MINIMUM STEEL REQUIREMENTS FOR MASONRY WALLS
OUT-OF-PLANE FORCES

By

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ABSTRACT

The behaviour of reinforced masonry walls subjected to out-of-plane forces, and the limitations on amounts and spacing of reinforcement required by the present code were examined.

A research project was carried out in order to determine experimentally the capacity of the walls, the appropriate spacing of the main reinforcing steel, and the effectiveness and appropriate spacing of transverse reinforcement. Full size non-loadbearing walls were tested under monotonic quasi-static loading.

Test results showed that for typical 8 feet (2.44 m) storey height walls, the main steel may be spaced at more than 4 feet (1.22 m) and full primary bending moment may still be achieved. Joint reinforcement appeared to be effective as distribution steel or as main steel for horizontal spans.
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Masonry is amongst the oldest but possibly the least understood of building materials in widespread use today. This lack of knowledge, mainly of its structural behaviour, has led to the misuse of masonry through inadequate or even non-existent design procedures and poor construction practice.

Historically masonry structures were unreinforced and gained their flexural strength through gravity loads. During the 1930's the use of reinforced masonry was introduced, permitting the construction of more slender structures. Some research work started in a number of countries in Europe and North America late in the 1920's and early in the 1930's. Engineered masonry construction, based on engineering analysis, was introduced in Codes during the 1960's when the American Standard Association (ASA) Building Code Requirements for Reinforced Masonry appeared, although reinforced masonry had been specified since 1935 in the Uniform Building Code (UBC).

The new provisions in the section on masonry design in the 1965 National Building Code (NBC) of Canada, on the structural analysis of reinforced and loadbearing masonry, marked the beginning of engineered masonry in this country. The section on masonry design was revised for the 1970 edition of NBC and again for its 1975 edition. The new Canadian Standard Association (CSA) Standard Can3-S304 Code was produced by a joint effort of the CSA and NBC.

The introduction of reinforced masonry allowed the use of thinner wall
sections, created the ability to span over openings, and generally made masonry a more economical building material. Today, for example, reinforced masonry is used as the structural support in high rise buildings located in seismically active zones.

Reinforced masonry is thus a relatively new structural material and only recently has there been any appreciable research into its behaviour. Many clauses of the codes presently in force have been produced by applying theories of reinforced concrete to reinforced masonry, and then introducing large factors of safety to account for the unknown behaviour of the masonry. Other clauses in the codes seem to have neither an experimental nor an analogous basis behind their requirements. Thus, there are many subjects in reinforced masonry that require experimental research and the development of suitable analytical methods before the full capability of the material can be safely utilized.

1.1 Scope

In this thesis the behaviour of a reinforced masonry wall subjected to lateral loads (loads applied perpendicular to the wall surface) is investigated. Lateral loads arising from earthquake excitation are the main consideration, but the results could be applicable to lateral loads arising from wind or mechanical means.

When reinforcing a wall to sustain lateral loads (out-of-plane bending), the direction of the main bars will depend upon the support conditions. The quantity of main reinforcement can be established following well known principles if the transverse forces and the boundary conditions are known. The design of the distribution steel and its influence on the maximum acceptable spacing of the principal reinforcement are not as
amenable to analysis. Thus, we can say that the designer's problem is not how much principal steel to use, but when and how to place the distribution steel in order to get the maximum spacing of the principal reinforcement.

The minimum amount and maximum spacing of steel in reinforced masonry walls is generally specified in codes without consideration of the location in the building, or the configuration of the wall, and in some cases the expected level of seismic activity; they are based on successful past practice. Those steel requirements represent a considerable economic factor in the use of masonry and have given rise to controversy among the engineer and the contractor.

The aim of this thesis is to examine the limitations on amounts and spacing of reinforcement required by the present Canadian Code, when the wall is loaded by out-of-plane forces. A review of some of the past research work carried out on plain and reinforced walls tested under in-plane compression, flexural loading and their combinations, is presented. The theories used in the analysis and design of plain and reinforced masonry, as well as the procedures for designing masonry walls under different codes of practices are examined. The properties of individual materials and masonry assemblages, of importance in the design of masonry as well as in research work, are studied and the experimental data obtained on the materials used in the University of British Columbia series of tests are presented. The tests on masonry walls described in this work are part of an experimental program design to determine the actual minimum amounts and maximum spacing of steel necessary to provide the strength required for the various seismic zones.
The long term program will take the following form:

(a) Study of reinforced masonry walls under monotonic quasi-static loading to failure to determine the strength and load-deformation characteristics of the wall panels with various amounts and arrangements of the steel reinforcement.

(b) The results obtained in (a) to be confirmed by quasi-static cyclic loading.

(c) Design procedures emerging from (a) and (b) to be verified by testing under simulated earthquake loading on a shaking table.

To date the tests have involved only monotonic quasi-static lateral loads with the exception of 2 walls which were rotated and subjected to one quasi-static load reversal. The immediate aims of the present study are:

1) To establish the maximum spacing of main steel without any transverse distribution steel, for walls spanning either horizontally or vertically.

2) To determine the effect of the distribution steel on the spacing of the main steel.

3) To determine the efficiency of joint reinforcement as horizontal distribution steel or as main steel for horizontally spanning walls.

4) To determine a method of predicting the ability of the masonry to span between the main reinforcement or lateral supports.
CHAPTER 2
STATE-OF-THE-ART

This chapter presents a review of some of the research work which has helped to form the basis of the current analysis and design of masonry walls.

Concrete masonry is primarily a load-bearing material with a relatively high compressive strength. The tensile strength is low and highly variable. Massive masonry walls have the ability to resist lateral loads by gravity stability, but very little information has been available to enable engineers to design thin walls for lateral loads.

Since masonry has a very low tensile strength and is brittle in tension, reinforcement is needed in tension areas. Reinforced masonry walls are reasonably ductile and their behaviour is quite similar to that of reinforced concrete. Thus, reinforced masonry design is based upon theories long used in designing reinforced concrete elements. Studies are being done to incorporate the more sophisticated concepts of ultimate strength and limit state design. Work in the laboratory is being helped by the use of the Finite Element analysis to reproduce, interpret or predict behaviour of walls during tests.

2.1 Bending of Masonry Walls

The strength of masonry walls subjected to lateral loads depends upon the horizontal and vertical flexural strength, which is affected by in-plane forces either from loads or arching action due to edge restraint, the amount and arrangement of reinforcing steel, and the strength of the
masonry components. Shear and bond failures may also cause overall failure.

A report on six single wythe concrete block wall panels tested in pure horizontal flexure was presented by Cox and Ennenga (Ref. 1). They believed that the building code requirements of transverse support provided at either horizontal or vertical intervals not exceeding 18 times the nominal wall thickness, though satisfactory for walls without openings, could not be applied regardless of the wall shape and, in the case of walls with openings, the size and arrangement of those openings.

Three panels were built of 8 x 8 x 16 in. (200 x 200 x 400 mm) hollow concrete blocks. Three additional walls, built in the same fashion, had standard joint reinforcement (#9-gauge wire). All the specimens were 8 ft. (2.44 m) long. The reinforced walls were 5 courses high, the unreinforced ones were 3, 5 and 7 courses high. All the panels were tested under conditions of simple supports, spanning horizontally with a concentrated line load in the center.

Test results indicated that the walls cracked at bending moments ranging from 1140 ft lb/ft (256 Nm/m) to 1280 ft lb/ft (287 Nm/m). The modulus of rupture, based on the gross area was about 110 psi (0.76 MPa). The reinforcement did not influence the load under which the wall cracked, but it did act to control the cracks and to preserve the wall after it cracked.

