAYNIT (10)

b) ϕ -capacity reduction factor not included

c) f'_c equal to 20 MPa

Table 4.26 Comparison of Development Lengths

Figure 4.22 presents the development lengths listed in Table 4.26 graphically for Baynit's (10) proposed values, CSA A 23.3 (12) calculated values and the author's proposed values. Baynit's (10) observed values as well as those used for the test walls, are shown as discrete points on the graph for 400 MPa yield steel. The Baynit (10) proposed values for the 25 M bar size was used for the author's proposed value on this graph since no 25 M bars were tested in this program.

Figure 4.22 Bar Size vs Development Length for Single Bars

4.2.6 Development Lengths: Discussion of Results

The listed values in Table 4.26 show that the development lengths observed and proposed by Baynit (10), as well as the values calculated by CSA A 23.3 (12) and ACI Committee 408 (6) overestimate the values observed in the test walls. The development lengths producing bond failures in both the beam and pullout specimens tested by Baynit (10) were greater than those used in these wall tests. The beams were loaded in a similar fashion with two point loading to establish development lengths at anchorage points. The block prism strength was (13.1 MPa) 1900 psi for the high slump grout used in Baynit's tests. His results also indicated that the pullout tests (underestimate the development lengths required), for beams. The significant difference between Baynit's tests and these wall tests is that all the lapped splices in the wall tests occurred in high moment, zero shear zones and that the reinforcing bars were mechanically anchored at the ends of the walls to prevent any bond failures at the anchorage points.

CSA Standard S304-1977 (18) states that the maximum allowable bond stress for deformed bars is 160 psi. The allowable tensile working stress for steel reinforcing is 24.0 ksi for 400 MPa grade deformed reinforcing. Using this criteria, and the formula $l_d = \frac{d_b \cdot f_s}{4u}$ we calculate the following lap lengths:

for a 10M bar	$l_d = 424\text{mm}$
15M bar	$l_d = 600\text{mm}$
20M bar	$l_d = 731\text{mm}$

These values can be compared against the listed values in Table 4.26 and it is seen that the l_d calculated by CSA S 304 greatly overestimates the values observed in the test walls.

The author's proposed development lengths for 400 MPa yield steel are values slightly above the values observed in the test walls. As seen from Figure 4.22 these values are below those proposed by Baynit (10) and CSA A 23.3 (12) as well as the observed point values.

No bond failures were encountered in the tested walls at yield of the bars. The possible bond failures at large deflections of the walls are not due to tensile bond stresses but likely due to cracking of the block work from the large deformations encountered. It is not possible to establish the required development length from these results, nor the degree of reduction in development length required to bring about bond failures prior to full yield of the bars.

The results also indicate that the variable f'_m , designating block prism strength, used in the development length formula for these wall tests underestimates the strength of the encasing material around reinforcement for bond strength. The concrete stresses calculated in Tables 4.8 to 4.24 resulted in a maximum value of 21.9 MPa for grouted cores, the mean value of the 4" x 4" x 8" grout absorptive prisms was 26.2 MPa. Further testing would be required to determine the block wall strength f'_m to be used to determine development length.

4.2.7 Flexural Failure

Flexural failure occurred in all the reinforced walls. Occurrence of flexural failure was determined by yielding of steel indicated by the steel strain readings on the reinforcement. After the reinforcement yielded, the walls were further loaded until the deflection increased rapidly with small increases in load. This can be seen from the moment deflection curves for the walls. The small increase in load is due to the large differences in yield and ultimate strength of the reinforcement. These increases in load are indicated by Table 4.27 where observations of the full yield moment and ultimate moments are listed. The load increases above full yield

	TEST WALLS	OBSERVED MOMENTS
TABLE 4.27	-	-

WALL No.	MOMENT AT MID-HEIGHT OF WALL (KN.M)			ULTIMATE
	INITIAL YIELD	FULL YIELD		
1	-	-	0.74	
2			1.43 +	
3	14.51	14.51	18.22	
4	9.82	11.27	12.06	
5	12.71	12.71	16.04	
6	22.13	25.0	31.94	
7	7.45	7.79	8.53	
8	9.18	10.29	11.92	
9	8.84	9.82	11.29	
10	10.10	11.31	12.94	
11	9.09	10.28	11.61	
12	15.45	15.45	18.22	
13	12.61	14.62	15.84	
14	16.21	18.92	18.92	
15	17.18	19.37	20.46	
16	6.74	6.74	6.74	
17	10.05	10.05	10.49	
18	9.43	10.77	11.70	
19	9.90	11.55	12.30

Figure 4.22 Vertical and Horizontal
Cracking Patterns for Compression Face
of Wall 6

Figure 4. 24 Deflected Wall 6 at
Point of Vertical and Horizontal
Cracking on Compression Face

Figure 4.25 Uniform Horizontal Cracking at
Tension Face of Wall 10

moment were 26 to 28 percent for Walls 3, 5 and 6 (no laps) and 9 to 18 percent for Walls 4 and all the walls with lapped reinforcement. Crack patterns at Wall 16 are indicated on Figures 4.23 and 4.24. Typical horizontal cracking patterns are indicated in Figure 4.25.

Walls 6 the most heavily reinforced appeared to fail in compression of the block and at the joint mortar. Wall 16 failed similarly in compression of the joint mortar, as did Wall 19 at deflections in excess of 5 cm.

The reinforced walls displayed characteristics of good ductility under flexural loads by their ability to absorb energy without collapse. Except for Walls 6, 8, 10 the walls sustained their loads under large deflections.

Initial tension cracks in the block joints appeared in the reinforced Walls 5, 7, 8, 9, 10, 17 and 19 at a moment of 2.55 kN.m minimum. At Wall 16 initial tension cracks occurred at a moment of 0.78 kN.m. In the unreinforced-walls, initial tension cracks appeared in Walls 1 and 2 at 0.72 kN.m and 1.43 kN.m. The tension stress of the mortar joints is 0.07 MPa for the hollow unfilled wall and 0.12 MPa for the grout filled wall. The allowable tension stress in the present codes is 0.16 MPa and 0.25 MPa respectively for unfilled and filled cores for Type S mortar. These unreinforced walls displayed a brittle failure pattern.

The values listed in Table 4.28 were derived from the test wall moments observed during the tests for lapped splices. The calculated moments are predicted using the formulas developed in CSA A 23.3 Reinforced Concrete Code (12) and the unpublished CSA Standard S304-1-Limit States Design in Masonry (55).

Table 4.28 lists the wall number, effective depth d , area of reinforcement A_s , the yield stress (f_y) and the ultimate stress (f_{ult}) of the reinforcement as well as the block prism strengths (f'_m) of the walls. M_y and M_{ult} are the observed values of the moments developed by the test walls at yield of the reinforcement and the ultimate moment observed respectively.

The ultimate moment values predicted by the proposed Limit States Code (55) are listed as M_{um} , those predicted by CSA A 23.3 (12) are listed as M_{uRC} . The moments calculated are derived from assumptions presented in Figure 4.26. The cross sectional plan of the wall and the idealized beam section are shown. The ultimate compressive strain (ϵ_{cu}) used is as proposed in the above codes, that is, 0.003 for the concrete (M_{uRC}) and 0.002 for the masonry (M_{um}). The ultimate steel strain (ϵ_s) used is derived using the modules of elasticity of steel (200×10^3 MPa) and the actual yield stress of the steel. The strains are assumed to be linear to determine the neutral axis of the section at a distance "c" from the compression face of the beam. The stress block is assumed to be rectangular, the depth of the block being an equal to β .c., where β equals 0.85 for M_{uRC} and 1.0 for M_{um} . The width of the block is equivalent to "b" the full width of the wall (beam) section excepting that where "a" is greater than 35mm, the thickness of the beam flange, the width of the stress block is adjusted to the core width for the section under compression beyond the T-flange. The maximum compressive stress on the beam section is $\xi .f'_m$, where ξ is 0.85 for M_{uRC} and 1.0 for M_{um} . Since all the lapped splice test walls were observed to be underreinforced, the steel stresses govern the flexural moment capacities of the sections.

From statics, using the above assumptions

$$M = A_s . f_y (d - a/2) \text{ where}$$

M is the calculated moment

A_s , f_y , d , a , are as defined above.

No ϕ factor is included in the derived moments.

The derived moments listed under M_{uTUD} in Table 4.28 are calculated using the above formula with f_y equal to the ultimate stress value of the steel and "a"