In 1961 Hedstrom (Ref. 2) reported a series of tests on concrete masonry walls. The program was aimed at determining: (a) the flexural strength of vertical spanning walls, (b) the effect of horizontal joint reinforcement on the strength of a horizontally spanning wall and (c) the effect of vertical loads on the flexural strength of walls spanning vertically.
The running bond specimens were 4 x 8 ft. (1.22 x 2.44 m) and 8 x 4 ft. (2.44 x 1.22 m), built with 8 x 8 x 16 in (200 x 200 x 400 mm) units. Some of the walls spanning horizontally were reinforced with #9 gauge joint reinforcement in the horizontal mortar joints (some at 8 in. on center, others at 16).

From his tests results the author concluded that: (a) for vertical spans, walls built with type M or S mortars showed mortar bond strengths of 60 psi (0.41 MPa) and 30 psi (0.20 MPa) respectively, (b) for horizontal spans, the particular joint reinforcement used did not significantly increase the load at first cracking but it was effective in increasing the ultimate strength of the walls; the ratio of cracking load in the horizontal direction to the one in the vertical direction ranged from 2 to 4, (c) the addition of a vertical compressive load to the walls spanning vertically significantly increased their flexural strength.

Fishburn (Ref. 3) performed flexural tests on 34 concrete masonry walls. Twenty-eight of the walls had normal horizontal bed joints, but 6 walls were constructed so that the bed joints were vertical. The supports consisted of simply supported top and bottom reactions, and the lateral load was applied through line loads at the quarter points.

The wall specimens were 4 feet long (1.22 m) by 8.67 ft. high (2.64 m) built with 8 x 8 x 16 in. (200 x 200 x 400 mm) concrete blocks.

The modulus of rupture of the walls tested with the bed joints normal to the span, based on the gross area as given by the authors ranged from 10 psi (0.09 MPa) to 24 psi (0.17 MPa) in walls with type N mortar and from 17 psi (0.12 MPa) to 33 psi (0.23 MPa) with type S mortar. The flexural strength of the walls tested with the bed joints parallel with the span was about 3 times higher.
Sahlin (Ref. 4) reported an investigation by Nilsson (Ref. H.23 in Ref. 4) where the modulus of rupture in horizontal bending was found to be 3 to 6 times higher than the modulus of rupture in vertical bending.

The importance of restraining the vertical edges of plane masonry walls against rotation and in-plane lateral displacement was studied by C. Anderson (Ref. 5) and by C. Anderson and N.J. Bright (Ref. 6). As reported in Ref. 5 six concrete block walls were tested. The base of all the walls were built on a mortar bed to provide partial rotational restraint, and the tops were free. Three of the walls were simply supported along the vertical edges while the other three were built into 3.3 foot (1.0 m) long crosswalls which in turn were tied to the reaction floor. A second series of tests (Ref. 6) consisted of 5 concrete walls and one composite wall, all built with the same type of blocks used in the previous series and constructed within a steel framework that could provide both rotational and in-plane edge restraint. The authors (Ref. 6) observed that when the vertical supports provided moment restraint and an arching type reaction, the walls could support much higher lateral loads. From this the authors concluded that any theory that attempts to predict the strength of practical walls must be capable of including the effect of edge restraint. A theory based simply on the bending capacity of unrestrained walls is not adequate.

West et al. (Ref. 7) described flexural tests on more than 1000 small masonry samples (wallettes) and on more than 100 full-sized walls up to 18 ft. (5.5 m) long and 11.8 ft. (3.6 m) high, under uniform lateral load. In their report, the writers defined the orthogonal ratio as the ratio of the flexural strength for horizontal spans to that for vertical spans. The small wall specimens, tested under four point loading were used to determine the flexural strength in the vertical and horizontal spans.
In their tests the authors obtained orthogonal ratios of about 3.

B.A. Haseltine et al. (Part II, Ref. 7) compared the experimental results with some theoretical methods of analysis, assuming that the walls behave as plates. One of the methods used was yield line analysis. The authors pointed out that one of the major advantages of this theory is that it is possible to easily feed into the calculations different strengths in two orthogonal directions, enabling the bending resistance of walls to be worked out using the actual ratio of strengths. It is also possible to take into account any bending resistance over a support.

Lateral pressures predicted using yield line theory were compared to the experimental failure pressures. The moment capacities were assumed to be proportional to the strengths determined by the wallets tests, and edge restraints as measured experimentally were included in the calculations. A plot of the experimental pressures vs. the calculated pressures showed scattered results, the prediction being good for long walls but unconservative for short walls.

A design method, based on yield line analysis, was recommended by the authors. They suggested that the partial end restraint be replaced with either full fixity or no restraint, depending on whether the walls were continuous or simply supported. Characteristic flexural strength of brickwork was given as a function of the water absorption of the bricks and the type of mortar, and an orthogonal ratio of 3 was recommended. Calculated pressures were plotted against the actual failure pressures. All the results were conservative, which was explained by the authors by pointing out that in this case continuity (i.e., edge restraint) of the walls was ignored and flexural strengths were smoothed out by using a characteristic value.
Scrivener (Ref. 8) reported on two series of tests of reinforced brick walls subjected to uniform lateral load. In the first series the walls were simply supported, but they were rotated to lie in the horizontal plane and so the wall dead load was incorrectly applied. A second series was then run in which the walls were kept in the vertical position and spanned vertically with simple supports. A few load reversals, by applying the load to the other face, were also carried out.

Scrivener found that in both series, assuming a lightly reinforced wide beam section (i.e. the whole wall cross-section as the beam section and assuming no failure between bars) and applying ultimate strength theory as for reinforced concrete, the test yield load could be predicted within a small deviation. The walls presented a highly ductile behaviour characterized by large inelastic deformations.

2.2 Walls Under Combined Bending and Compression Loads

Many walls are subjected to both vertical axial loads and bending moments which are caused by either lateral loads or eccentric axial loads.

In his report on tests on 4 ft. x 8 ft. x 8 in. (1.22 m x 2.44 m x 200 mm) concrete masonry walls, Hedstrom (Ref. 2) found that with few exceptions walls subjected to an eccentric vertical load with \( e = t/6 \), where \( e \) is the load eccentricity and \( t \) the thickness of the wall, failed at 35 to 50% of the strength of the individual blocks. He also observed that the compressive strength of the walls depended primarily on the strength of the blocks and was little affected by the strength of the mortar. For walls subjected to a vertical plus lateral load, he found that after the first cracking the precompressed walls continued to carry increasing lateral loads up to about 1.25 times the cracking load.
Fishburn (Ref. 3) performed a series of 26 tests on 4 ft x 8 ft x 8 in (1.22 m x 2.44 m x 200 mm) walls loaded with a vertical load applied with an eccentricity of t/6.

The author concluded that the compressive strength of the concrete masonry walls increased only slightly with increasing mortar compressive strength. Based on the net area, the strength of the walls was about half of the block strength.

In 1970 Yokel et al. (Ref. 9) reported on a research program aimed at determining the effects of slenderness and load eccentricity on the strength of slender concrete masonry walls. The specimens included 32, 8 in. (200 mm) unreinforced concrete masonry walls and 28, 6 in. (150 mm) reinforced concrete masonry walls. The specimens were 4 ft. (1.22 m) wide by 10 ft. (3.05 m), 16 ft. (4.88 m) and 20 ft. (6.10 m) high. The walls were tested to destruction under compressive loads applied axially and at eccentricities of 1/6, 1/4 and 1/3 of the wall thickness. The test set up was designed to prevent rotation at the bottom while allowing rotation at the top.