Figure 4.26 Idealized Beam Design Assumptions

WALL NO.	d (mm)	A _s (mm ²)	f _y (MPa)	f _{ult} (MPa)	f' _m (MPa)	M _y (kN.m)	M _{ult} (kN.m)	M _{um} (kN.m)	M _{uac} (kN.m)	M _{ustub} (kN.m)
7	95	100	580	810	9.4	7.79	8.53	5.2	5.2	7.7
8	95	200	505	800	9.4	10.29	11.92	8.7	8.5	15.3
9	95	200	505	805	9.4	9.82	11.29	8.7	8.5	15.3
10	95	200	500	800	9.4	11.31	12.94	8.6	8.5	15.1
11	95	200	500	800	9.4	10.28	11.61	8.6	8.5	15.1
12	95	300	485	780	9.4	15.54	18.22	12.0	11.6	22.2
13	140	200	505	800	9.4	14.62	15.84	13.2	13.1	22.5
14	95	400	505	800	9.4	18.27	18.92	15.5	14.3	30.5
15	95	400	505	800	9.4	19.37	20.46	15.5	14.3	30.5
16	50,140	200	500	800	9.4	6.74	6.74	4.1, 13.1	4.0, 13.0	-
17	95	200	500	790	14.4	10.05	10.72	8.9	8.8	14.9
18	95	200	525	820	14.4	10.77	11.68	9.3	9.2	15.7
19	95	200	470	765	14.4	11.55	12.29	8.4	8.3	14.3

f_{ult} = ultimate strength of reinforcing steel

M_y = moment developed by test when all steel has reached average yield

M_{ult} = ultimate moment developed by test wall

M_{um} = moment capacity predicted by the proposed limit states masonry code

M_{uac} = moment capacity predicted by the reinforced concrete code CSA A23.3

M_{ustub} = maximum moment capacity predicted by theory assuming steel is fully strain hardened and that the moment arm is approximately equal to 'd'

Table 4.28 Observed and calculated moments for test walls with lapped splices

equal to zero. The assumptions made for this calculation are that the steel is fully hardened and that the concrete or masonry has infinite strength. These M_{USTUD} values should be an upper limit to the moments that could be developed by the test walls.

The listed values of predicted moments M_{um} and M_{uRC} are below the observed moment M_y and M_{ult} of the test walls in all cases except for Wall 16, the wall with misplaced bars. In the case of Wall 16, the predicted moments using the smaller effective d is below the observed values. Since the steel stresses govern, the range of predicted moments are similar but below the observed values; the predicted values by both codes are similar. The difference between the two code values for Walls 14, 15 are due to differences associated with the term $(d-a/2)$. Since the amount of reinforcement is relatively large, the concrete stress block involves the core section of the T-beam section. The maximum compressive stress is modified by γ reducing the internal moment arm for M_{uRC} compared to M_{um} and resulting in the smaller moment value for M_{uRC} .

In all other cases the difference between the internal moment arm $(d-a/2)$ is negligible and the moment values are similar.

The differences between the predicted and observed moments are possibly due to local strain hardening at block joints because of the high local curvatures of the test walls and because predicted moments are conservative.

The predicted ultimate moment M_{USTUD} is large than the observed in all cases except for Wall 7. These predicted values are an upper limit to the moments and should be above the observed ultimate moments in all cases. The M_{USTUD} for Wall 7 is below the observed M_{ult} . A possible explanation is that there was more strain hardening than expected.

4.3 RECOMMENDATIONS

4.3.1 Development Length

The formulas developed by Baynit (10) approximate the proposed ACI 408 Committee (6) recommendations for similar strengths of concrete as indicated by Figure 4.21.

The values listed in Table 4.26 indicate that the values predicted by the codes (6), (12) overestimate the values of development length observed at the test walls. The author's recommendations for development lengths for single lapped bars in thin reinforced masonry walls are chosen slightly above the values observed for the test walls, for 400 MPa yield steel and are as follows:

for 10M bars	$l_d = 150 \text{ mm}$
for 15M bars	$l_d = 300 \text{ mm}$
for 20M bars	$l_d = 425 \text{ mm}$

4.3.2 Ultimate Flexural Moment

Predicted ultimate moment capacities of the test walls are noted in Table 4.28 and are below the observed full yield moments of the test walls. The formula proposed by the codes (12), (55) predicted ultimate moments that are reasonably close to the observed values.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The Conclusions derived from these tests on 17 thin reinforced concrete masonry walls are presented below:

- (i) For single lapped splices the present masonry code requirements overestimate the development lengths for splices in high moment, zero shear areas.
- (ii) For single lapped splices the proposed ACI 408 Committee recommendations for reinforced concrete more closely approximate the required development lengths.
- (iii) For the same strength of block, there is no appreciable difference in the ultimate flexural moment capacity of reinforced block walls for different strengths of grout or mortar fill.
- (iv) The development lengths of lapped splices in these test walls were found to be significantly greater than required. The development lengths used were anticipated from similar masonry beam tests (10) which were performed to evaluate development lengths at the beam supports, with high values of shear and incremental shear changes. The increased capacity of the lapped splices of these test walls is probably due to the absence of significant shear stresses in the area of the spliced bars, as well as the higher than specified block fill strength.