Three and two-block high prisms built in stacked bond were tested, under the same loading conditions that were used for the full scale specimens. The prism tests indicated that flexural compressive strength increased with increasing strain gradients (i.e. eccentricity). The flexural compressive strength was defined as $a_\text{f'}$, where $f'_m$ is the uniaxial compressive strength of the masonry, and $a$ is a function of $e$ (i.e. the strain gradient) and $a > 1.0$.

There was a good correlation between short wall (10 ft) strength and prism strength under concentric load. Under eccentric load the prisms
developed higher compressive strength than the reinforced wall specimens which failed near the top, where the eccentric load was applied. The authors concluded that failure in this region is an indication that slenderness had no significant effect on these short walls. The discrepancy between the flexural strength of the prisms and walls is attributed to poor composite action amongst the grout, blocks and reinforcement of the walls.

The authors recommended the introduction of the moment magnifier method in calculating the moments acting on slender masonry walls. The maximum moment should be approximated by: $M = P e C_m/(1-P/P_{cr}) > P_e$, where $C_m = 0.6 + 0.4 M_1/M_2$, a factor dependent upon different end conditions ($M_1$ and $M_2$ are the end moments and $|M_1| > |M_2|$), $P$ is the axial load, $e$ is the largest eccentricity, and $P_{cr} = \pi^2 E I/(kh)^2$. The magnitude of this moment magnifier effect in the case of masonry depends on several parameters: 1) end fixity: flat ended conditions at the base of the walls gave on a minor amount of fixity. For such a condition the authors assumed end moments and an effective length as shown in Figure 2.1. 2) Stiffness EI: $E$ decreases with increasing stresses and $I$ decreases if the section cracks. For the reinforced walls the equivalent EI was approximated by $EI = E I_n /2.5$, where $E_i$ is the initial tangent modulus of elasticity and $I_n$ is the moment of inertia of the uncracked section. For the unreinforced walls EI was taken as $E I_n /3.5$.

For the 6 in. (150 mm) walls a short wall interaction curve for the section capacity was developed on the basis of the average axial strength of the prisms ($f'_m$). From this curve, interaction curves for slender walls were developed by reducing the moment at each level of $P$ by the moment magnifier factor. It was concluded that, except for the case of the 20 ft.
(6.10 m) walls with $e = t/3$, the theoretical interaction curves were conservative. A new wall interaction curve was calculated on the basis of the flexural strength of the prisms for $e = t/3$. In this case the trend of the tests results and the actual failure loads were in good agreement with the theoretical predictions.

In the case of the 8 in. (200 mm) walls the conclusion was that the strength of slender walls was conservatively predicted by the moment magnifier method when basing the calculations on the average axial prism strength. The magnification of the moments, as well as the strength of slender walls were approximately predicted by the moment magnifier method, when the flexural compressive strength at load eccentricities greater than $t/3$ was assumed to equal the average flexural strength of the prisms loaded at $e = t/3$.

In 1971, Yokel et al. (Ref. 10) published a report on a second series of tests on 90 walls of 10 different types of masonry construction using both bricks and concrete blocks under various combinations of vertical and transverse load. The purpose of their program was to develop analytical procedures to predict the strength of masonry walls subjected to combined axial and lateral loads. Both the axial and lateral loads were applied uniformly.

The concrete walls were 4 x 8 ft. (1.22 x 2.44 m) built in running bond with 8 x 8 x 16 in. (200 x 200 x 400 mm) blocks with either type N or high-bond mortars.

In order to analyze their test results the authors derived the equations required to construct an interaction curve. The flexural tensile
strength of masonry was assumed to be low compared with the flexural compressive strength. The latter was assumed, as in Ref. 9, to be equal to $f'_m$. A linear stress approximation to the stress block was assumed. Figure 2.2 shows the interaction curve for a short prismatic wall.

The authors also derived the interaction equations for symmetrical and asymmetrical hollow sections.

The moment magnifier method, was used by the authors to account for the slenderness effect. The equivalent stiffness was assumed as $EI = E_i I_n (0.2 + P/P_o) < 0.7 E_i I_n$ and also as $EI = E_i I_n /3$ (the latter giving more conservative results). The top of the walls was assumed to be pinned while the bottom rested on fibreboard. With these end conditions it was assumed that the effective length of the wall ($k_l$) was 0.8h and that the resulting moment at the base was equal to the maximum moment along the span and equal to 68 percent of the moment that would be obtained in the case of a pinned connection at the bottom of the wall.

In order to make a meaningful comparison between the interaction curve (predicted for short walls) and the strength of a slender wall, the moment attributable to deflections, approximated by the axial load times the measured maximum deflection, was added to primary moments. Reduced interaction curves were drawn using the moment magnifier method. See Figure 2.3.

Yokel et al., concluded that the trend of the relationship between vertical loads and moments was correctly predicted, and that the order of magnitude of the observed added moment due to slenderness effects showed fairly good agreement with the predicted values calculated using the moment magnifier method.
In 1976 Fattal and Cattaneo (Ref. 11) reported a program aimed at continuing the work started by Yokel et al., (Refs. 9 and 10). Tests were carried out on prisms and walls, built of 4 in. (100 mm) bricks, 6 in. (150 mm) concrete blocks and 10 in. (250 mm) composite walls. The tests included 95 prisms and 56 walls. The prisms were subjected to vertical compressive loads at various equal top and bottom eccentricities. The walls were tested under various combinations of transverse and vertical load. The test set-up was the same as described by Yokel et al, (Refs. 9 and 10). All the walls were constructed in running bond to a height of approximately 8 ft. (2.44 m) and were nominally 32 in. (0.81 m) wide.

The observed stress-strain relationships were fairly linear with a maximum strain at failure ranging between 0.001 and 0.002.

The authors showed that the empirical relationship for the flexural rigidity proposed in Ref. 10 (\( EI = E_I (0.2 + P/P_o) < 0.7 E_I \)) produced an interaction diagram that gave predicted values which were consistent with experimental results for the 3 types of masonry walls considered in their program.

As in previous works (Ref. 9 and 10) the authors found that the compressive strength in flexure (\( f'_m \)) exceeds the compressive strength (\( f'_m \)) developed in axial compression. Within the range of eccentricities used (\( t/12 < e < t/3 \)) the average value of the coefficient "a" for concrete blocks was found to be 1.27 with little variation. Thus the hypothesis advanced in the previous studies (Refs. 9 and 10) that the compressive strength increases with flexural strain gradient was, according to the authors, not confirmed. The question remains of what happens in the range \( 0 < e < t/12 \).
The authors concluded that in the case of walls, where flexure was induced by lateral loads singly or in combination with eccentrically applied compressive loads, the agreement between theory (using the moment magnifier method) and experiments constituted a generalization of the basic theory proposed in Refs. 9 and 10.

In 1980, R.H. Brown and F.N. Wattar (Ref. 12) presented the results of an analytical program that reviewed the results of the 252 tests reported by Refs. 9, 10 and 11.

The authors developed failure relationships between applied axial load and bending moment assuming:

a) The stress-strain curve is linear

b) Failure takes place at a compressive strain of 0.002

c) The ratio $f'/f'_m$ is equal to 0.02 ($f'_m$ = flexural tensile strength of masonry)

The equations derived in Ref. 10 were used to develop the interaction curves for solid prismatic rectangular sections. A numerical analysis method was used to define the interaction curve for hollow prismatic rectangular sections. A minimum eccentricity of t/12 was used to account for imperfections.

The maximum or magnified moment at failure of the 252 tests were superimposed with the short wall interaction curve. The magnified moments were obtained by multiplying the applied moments by the moment magnifier factor: $C_m/(1-P/P_{cr})$. The safety factor was defined by the ratio between the magnified moment and the moment capacity given by the short wall interaction diagram at the same axial load level.