- (v) No definite values can be recommended regarding required development lengths, effect of multiple laps, cover and spacing, type of fill material on lapped splices from these tests since no bond failures occurred for the lapped lengths used.

5.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Test data is required in the following areas to reliably establish the values of development length and ultimate flexural capacity in reinforced concrete block walls:

1. Further tests on lapped splices. The lap lengths in such additional tests should be short enough to achieve bond failure in the majority of cases.
2. The effect of the stress gradient across the reinforcing in thin reinforced block walls loaded in the flexural mode.
3. A reliable test to establish a relationship between the performance of ultimate axial compressive stress and ultimate flexural compressive stress of reinforced block walls.
4. The ultimate compressive strength of filled block for different strengths of block and fill.
5. The ultimate compressive strength of filled block for grout versus mortar fill.

APPENDIX A

LOADING FRAME

The steel loading frame was designed as part of the test program. The frame had to be relatively mobile in a confined space, yet stiff enough to apply loads without significant frame deformations. The basic steel frame was supplied and fabricated by Zagerman and Sons Ltd. The components attached to the frame for use in loading and measurements of the wall were supplied by and fabricated at Carleton University. The frame was donated by Carleton University. The following Figures A.1 to A.7 illustrate the frame.

Figure A.1 Test Frame:As Delivered to Carleton
University

Figure A.2 Test Frame: Ready for
Tests

Figure A.3 Test Frame: Bottom Leg Casters

Figure A.5 Basic Test Frame: Elevation C-C

Figure A.7 Gauge Frame

APPENDIX B

SURFACE PREPARATION AND STRAIN GAUGE INSTALLATION

B.1 STRAIN GAUGE ACCESSORIES

The following were the strain gauge accessories used for surface preparation, abrasion and strain gauge installation (see also Figure B.1).

Surface Preparation and Cleaning

- (1) Disc grinder
- (2) Course and fine files
- (3) 320 grit silicone carbide paper
- (4) 400 grit silicone carbide paper
- (5) M-PREP CONDITIONER-A
- (6) Gauze sponges
- (7) Cotton tipped applicators
- (8) Cholorothene (NU)
- (9) M-PREP NEUTRALIZER-5

Strain Gauge Installation

- (1) 4" x 4" x 1/4" decontaminated glass plate
- (2) Tweezers
- (3) Scotch tape and masking tape
- (4) M-Bond 200 adhesive
- (5) M-Bond 200 catalyst

Figure B.1 Strain Gauge Accessories

- (6) Strain gauge (EA-13-250BG-120)
- (7) Terminals

Lead Wire Installations

- (1) Solder
- (2) M-LINE ROSIN solvent
- (3) Scissors
- (4) Soldering iron
- (5) M-COAT-A
- (6) Lead wires

B.2 SURFACE ABRASION AND CLEANING

A length of 1-1/2" is ground using a disc grinder on the longitudinal rib of the bar. The width of the ground surface is approximately 3/8" with a slight convex curvature. The surface is ground to this slight curvature to reduce the chance of excessively reducing the cross-sectional area. After the initial grinding is done with the disc grinder, the surface is further abraded first with files and then with 320 grit carbide paper. The surface is then degressed with a chlorothene NU solvent and further dry abraded with 320 grit carbide paper. The surface is finally wet abraded with 400 grit carbide paper using M-PREP CONDITIONER-A.

B.3 STRAIN GAUGE INSTALLATION

A liberal amount of M-PREP NEUTRALIZER 5 is applied to the prepared surface and scrubbed with a cotton-tipped applicator. The surface is then carefully

dried using a gauze sponge. The strain gauge and the terminals are placed on a clean decontaminated glass plate (4" x 4" x 1/16"). The strain gauge and the terminals are then picked up from the glass plate with scotch tape and placed on the clean decontaminated bar surface. M-BOND 200 Catalyst is applied at the back of the strain gauge and terminals for at least 1 minute. M-BOND 200 adhesive is then applied and left to cure for approximately 1 minute under thumb heat. Five minutes after the bond is cured, the scotch tape is removed and the masking tape placed over the strain gauge to protect the strain gauge from possible mechanical abrasion. Small diameter wires, 32 gauge, are then soldered between the gauge terminals and anchor terminals with strain relief loops. Three lead wires are then soldered to the anchor terminals for the purpose of a bridge circuit. After the lead wires are installed, the masking tape is removed and the whole gauge is cleaned with M-LINE ROSIN solvent to remove flux. M-COAT-A solution is applied over the strain gauge as a protective coating.

B.4 STRAIN GAUGE TESTING AND PROTECTION

After a strain gauge is installed, it is inspected to determine the adequacy of the bond. The inspection procedure attempts to establish whether any voids exist between the gage and the specimen and whether the adhesive is completely cured. Voids can result from bubbles originally in the adhesive or from the release of volatiles during the curing process. The presence of voids is detected by tapping the gauge installation with a soft rubber eraser and observing the effect on a strain indicator. When a non-zero strain reading is noted, the strain gauge is removed and a new one re-installed.

The stage of cure of the adhesive is determined by two methods. The first method is the measurement of the resistance of the strain gauge. The accepted resistance of the strain gauge was $120 \pm 2 \Omega$. The second method involves subjecting the strain gauge to a strain cycle. The circuit is connected and the bar placed on two supports. The bar is then loaded, by pushing transversely on the bar, so that flexural stresses were induced. A change in the zero load reading indicates the adequacy of cure. A zero shift of 2μ in/in is considered acceptable.

After the strain gauge is tested, a polyethylene sheet is wrapped around it to further protect it from mechanical abrasion. The ends of the polyethylene wrapping are sealed with epoxy glue to prevent water from reaching the strain gauge.

APPENDIX C

COMPUTER CALCULATIONS OF TEST DATA

C.1 GENERAL

After the large amount of test data was assimilated, and corrected by the measuring device calibrations, a series of calculations and curves were made manually to determine, if in fact, any of the test specimens had failed in bond prior to full yield of the reinforcement. The results of these calculations indicated that none of the test specimens had failed. It was also found that the associated specimens ultimate strength in axial compression for the grout and mortar fill was a much higher value than originally contemplated, and as noted in the conclusions, it is felt that further research into the performance of the flexural capacity of reinforced masonry walls is required. The strength of the encasing fill material, rather, than the filled block assemblage strength may also be the governing criteria, for establishing development lengths. This data can contribute to further research in the performance of reinforced masonry walls in these areas, such as the establishment of compressive stress blocks, stress strain and moment curvature relationships for different strengths of block and fill material. With this view and keeping the objectives of the paper in mind, a program was developed at the Carleton University computer center, to further evaluate the test data. The program was developed with the direction of this author and the collaboration of Prof. G. Suter.

C.2 DATA INPUT

The test data (see Table 4.6 for example of Wall 9), was corrected for measurement device adjustment; the wall reinforcement, effective depth, lap length and type of fill were the input data for every wall. This data is shown on Figure C.1 to C.3.

The effective depth used in the calibrations was 95 mm since this was the depth measured and installed rather than the 100 mm listed in the wall designations (Table 3.2). It is noted that the wall numbers noted in the raw test data were renumbered for presentation in this paper and Wall 9A noted in the field tests is now Wall 9.

The formulas, assumptions and procedures, other material variables were programmed into the computer to perform the required calculations for the desired outputs.

C.3 DATA OUTPUT

The data collected is indicated in Figure C.4 to C.7 and is reviewed below in itemized form. All data output was in metric units.

Item 1 Flexural Moment (kN.m)

Flexural moments were calculated from the load cell readings on the walls and are listed at 5 points, that is, the points of application of load, the mid-height of the wall and the strain gauge locations. Validity 1.0 meant load cell data assumed, validity 2.0 meant no load cell data available.

Item 2 Deflection (cm)

The deflections were calculated from the three dial gauge readings on the center line of the wall. Validity 1.0 and 2.0 had similar meanings as noted for flexural moment.

Item 3 Dial Gauge Curvature (rad/m)

The dial gauge curvature was calculated at the mid-height of the wall, using the dial gauge measurements to approximate, (using the equation $z = ay^2 + by + c$ for a parabolic curve), the deflected shape of the wall. a , b and c are derived from the dial gauge readings. z is the vertical distance (from mid-height of the wall) to a point on the curve, and y is the horizontal distance to the same point. Differentiating z with respect to y twice we obtain ϕ , the curvature in terms of the dial gauge measurements.

Validity 1.0 meant that one or two dial gauge readings were missing at that load level, 2.0 meant that all dial gauge readings were missing at that load level. Missing dial gauge readings were set to -999. The curvature was calculated only if all dial gauge readings were available. Validity 1.0 and 2.0 occurred at load levels over 4 cms where reinforcement was beyond yield stress levels.

Item 4 (EI.kN.m²)

Since $\phi = \frac{M}{EI}$ and where M (item 1) and ϕ (item 3) are known, EI can be calculated.