A statistical study was performed and after a log transformation the
factor of safety fitted a nearly normal distribution. Only 4 walls (all hollow concrete block walls) would have been overestimated by the moment magnifier method. The confidence with which this method can be applied without any additional load factors, is shown by the 99.99% probability (almost certainty) of obtaining a safety factor exceeding 1.0 (some walls have a safety factor greater than 10.0). The authors also concluded that the factor of safety was independent of the eccentricity.

Hatzinikolas et al., (Refs. 13, 14, 15, 16 and 17) have carried out an extensive program studying the behaviour of eccentrically loaded walls. They also used the moment magnifier method as a means to account for the slenderness effect.

In order to obtain a value of $P_{cr}$ required for the evaluation of the magnification factor, they approximated the solution given by Yokal (Ref. 18) for walls without tensile strength and in single curvature by:

$$P_{cr} = 8\pi^2 (0.5 - \frac{e}{t})^3 \frac{E I_o}{h^2} (I = \text{uncracked moment of inertia}) \quad \text{(Refs. 13, 15 and 16)}.$$  

If some tensile strength ($f'_t$) was to be considered: $P_{cr} = 8\pi^2 (0.5 - \frac{e}{t} + \frac{\xi}{2t})^3 \frac{E I_o}{h^2}$ where $\xi$ is the distance from the point of zero stress to the end of the crack. For the case in which the wall is subjected to unequal end eccentricities producing single curvature, the writers recommend the use of the average $e$ value.

A method for determining the buckling load for the case of double curvature was also developed (Refs. 13 and 15). The authors pointed out that in this case buckling will also tend to occur in the primary single loop. This behaviour was substantiated by test results. The critical buckling load would be: $P_{cr} = \lambda E I_o / h^2$ where $\lambda$ is a buckling coefficient depending on the end eccentricities (Ref. 15).
The buckling load for reinforced masonry walls in single curvature with \( e/t < 1/3 \) can be evaluated as for plain masonry. For larger values of \( e/t \) the behaviour will be a function of the extent of cracking and the transformed moment of inertia. A lower limit of \( EI_o (1/2 - e/t) > EI/o /10 \) is recommended for the flexural stiffness. The analysis of reinforced walls in double curvature can be carried out similarly to the analysis of plain masonry walls.

The authors found that the experimental results were in fairly good agreement with the analytical values.

In 1980, Ojinaga and Turkstra (Ref. 19) reviewed some of the limitations of the moment magnifier method. A second approach, commonly termed the P-\( \Delta \) method, in which moments due to axial forces acting through deflections are combined with elementary bending moments and then compared to the section capacity, was investigated for unreinforced brick and concrete block masonry.

As in the previous works a linear strain-stress relationship was adopted. For hollow concrete blocks the initial tangent modulus of elasticity was assumed to be: \( E = 440 f'_m + 73600 \text{ psi} < 3000000 \text{ psi} \) based on the results of 52 tests.

Interaction curves for short columns were calculated using the equations developed by Yokel et al, (Ref. 10). A maximum flexural compressive strength equal to \( f'_m \) was assumed.

The moment magnifier theory was first studied. The magnification factor was defined as before (Refs. 9, 10, 11) and in defining \( Pcr = \frac{H^2EI}{m(kh)^2} \), a rigidity reduction factor \( m \) equal to 2.5 was used. This theory was compared with experimental data and found to have some
limitations, the major one being a tendency to be unconservative for single curvature bending under relatively large eccentricities. The authors concluded that cracking across the section and along the height of the wall could not be modelled by simply adjusting the rigidity reduction factor, m. When compared to a theoretical "exact" analysis assuming linear stress-strain, cracked regions and a moment curvature approach described by Sahlin (Ref. 4) the moment magnifier method appeared to be conservative for small eccentricities but become unconservative for large eccentricities. Although the use of a rigidity reduction factor varying with load level could reduce these problems, this solution could lead to practical problems in design.

The authors introduced their beam-column analysis in which lateral deflections due to end moments, transverse loads, and P-Δ effects are computed. The maximum moment along the span is found taking all those contributions into account. Axial load and maximum bending moment are then compared with the short wall strength. Successful application of such an analysis requires an estimate of the effective rigidity $E I_{\text{eff}}$. The initial tangent modulus can be estimated by best fit equations as the one presented above. $I_{\text{eff}}$ must consider the uncracked section as well as the end inertias for either cracked or uncracked conditions. The authors recommended a general approximation as follows:

(a) $I_{\text{eff}} = (I_{\text{end1}} + I_{\text{end2}})/4$ for $0 < e_1/e_2 < +1$

(b) $I_{\text{eff}} = \min \left\{ (I_{\text{end1}} + I)/4 \right\}$ for $-1 < e_1/e_2 < 0$

where $I_{\text{end1}}$, $I_{\text{end2}}$, and $I$ are the two end inertias (which could be of
cracked sections) and the uncracked section inertia, respectively and $e_1$, $e_2$ are the end eccentricities. The effective length concept is replaced by consideration of applied end moments and estimated deflected shape. In their analysis, the writers considered only the elastic deflections due to primary bending moments, which caused underestimation of deflections but avoided iterative solutions. This type of analysis will not predict buckling loads nor does it give a good estimation of the maximum moments for small eccentricities, thus a limiting slenderness ratio and minimum eccentricities were imposed. In order to evaluate the approximation implicit in the approach and establish reasonable limits of applicability, the ratio of failure load to axial capacity vs. the slenderness ratio for different eccentricities was plotted against the values obtained using the Sahlin method (Ref. 4). As a result of this comparison a minimum eccentricity of $t/12$ and a maximum slenderness ratio of 80 are recommended (for all eccentricities $> t/12$).

The authors recommended, in the case of no lateral loads and short walls, the use of the short wall capacity, without calculation of lateral deflections and slenderness effects, within the following limits:

(a) $L/r = 35 - 17.5 \frac{e_1}{e_2}$ for $0.0 < \frac{e_1}{e_2} < 1.0$

(b) $L/r = 35 - 35 \frac{e_1}{e_2}$ for $-1.0 < \frac{e_1}{e_2} < 0.0$

where $L$ is the wall height and $r$ the radius of gyration.

Comparison was also made with experimental data. In this case a minimum eccentricity of $0.01t$ was included, rather than the minimum of $t/12$ recommended for design, because of the well controlled conditions. On average, the analysis was found to be conservative.
A comparison is made also with the procedures outlined in the present Canadian Code (Ref. 20). It was found that the present code permits relatively higher theoretical strengths at low slenderness. In general the proposed approach may be more or less conservative than the code depending on the slenderness ratio, $e_1/e_2$ ratio and maximum eccentricities.
Loading Conditions and Moment Distribution Assumed in Ref. 9.

Figure 2.1
Interaction Curve for Short Prismatic Wall

Figure 2.2

\[ M = \frac{(P_o - P)}{t} \]

Cracking line:

\[ M = \left( sP_o + P \right) \frac{t}{6} \]

\[ M = \frac{Pt}{2} \left\{ 1 - 1.33 \frac{P}{aP_o} \left[ \frac{a - 2s}{(a - s)^2} \right] \right\} \]

\[ = \frac{Pt}{2} \left\{ 1 - 1.33 \frac{P}{P_o} \left[ \frac{1 - 2s}{(1 - s)^2} \right] \right\}, \text{ for } a = 1 \]

\[ P_o = \frac{f'}{b}t \]

\[ M_k = \frac{P_o t}{12} \]

\[ s = \frac{f_t}{f_m} \]
The solid bar represents the added moment due to the horizontal deflection, i.e., the left end represents the maximum moment excluding the effect of the vertical load acting on the horizontal deflection and the right hand end represents the total maximum moment at failure. The right hand end of the solid bar should be compared to the solid line interaction curve. The left hand end should be compared to the reduced interaction curve (broken line).

Figure 2.3 P-M Interaction Curve
CHAPTER 3
CURRENT ANALYSIS METHODS AND CODE APPROACH

It is intended in this chapter to present a brief discussion of those concepts which form the basis for the analysis and design of plain and reinforced masonry.

Masonry structures are, at present, designed by working stress analysis. The concepts involved here are those of the elastic theory, long applied in designing reinforced concrete elements, but in this case the ultimate uniaxial strength of the masonry \( f' \) is used in the analysis formulas to reflect the properties of masonry. The determination of masonry properties is discussed extensively in the next chapter.

The decision whether to use plain or reinforced masonry depends on the magnitude of the loads as well as minimum requirements established by the codes.

3.1 Analysis of Walls Under Combined Bending and Axial Loads

3.1.1 Plain Masonry

The load carrying capacity of a plain masonry wall subjected to combined bending and axial loads can be determined if the tensile and compressive strength of the masonry, as well as the stress distribution at failure, are known. It is usually assumed that a linear stress-strain relationship exists up to failure, thus, the stresses can be determined for a given loading. This type of failure can be illustrated by the following examples:
26.

(a) For non-cracked sections the stresses at the outer faces will be

\[ f = \frac{P}{A} \pm \frac{Mt}{2I} \]  \hspace{1cm} (1)

where

- \( A \) = area
- \( I \) = moment of inertia of the non-cracked section
- \( P \) = applied load
- \( M \) = applied moment
- \( t \) = thickness of gross section.

If the tensile strength of the assemblage is assumed to be zero, Equation (1) is valid for loads applied within the kern, i.e. for

\[ e < e_k \] \hspace{1cm} (2)

where

\[ e_k = \text{kern eccentricity} = \frac{2I}{A t} \text{ (for a symmetrical section) and} \]

\[ e = \frac{M}{P} \] \hspace{1cm} (3)

(b) For vertical loads at eccentricities falling outside the kern but within the thickness, and for assumed zero tensile strength, the stress in a solid section would be:

\[ f_{\text{max}} = \frac{4}{3} \frac{P}{A} \left[ \frac{1}{1-2e/t} \right] \] \hspace{1cm} (4)

(c) In some cases some tensile strength can be attributed to the masonry, and the capacity of a solid section is then given by:

\[ P = \frac{bu}{2} \left( f_{\text{max}} - f'_t \right) \] \hspace{1cm} (5)

and

\[ M = \frac{Pt}{2} \left[ 1 - 1.33 \frac{P}{P_o} \left[ \frac{a-2s}{(a-s)^2} \right] \right] \] \hspace{1cm} (6)

where,

- \( u \) = length of uncracked section
- \( b \) = width of section
f_{\text{max}} = \text{maximum compressive stress} = a \ f'_{m}

f'_t = \text{tensile strength} = s \ f'_{m}

P_o = f'_m \ bt

For masonry with no tensile strength \( s = 0 \) and Equation (6) reduces to Equation (4).

Similar equations to the ones presented above can be derived for hollow symmetrical sections. It is very difficult to find a continuous equation for the moment capacity of a cracked section that will be applicable to all hollow symmetrical sections because of the discontinuities in the sections.

3.1.2 Elastic Analysis of Reinforced Masonry Walls

Plain masonry walls are in some cases inadequate in strength, stiffness or ductility, therefore the walls have to be reinforced. If an elastic analysis is used then the usual assumptions are made.

There are two cases to be considered when analyzing reinforced masonry walls: (a) the effective eccentricity \( (M/P) \) is so small that no tensile stress is developed in the section, (b) the eccentricity is sufficient to produce a cracked section.

3.1.2(a) Uncracked Section

In this case the effect of the reinforcement is not significant. Therefore, for a solid section,

\[
P = bt \ f_{av}, \quad \text{where} \quad f_{av} = \frac{f_{\text{max}} + f_{\text{min}}}{2}
\]

\[
M = \left( \frac{bt^2}{12} \right) \left( \frac{f_{\text{max}} - f_{\text{min}}}{2} \right)
\]
3.1.2(b) Cracked Section

3.1.2(b.1) Steel in Compression

In this case, as in the previous one the steel can be ignored, then:

\[ P = \frac{1}{4} f_{\text{max}} b p_t \]  
\[ M = \frac{1}{4} f_{\text{max}} b p_t \left[ \frac{t}{2} - p_6 t \right] \]

where \( p_t/2 \) is the length of the compression zone, and \( 1.0 < p < 2.0 \).

3.1.2(b.2) Steel in Tension

In this case from basic equilibrium considerations the following equations are obtained:

\[ \frac{f_s}{n} = f_{\text{max}} \frac{1-p}{p} \]  
where \( n = \frac{E_{\text{steel}}}{E_{\text{masonry}}} \)

and for the steel placed at the centre of the section,

\[ P = f_{\text{max}} \frac{(P_t^2)}{2} b - A_s f_s \]  
\[ M = f_{\text{max}} \frac{p b t^2}{24} (3-p) \]

where \( p_t/2 \) = length of the compression zone, and \( 0.0 < p < 1.0 \).

3.1.3 Ultimate Strength Analysis

As in the case of elastic analysis, the principles of ultimate strength design for reinforced concrete elements can be used to analyze reinforced masonry.

In this case the familiar \( f_c' \), uniaxial compressive strength of the concrete, is replaced by \( f_m' \), the uniaxial compressive strength of the masonry assemblage (the grout and masonry block are not considered as two different materials). The compression on the masonry is given by a
rectangular stress block whose magnitude is $0.85 f'$.

3.2 Code Design Procedures

In the following sections the procedures for designing masonry walls under different codes of practice are examined.

3.2.1 Loadbearing Walls

3.2.1(a) The ACI (Ref. 21), NCMA (Ref. 22) and UBC (Ref. 23, only for Reinforced Masonry) Codes

In the new ACI recommendations, (Ref. 21) the equations have been developed to bring non-reinforced and reinforced concrete masonry into one formula which is applicable to both cases.

Experience and tests indicated that strength of masonry can be significantly reduced by poor workmanship and the use of uncontrolled materials. According to the ACI and UBC recommendations (Refs. 21 and 23) the designer may use full allowable stress values if the construction is inspected, but must use reduced stress values if there is no inspection. ACI-531 recommends, for cases where there is no inspection, reduced allowable stresses of $2/3 \sigma_{\text{compr.}}$, $1/2 \sigma_{\text{tens.}}$ and $1/2 \tau_{\text{shear}}$. The UBC code recommends a reduction of the allowable stress by one half for all the different types of stress.

In the ACI provisions, there is an increase of one-third in the allowable values for stresses due to wind or earthquakes combined with dead and live loads, provided the strength of the member is not less than that required for dead and live loads alone.

The 1968 NCMA (Ref. 22) does not allow tensile stresses in unreinforced masonry walls built with hollow units, therefore limiting the
eccentricity to be within the kern. For solid unreinforced walls and for reinforced masonry, cracked sections are allowed. It is also stated that up to an eccentricity of t/3 reinforced walls may be designed assuming uncracked sections.

The slenderness reduction factor $C_s$ used by these codes is given by:

$$C_s = \left[1 - \left(\frac{h}{40t}\right)^3\right]$$

where $h$ is the height of the wall.

Equation (14) does not differentiate between solid and hollow sections. Other variables associated with slenderness effects and not considered in these design equations are: end fixity (effective length), the manner in which the member is loaded (ie. the moment diagram and the resultant deflection curve) and the relationship between the strength and the modulus of elasticity of masonry. According to Yokel et al. (Ref. 10) the justification for not considering some of these variables may be in part attributed to the fact that there is a linear relationship between $f'_m$ and $E$ within a certain range of masonry strength, and the end conditions are similar for most conventional masonry structures. It is questionable whether, with the increasing use of high strength masonry and of high rise masonry construction, it is still possible to disregard these variables without the use of unduly high margins of safety.