Item 5 Dial Gauge Neutral Axis (mm)

The dial gauge neutral axis was calculated using the curvature ϕ (item 3), and the average of the steel strains at the locations of the strain gauges at the top and bottom bars. Where no laps were used the computer printout indicates top and bottom bar readings which are identical, since there was only one set of two strain gauges at the mid-height of the wall on these unlapped bars. The printout notes values similarly in other items for single unlapped bars.

The neutral axis noted in the printout is the distance from the compression face to the center line of the bar.

Item 6 Steel Force (kN)

The steel force in the reinforcement was calculated using the steel strain measurements recorded to obtain stress levels and yield stresses of 565, 500, 500 and 400 MPa for the 10, 15, 20 and 25M bars respectively. The printout indicates the values for top and bottom bars. Where double lapped bars are used the printout averages the strain readings at both the top and bottom bars to obtain a total bar force at these two locations.

Item 7 Curvature Based on Mid-Block Strain and Each Steel Strain (rad/m)

This curvature was calculated using each steel strain gauge at each bar, and the average of the two 2" Demec readings (item 13). Since the effective depth was known, the curvature could be calculated. If there was insufficient data, the curvature was set to -1000.

Item 8 Curvature Based on Joint Strain and Each Steel Strain (rad/m)

This curvature was calculated using the average of the three 8" Demec readings (item 14) and the steel strains noted in Item 7.

Item 9 Rear Strain Curvature (rad/m)

This curvature was calculated using the average of the three 8" Demec readings across the block joint on the compression side (item 14), the readings of the 8" Demec gauge on the tension face (item 16) of the wall, and the total wall thickness.

**Item 10 Curvature Based on Average Block Strain and
Each Steel Strain (rad/m)**

This curvature was calculated using the average of all the compression strains (item 15) and the steel strains noted in Item 7.

Items 11 & 17 Average Block Strain Curvature (rad/m)

This curvature was calculated using the average of the two 2" Demec readings (item 13) and the average of all the steel strain gauge readings.

Items 12 & 18 Average Joint Strain Curvature (rad/m)

This curvature was calculated using the average of the three 8" Demec readings (item 14) and the average of all the steel strain gauge readings.

Item 13 Block Strain

The block strain is calculated from the average of two 2" Demec readings on the compressive side of the wall.

Item 14 Joint Strain

The joint strain is calculated from the average of three 8" Demec readings on the compression side of the wall.

Item 15 Average Strain

The average strain is calculated from the average of the average joint and average block strains on the compression side of the wall.

3 3

OF / DE

Item 16 Rear Strain

The rear strain is calculated from the 3" Demei gauge on the tension face of the beam.

Item 19 Block Strain Neutral Axis (mm)

The neutral axis, measured from the compression face, is calculated using the average of the two 2" Demei readings (Item 13) and the average of the steel strain gauge readings at the top and bottom bars. Since the effective depth is known, the location of the neutral axis can be calculated.

Item 20 Joint Strain Neutral Axis (mm)

This neutral axis is calculated similarly to Item 19, using the values of the three 3" Demei readings in Item 14.

Item 21 Dial Gauge Neutral Axis (mm)

This neutral axis is calculated using the dial gauge curvature (Item 5) and the average steel strains at the top and bottom bar locations.

Item 22 Resisting Moment (kN.m)

The resisting moments were calculated using the steel force, Item 6, and the neutral axis calculated in Item 21 for the "Dial Gauge Neutral Axis" moments, or the neutral axis calculated in Item 20 for the "Joint Strain Neutral Axis" moments. For load levels below which the average stress in any rebar had not reached yield stress, a triangular stress block was used to determine the moment lever arm. Once yield in any bar was reached, the Whitney stress block (distance from compressive face to neutral axis times .85) was used to determine the moment lever arm. The steel force multiplied by the lever arm was the resisting moment with no capacity reduction factor ϕ included.

Item 23 Concrete Stress (MPa)

The concrete stresses were calculated based on the stress block distribution and neutral axis noted in Item 22 using the bar steel forces to attain the concrete stresses.

Item 24 Yielding - Steel Stress Level

This level was noted as the level at which the average stress in any bar reached yield stress levels, as determined from the steel strain gauge readings.