The Codes being discussed (Refs. 21, 22, 23) recommend that members subjected to combined axial and flexural stresses shall be designed according to the following interaction equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$$

(15)
where  
\[ f_a = \text{computed axial stress} \]
\[ f_b = \text{computed compressive bending stress} \]
\[ F_a = \text{maximum allowable axial stress, including any stress increase permitted for wind and earthquake and reduced for slenderness effect by the slenderness coefficient presented above (Equation 14)} \]
\[ F_b = \text{maximum allowable flexural stress, including any stress increase permitted for wind and earthquake.} \]

Yokel et al. (Ref. 9) compared test results with allowable stress interaction curves calculated using the code procedure as given above. The margin of safety was computed in two ways: (a) the ratio of average experimental ultimate loads to allowable loads for specific load eccentricities and (b) the ratio of ultimate to allowable moments for specific levels of vertical loads. In both cases, an increase in the margin of safety with increasing slenderness was apparent. The second method, showed margins of safety decreasing with increasing eccentricity. The authors (Yokel et al.) commented that for eccentricities greater than \( t/6 \) the margin of safety against an increase in lateral loads (i.e. moving horizontally on the interaction diagram) was significantly smaller than that provided against an increase in vertical loads.

In Ref. 10, Yokel et al. derived wall interaction curves for ultimate stresses following the principles established by the Code (Equation 15). Ultimate stresses were obtained by multiplying the code allowable stress by 5, which according to the authors can be considered the axial load margin of safety (see also Ref. 9). Those curves were modified for slenderness using Equations (14) and (15).
The code curves were compared to test results and other interaction curves derived using the moment magnifier method. For the slenderness of the walls tested (h/t = 13), the modification of the interaction curves was relatively minor, thus curves for h/t of 30 were constructed to provide a better comparison between the Code approach and the moment magnifier method. For both slenderness ratios (h/t = 13 and h/t = 30) it appeared that the moment magnifier curve was more conservative than the code curve, except for very small eccentricities. For a small slenderness ratio (i.e. h/t = 13) the code curve produced a better fit to the experimental results than the moment magnifier curve. In the case of greater slenderness (h/t = 30), although no slender concrete block walls were tested, Yokel et al. concluded, based on the agreement between observed and predicted strength of more slender brick walls, that the code curve probably overestimates the transverse strength of transversely loaded slender walls, but it is probably conservative for the case of eccentric vertical loads.

3.2.1(b) Commentaries on part 4 of the 1975-NBC (Ref. 24), CSA-CAN3-S304 (Ref. 20), 1969 SCPI: Building Code Requirements for Engineered Brick Masonry (Ref. 25); UBC Code (Ref. 23, only for Unreinforced Masonry).

The UBC and SCPI codes prescribe the use of reduced allowable stresses for cases where there is no acceptable inspection. CSA-CAN3-S304 and the 1975 NBC commentary do not allow uninspected construction.

The UBC and SCPI codes establish that the allowable vertical load for plain sections with \( e < t/3 \) is given by:

\[
P = C_c f_s A_e s m g
\]
where $f_m$ is the allowable axial compression stress, $A_g$ the gross cross
sectional area, $C_e$ an eccentricity factor and $C_s$ a slenderness factor.
CAN3-S304 uses the net area $A_n$ instead of $A_g$. The coefficient $C_e$ is deter-
mmed by the following equations:

$$C_e = 1.0 \quad \text{For } e/t < 1/20$$

$$C = \frac{1.3}{e} \left[ \frac{1}{2} \left( \frac{e}{t} - \frac{1}{20} \right) \left( 1 - \frac{e}{e_1} \right) \right] \quad 1/20 < e/t < 1/6$$

$$C_e = 1.95 \left( \frac{1}{2} - \frac{e}{t} \right) + \frac{1}{2} \left( \frac{e}{t} - \frac{1}{20} \right) \left( 1 - \frac{e}{e_2} \right) \quad 1/6 < e/t < 1/3$$

where $e = \text{maximum vertical eccentricity}$

$e_1 = \text{smaller end eccentricity}$

$e_2 = \text{larger end eccentricity}$

$t = \text{wall thickness}$.

For some conditions of loading and methods of support, the maximum
vertical eccentricity ($e$) may occur at a location other than the point of
support. The vertical eccentricity may, therefore, differ from the eccen-
tricities used for the ratio of end eccentricities ($e_1/e_2$). The UBC and
SPCI specifications prescribed a ratio $e_1/e_2$ equal to +1.0 for members
subjected to lateral loads higher than 10 psf. No lateral loads effects
are taken into consideration by CAN3-S304.

The slenderness factor $C_s$ is given by:

$$C_s = 1.2 - \frac{h/t}{300} \left[ 5.75 + \left( 1.5 + \frac{e_1}{e_2} \right)^2 \right] < 1.0$$

This factor (similar to the one presented in section (a) equation 14) does
not explicitly take into consideration some of the variables which influ-
ence the slenderness effect, e.g.: the load level and loading conditions,
variation in the moment of inertia due to cracking, the end conditions, and the \(\frac{f'_m}{E}\) ratio.

Loads applied at eccentricities greater than \(t/3\) are limited by the allowable flexural tensile stress. When this allowable stress is exceeded the section has to be reinforced (Refs. 20, 23, 25).

The recommendations presented in the Commentary to Part 4 of the 1975 NBC (Ref. 24) show a similar approach as the one stated above except that \(A_g\) is replaced by the area \(A_n\) in Equation (16). The slenderness and eccentricity coefficients are also slightly different and given by:

\[
C_s = 1 - C_b \left(\frac{h}{t} - 5\right)
\]

where

\[
C_b = 0.003 \left(\frac{e_1}{e_2}\right)^2 + 0.012 \left(\frac{e_1}{e_2}\right) + 0.025
\]

and

\[
C_e = \begin{cases} 
1.0 & \text{for } e/t < 1/20 \\
\frac{1}{1 + 6\frac{e}{t}} & \text{for } 1/20 < e/t < 1/6 \\
\frac{3}{4} (1 - 2\frac{e}{t}) & \text{for } 1/6 < e/t < 1/3 \\
\frac{1}{6 \frac{e}{t} - 1} & \text{for } 1/3 < e/t
\end{cases}
\]

For \(e/t > 1/3\), \(f_m\) in Equation (16) is replaced by the allowable tensile stress \(f_t\).

For reinforced masonry, use is made of the same equations as for unreinforced masonry, but in this case the 1975 NBC Commentary increases the eccentricity for which bending is ignored from \(t/20\) to \(t/10\). For eccentricities greater than \(t/3\) or a value producing tensile stresses in the steel bars, the axial load capacity \(P\) is to be taken as the short wall
capacity, calculated on the basis of a transformed section and a linear stress distribution, reduced by the slenderness coefficient $C_s$ defined above.

For reinforced masonry, the SPCI and CAN3-S304 Codes present a similar approach as the one described above, except that the definition of the coefficient $C_e$ is the same as that of plain masonry (for $e/t < 1/3$).

All the above codes approach the case of biaxial bending in the same way as they do for uniaxial bending where $e/t$ is replaced by $(b e_t + t e_e)/b t$.