(5)

[illegible]

Figure C.1 Computer Printout Sheet 1

11020	111280	111337
10796	10447	10423
5491	9803	10037
8902	8918	9245
8360	8312	8712
6106	5903	6819
7492	5401	6371
1740	5545	1932
578	7	1823
1000	9	0

40 CELL READINGS (LBS.)			
I.C.	P.L.	P.R.	I.R.
0	0	0	0
151	102	31	42
315	244	152	209
452	400	309	449
600	543	464	503
740	618	533	580
810	803	723	733
1000	890	878	855
1030	944	1027	924
1070	824	916	971
1050	398	1086	965
1130	927	1115	993
1140	890	1110	1000

STEEL STRAINS			
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Figure C.2 Computer Printout Sheet 2

J.R.	I.F.	D.R.	B.F.
0	0	0	0
6	0	6	0
45	22	143	132
1132	802	608	623
1712	1225	1214	1143
2195	1578	1708	1537
2775	2028	2349	2044
2937	2492	2698	2384
3045	4772	2991	2955
3078	4900	4840	10124
3115	6215	1204	13220
3163	69874	1134	15542
3818	69874	500	10124
J.R.	I.F.	D.R.	B.F.
0	0	0	0
6	0	6	0
45	22	143	132
1132	802	608	623
1712	1225	1214	1143
2195	1578	1708	1537
2775	2028	2349	2044
2937	2492	2698	2384
3045	4772	2991	2955
3078	4900	4840	10124
3115	6215	1204	13220
3163	69874	1134	15542
3818	69874	500	10124

Figure C.3 Computer Printout Sheet 3

MOMENT LENIAM	DIAL GG. CURVATURE	CURVATURE BASED ON COVERAGE MODA STRAIN AT 2.5 INCH CELL STRAIN (AR/IN)					P.R.	P.F.
		AREA ST. CURVATURE	T.A.	T.F.	JOINT STRAIN	AVERAGE STRAIN		
.000	.000000	.000000	.000000	.000000	.000000	.000000	.000000	.00
1.714	.000194	.000194	.000194	.000194	.000194	.000194	.000194	.00
3.524	.001282	.001282	.000812	.000489	.000489	.000489	.001282	.00
4.025	.003216	.003216	.001108	.000688	.000688	.000688	.003216	.00
5.722	.013188	.013188	.019112	.014823	.014823	.014823	.014254	.00
7.167	.039184	.039184	.031108	.024601	.024601	.024601	.024601	.00
8.827	.078176	.078176	.062216	.049202	.049202	.049202	.049202	.00
10.824	.123750	.123750	.097747	.077737	.077737	.077737	.077737	.00
11.224	.000000	.000000	.000000	.000000	.000000	.000000	.000000	.00
MOMENT (MEM) LENIAM	DIAL GG. CURVATURE	CURVATURE BASED ON COVERAGE MODA STRAIN AT 2.5 INCH CELL STRAIN (AR/IN)					P.R.	P.F.
		AREA ST. CURVATURE	T.A.	T.F.	JOINT STRAIN	AVERAGE STRAIN		
		.000100	.000100	.000100	.000100	.000100		
		.000194	.000194	.000194	.000194	.000194		
		.001282	.001282	.000812	.000812	.000812		
		.003216	.003216	.001108	.001108	.001108		
		.013188	.013188	.019112	.019112	.019112		
		.039184	.039184	.031108	.031108	.031108		
		.078176	.078176	.062216	.062216	.062216		
		.123750	.123750	.097747	.097747	.097747		
		.000000	.000000	.000000	.000000	.000000		
		.000000	.000000	.000000	.000000	.000000		
MOMENT (MEM) LENIAM	DIAL GG. CURVATURE	CURVATURE BASED ON COVERAGE MODA STRAIN AT 2.5 INCH CELL STRAIN (AR/IN)					P.R.	P.F.
		AREA ST. CURVATURE	T.A.	T.F.	JOINT STRAIN	AVERAGE STRAIN		
		.000100	.000100	.000100	.000100	.000100		
		.000194	.000194	.000194	.000194	.000194		
		.001282	.001282	.000812	.000812	.000812		
		.003216	.003216	.001108	.001108	.001108		
		.013188	.013188	.019112	.019112	.019112		
		.039184	.039184	.031108	.031108	.031108		
		.078176	.078176	.062216	.062216	.062216		
		.123750	.123750	.097747	.097747	.097747		
		.000000	.000000	.000000	.000000	.000000		
		.000000	.000000	.000000	.000000	.000000		

Figure C.6 Computer Printout Sheet 6

(17)		(18)		(19)		(20)		(21)	
MOMENT (UNIT)		AVE. BLOCK CURVATURE		BLOCK AREA (SQ. FT.)		AVERAGE N.A. (IN)		MIN. CG N.A. (IN)	
CENTRE		CURVATURE		MOMENT (UNIT)		MOMENT (UNIT)		MOMENT (UNIT)	
0.0000	0.0000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2.5159	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
4.0437	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
5.6894	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
7.0413	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
8.3013	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
9.7951	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
10.7097	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
13.0939	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
17.8534	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11.2648	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11.1792	0.0000000	0.0000000	0.0000000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