In Ref. 10, Yokel et al. compared the results of tests on laterally loaded brick walls with an interaction curve derived using the moment magnifier method (using $C_m = 1.0$, $k = 0.8$), and also with an interaction curve based on the preliminary 1969 SCPI Standard where the ultimate loads were taken as $C C_s f' A$. The code curve was developed for eccentric vertical load with $e_1/e_2 = -0.4$, which assumes partial fixity at one end and a pinned condition at the other end. This curve predicted axial load capacities that were in good agreement with test results and the moment magnifier curve except for small values of $P$, where there was a considerable difference in the moment capacities due to the differences in loading and end conditions between Yokel's test program and those assumed by the 1969 SPCI code (see Figure 3.1). When Yokel et al. changed to the conditions used in the SPCI code (i.e. $C_m = 0.5$ and $k = 0.8$, see Figure 3.1), the moment magnifier method for eccentric vertical loads agreed approximately with the SPCI curve.

The original SPCI recommendations were developed on the basis of tests
on walls loaded with eccentric vertical loads only, and so they were not necessarily appropriate for the case where the wall is also loaded laterally. The case of lateral loading was recognized in the 1969 SPCI Standard (see Ref. 25, Section 4.7.6.2(5)(c)) as a result of the investigation of Yokel et al. (Ref. 10). Based on test data, the new SPCI code recommends that for walls subjected to combined vertical and transverse loading, $C_e$ should be calculated using an $e_1/e_2$ value of +1.0, but $C_s$ and the maximum slenderness ratio should be calculated on the basis of the actual value of $e_1/e_2$.

Consideration to lateral loads is also given in the UBC Code (same recommendations as SPCI Code), but not in the Canadian Codes (Refs. 20 and 21).

3.2.1(c) Minimum Thickness for Loadbearing Walls

The ACI Code (Ref. 21) establishes a minimum thickness for masonry walls of 1/36 times the least distance between lateral supports either horizontal or vertical. For non-reinforced bearing walls the ratio is increased to 1/20.

The UBC Code limits the sizes of various masonry walls in the form of a specified minimum wall thickness (e.g.: 6 in. for reinforced walls) and a maximum unsupported height to thickness ratio (e.g.: $h/t = 25$ for reinforced hollow masonry). Amrhein (Ref. 26) explains that the limiting value of $h/t = 25$ for reinforced grouted or hollow masonry, was established in order to limit the tension on a mortar joint to 50 psi when the wall is subjected to a wind load of 15 psf, with no account taken of the self-weight of the wall. The same explanation is presented by Schneider and Dickey (Ref.
According to the UBC recommendations, the h/t ratio may be increased and the minimum thickness decreased "when data is submitted which justifies a reduction in the requirements specified", for example, making some provision for end restraints of the wall.

In the Canadian Codes (CAN3-S304 and 1975 NBC) minimum requirements are given as part of the "Empirical Rules for Plain Masonry Design not Based on Engineering Analysis". For unreinforced masonry it is recommended: (1) h/t < 18 for solid masonry made of hollow units and (2) h/t < 36 for partition walls.

All the codes (for UBC this applies only to unreinforced masonry) set an upper bound for the slenderness ratio of a load bearing wall: h/t should not exceed 10(3 - e₁/e₂) where (e₁/e₂) > 0 for single curvature and (e₁/e₂) = 0 when e₁ and e₂ are zero. The UBC Code (Ref. 23) and the SPCI Standard (Ref. 25) stipulate that when the walls meet all other requirements, the designer may present a written justification in order to disregard the limit on the slenderness.

3.2.1(d) Minimum Reinforcement for Loadbearing Walls

In the planning of a building, it is essential to determine whether the masonry should be plain or reinforced. Part 4 of the NBC (Ref. 28) dictates that in seismic zones 2 and 3, masonry must be reinforced in load-bearing and lateral load-resisting structural elements, in walls enclosing elevator shafts and stairways, in exterior cladding and in certain categories of partitions. Only in zones 0 and 1 does the designer have the option to select either plain or reinforced masonry. Thus, in CAN3-S304, since all reinforced masonry is subjected to design by engineering analysis the use of the conventional rules for plain masonry (Section 5) is restricted
to masonry in buildings in zones 0 and 1.

All codes discussed in this chapter require a total minimum steel area of 0.002 times the gross cross sectional area of the wall and that not less than a third of it be placed either horizontally or vertically.

The maximum spacing between bars is set by the UBC Code at 4 feet (1.22 m). The 1969 SPCI, 1975 NBC Commentary and S304 have the same upper limit but add that it should not exceed 6 times the thickness of the wall. According to ACI-531 the spacing between bars shall not exceed 12 times the wall thickness nor 8 feet (2.44 m) on centers in each direction. The maximum horizontal steel spacing is reduced to 4 feet on centers when the wall is fully grouted. The ACI standard also requires that the bound area between vertical and horizontal bars should not exceed 32 square feet (~3 m²), and when the wall is subjected to large loadings or movements it is recommended that the size of the bound area be reduced to 20 square feet (~1.9 m²).

CAN3-S304 and the UBC Code, allow joint reinforcement (wire reinforcement) to be considered as required horizontal reinforcement.

Amrhein (Ref. 26) explains that many of the requirements that were used for reinforced concrete were revised and applied to reinforced masonry. Minimum reinforcing steel is placed in concrete to compensate for the shrinkage, moisture changes, and temperature stresses that develop not only during the life of the structure, but also during the initial placing and curing of concrete. In masonry the units have already experienced most of the shrinkage that will occur before the wall is constructed. Mortar and grout do shrink but comprise only about one-half of the volume of the wall even if fully grouted. Therefore, only half or less of the material that shrinks is used and for this reason the codes usually require only
about half as much minimum reinforcement in masonry as in concrete.

3.2.2 Non-Loadbearing Walls

Non-loadbearing walls are walls which support no vertical load other than their own weight. This classification may, therefore, include earth retaining walls and walls of hydraulic structures, as well as interior and exterior non-loadbearing walls of buildings. When due to the magnitude of the applied load or to the method of support, non-loadbearing walls require reinforcing steel, they should be designed in accordance to the Codes' guidelines for the design of flexural members.

3.2.2(a) Minimum Thickness For Non-Loadbearing Walls

The 1969 SCPI standard provides lateral slenderness guidelines for non-loadbearing, non-reinforced brick masonry walls, subjected to various lateral loads. The recommendations are based on the assumption that the wall is simply supported and contains no openings or other interruptions. The analysis neglects the weight of the wall and the allowable stresses are increased by 33.3% for wind. Provisions are made also for walls supported in two directions; in that case the distance between supports can be increased but the sum of the horizontal and vertical span cannot exceed 3 times the allowable distance permitted for support in one direction.

The UBC Code also gives a ratio of unsupported height or length to thickness with a minimum thickness requirement of 2 inches (5 cm).

3.2.2(b) Minimum Reinforcement for Non-Loadbearing Walls

The UBC Standard and 1975 NBC recommendations do not differentiate between loadbearing walls and non-loadbearing walls when establishing the minimum steel requirements. In the "Recommended Practice for Engineered Brick Masonry" of the SCPI Standards (Ref. 25) it is required that non-
loadbearing walls should be designed in accordance with the flexural design of reinforced brick masonry. The minimum steel ratio for flexural members should be not less than $\frac{80}{f_y}$, unless the reinforcement provided at every section is at least one-third greater than that required by analysis, where $f_y$ is the steel yield stress in psi. CSA Standard CAN3-S304-M78 establishes that for non-loadbearing walls when reinforcement is required, it should be provided in one or more directions with reinforcing steel having a minimum area of $0.0005 \frac{A}{g}$ in seismic zone 0, 1 and 2 and $0.001 \frac{A}{g}$ in seismic zone 3. The maximum spacing for one way reinforcement is 16 inches (400 mm) whereas if at least one-third of the steel is placed in a second direction the maximum spacing can be increased to 4 feet (1.22 m). This reduces the minimum requirements for non-loadbearing walls in comparison with the requirements established in the NBC 1975 and the UBC codes; it has a more rational approach and takes into account the different seismic zones. The designer must ensure that the reinforcement is adequate to resist the design seismic forces. Joint reinforcement can be used to meet the minimum requirements.
Note: Case (a) when using the moment magnifier method will have $C_m = 1.0$ and $k = 0.8$. Case (b) will have $C_m = 0.5$ and $k = 0.8$.