(22)		(23)		(24)	
MOMENT (UNIT)		AVE. BLOCK CURVATURE		BLOCK AREA (SQ. FT.)	
CENTRE		CURVATURE		MOMENT (UNIT)	
0.0000	0.0000	0.0000000	0.0000000	0.0000	0.0000
2.5159	0.0000	0.0000000	0.0000000	0.0000	0.0000
4.0437	0.0000	0.0000000	0.0000000	0.0000	0.0000
5.6894	0.0000	0.0000000	0.0000000	0.0000	0.0000
7.0413	0.0000	0.0000000	0.0000000	0.0000	0.0000
8.3013	0.0000	0.0000000	0.0000000	0.0000	0.0000
9.7951	0.0000	0.0000000	0.0000000	0.0000	0.0000
10.7097	0.0000	0.0000000	0.0000000	0.0000	0.0000
13.0939	0.0000	0.0000000	0.0000000	0.0000	0.0000
17.8534	0.0000	0.0000000	0.0000000	0.0000	0.0000
11.2648	0.0000	0.0000000	0.0000000	0.0000	0.0000
11.1792	0.0000	0.0000000	0.0000000	0.0000	0.0000

Figure C.7 Computer Printout Sheet

Sample calculations of the calculated data for moment and deflection at the mid-height of the test walls listed in Tables 4.7 to 4.24 is presented below. The calculations are presented for Wall 9 from observed data listed in Table 4.6 and in the computer printout sheets in this appendix for load level 9. Similar calculations can be made for the steel force, curvature, neutral axis and concrete stresses as noted in this appendix.

- a) The moment calculated at mid-height of the wall is as follows:

Applied jack loads at load cell No. 1 and No. 4

$$= 1030 \text{ lbs} + 924 \text{ lbs} = 1954 \text{ lbs (8.9 kN) total at top jacks}$$

Applied jack loads at load cell No. 2 and No. 3

$$= 964 \text{ lbs} + 1027 \text{ lbs} = 1991 \text{ lbs (9.0 kN) total at bottom jacks}$$

Using the 1.2m spacing between jack loads and supports we get a

$$\text{bottom wall reaction equal to } \frac{8.9 \times 1.2 + 9.0 \times 2.4}{3.6} = 9.0 \text{ kN}$$

The moment at mid-height of the wall is then

$$\text{equal to } 9.0 \times 1.8 - 9.0 \times 0.6 = 10.8 \text{ kN.M}$$

- b) The deflection at mid-height of the wall is derived from dial gauge

No. 2 readings:

$$\text{Initial reading} = 12285 \times 10^{-2} \text{ mm}$$

$$\text{Reading at load level 9} = 5905 \times 10^{-2} \text{ mm}$$

$$\text{Difference} = 6380 \times 10^{-2} \text{ mm (6.38 cm)}$$

APPENDIX D

STRAIN GAUGE WIRING, MEASURING DEVICES

The following Figures D.1 to D.4 illustrate the schematic wiring diagrams, the Ellis-Vishay digital readout, the Budd SB-switch and balance units used to record the steel strain gauge readings during the wall tests.

Figure D.1 Schematic Wiring Layout for Strain Gauge Measurements

Figure 1-1

Unit Model SB-1

SPECIFICATIONS (D-3)

2.1 CAPACITY	- 10 channels plus OPEN position.
2.2 EXTERNAL CIRCUITS	- Will accept quarter, half or full bridge circuits or any combination. Quarter bridge inputs may be 2- or 3- wire circuits.
2.3 INPUT BRIDGE RESISTANCE	- 50 to 2000 ohms.
2.4 OUTPUT	- Compatible with any Strain Indicator.
2.5 BALANCE RANGE
(Gage Factor at 2.00)	- Quarter and Half Bridge:
$\pm 2000\mu c$ with 120-ohm half bridge in indicator.
Full Bridge:
$\pm 2000\mu c$ for 120-ohm bridge
Range proportional to bridge resistance.
2.6 BALANCE RESOLUTION
(Gage Factor at 2.00)	- Better than $1\mu c$
2.7 SWITCHING REPEATABILITY	- Better than $1\mu c$
2.8 CASE	- Aluminum; dust and spray-tight with detachable cover.
2.9 SIZE	- 9" x 6" x 6".
2.10 WEIGHT	- 5 1/2 lbs.

Figure D-3: Specifications: Switch and Balance Unit

Figure D-4: STRAIN GAUGE INDICATOR V E 20A

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