End and Loading Conditions Assumed by (a)
Yokel's Series of Tests in Ref. 10
and (b) 1969 SCPI Code

Figure 3.1
In this chapter the properties of the individual materials and of composite masonry assemblages are studied both theoretically and experimentally. All the materials used in the construction of the test specimens at the University of British Columbia are commercially available and typical of those commonly used in concrete masonry construction.

4.1 Concrete Block Units

Concrete blocks are classified as hollow or solid units. A hollow unit is defined as one in which the net area is less than 75% of the gross cross sectional area. The net cross sectional area of most concrete units ranges from 50% to 70% depending on the unit width, face shell, web thickness, and core configuration. Because of their lighter weight and easier handling, hollow units are more popular than solid units.

For structural reasons some standards require a minimum face shell and web thickness (e.g.: ASTM: C-90, CSA 165). Concrete unit dimensions are usually based on modules of 4 or 8 in. (100 or 200 mm). From common usage the 3/8 in. (~10 mm) thick mortar joint has become standard, therefore the exterior dimensions of modular units are reduced by the thickness of one mortar joint. The nominal block size that dominates the industry is 8 x 8 x 16 in. (200 x 200 x 400 mm).

The majority of concrete block units produced in Canada are classed as Concrete Masonry Units under CSA A165.1 (Ref. 29), and some are also manufactured to conform the requirements of the American Society for Testing and Materials: ASTM C-90 (Ref. 30). CSA Standard A165.1 "Concrete Masonry
Units", classifies masonry units, except concrete bricks, by their physical properties using the four-facet system. For instance, H/1000/C/0 is a hollow unit with a strength of 1000 psi (average of 5 units), a density of less than 105 pcf and a undefined moisture content at the time of shipment. The standard does not fix the weight, colour, surface texture, fire resistance, thermal transmission or acoustical properties of the blocks. In the CSA specifications certain type of blocks are excluded for exterior use.

ASTM specifications classify concrete masonry units according to grade and type. The grade describes the intended use of the concrete masonry units, while the type refers to the moisture control of the unit: type I moisture controlled, type II non-moisture controlled. Similarly to the CSA standard, the ASTM does not establish the required weight, colour, surface texture, fire resistance, thermal transmission or accoustical properties of the units.

4.1.1 Compressive Strength of Masonry Blocks

The determination of the block compressive strength is one of the most difficult aspects of testing masonry units. It is also very important because many masonry codes allow the use of a formula to calculate the design value of masonry compressive strength ($f'_m$), and this formula uses the uniaxial compressive strength of the masonry unit.

Normally the blocks are capped before testing and are subjected to compressive stress in a standard testing machine, with the capping directly
against the steel surface of the testing machine.

J.J. Roberts (Ref. 31) reported a program in which one of the aims was to compare various methods of capping; single block specimens consisted of 4 types:

1. mortar-capped blocks tested wet
2. " " " dry
3. board-capped blocks tested wet
4. " " " dry.

Board-capping was used in the tests because it has several advantages over mortar capping: it is easier, quicker and cheaper. After using five types of fibre-board the authors concluded that the type of fibre-board had little effect upon the mean indicated block strength; four types gave similarly consistent results but one case yielded a somewhat larger coefficient of variation. Board-capped specimens produced a lower indicated strength than mortar-capped specimens.

In 1980, W. Ridinger et al. (Ref. 32) presented the results of an investigation that was primarily designed to evaluate the influence of capping, loading conditions and the influence of reduced interface friction upon the results of uniaxial compressive tests of hollow clay units.

The authors tried to reduce the interface friction by using an interface assembly consisting of two layers of polyethylene plastic (4 mil), separated by a thin layer of high viscosity lubricant. In order to consider the influence of the loaded area, some specimens were capped only in the face shell and in others the capping was applied to the net area of the unit (current practice). For specimens loaded over the entire net area with full interface lateral restraint, the familiar pyramidal mode of
failure, typical of shear failure, was observed. The units loaded on the face shells only, with full interface restraint, showed evidence of horizontal compressive forces within the cross-webs, together with spalling of the outer portions of the face shells. Units tested with reduced interface friction seemed to expand more freely at their upper and lower boundaries, resulting in vertical tension cracking. For face shell loaded specimens with reduced interface friction, a physical separation of the face shells from the cross-webs was also observed, as well as localized splitting within the face shells themselves. The visual resemblance of the failed specimens to the prism and wall failures suggests that this test may be a more accurate indication of unit compressive capacity than the current standard test. The unit compressive strength based on these tests was between 73 and 78% of the unit compressive strength based on the normal test where the ends are restrained somewhat. Since the latter test is the current industry standard, present compressive strength tests of hollow clay masonry units may give erroneously high impressions of in-place compressive capacity.

4.1.1(a) Block Compressive Strength Tests at the University of British Columbia.

The basic units used for constructing all test specimen walls were the 8 x 8 x 16 in. (200 x 200 x 400 mm) stretcher block, the 8 x 8 x 16 in. end block, the 8 x 8 x 16 in. knock out block, and the 8 x 8 x 8 in. half block. All the units were manufactured by Ocean Construction Supplies Ltd. of Vancouver.

Block compressive strength test specimens were selected from 8 x 8 x 16 in. stretcher and end blocks. The tests were made in conformance with
Concrete Masonry Units as required by CSA Standard CAN3-S304-M78 (Ref. 20). The bearing surfaces of the units were capped with Hydrostone gypsum cement. The results are listed in Table 4.1.

According to CSA.CAN3-S304 the compressive strength shall be obtained by subtracting one and a half times the standard deviation from the average compressive strength. In our case:

on the net area: \[ \sigma_{\text{block}} = 3520 \text{ psi (24.3 MPa)} \]

and on the gross area: \[ \sigma_{\text{block}} = 1865 \text{ psi (12.9 MPa)} \]

4.1.2 Block Tensile Tests

One of the major difficulties faced by researchers in this subject is the development of test techniques that will determine properties adequate for the analysis of larger elements. Many different testing techniques have been proposed and used in various investigations, with a consequent lack of correlation between them.

Indirect tests are carried out by supporting the unit as a beam on two roller supports and loading it at mid-span in order to determine the modulus of rupture.

Direct tensile tests consisted of gluing plates to the unit ends or clamping the ends with special grip devices and then pulling apart the unit. Extreme care is required when performing direct tensile strength tests to minimize eccentricity of the loading.

4.1.2(a) Block Tensile Tests at the University of British Columbia

At the University of British Columbia tensile strength of concrete blocks was determined by direct tension tests. The experimental specimens were the teeth of knock-out blocks, clamped at their ends with a grip device and pulled in the direction of the long dimension.
47.

Test results are shown in Table 4.2.

4.2 Mortar

Mortar consists of a plastic, workable mixture of cement, sand, water and lime. Other admixtures might be added because of architectural or engineering requirements. Mortar for concrete masonry should be designed to (1) join the blocks into an integral structure, (2) seal irregularities of the masonry blocks, providing a weathertight wall and preventing penetration of wind and water into and through the wall, (3) bond with steel joint reinforcement, metal ties and anchor bolts, so that they become an integral part of the masonry assemblage, and (4) compensate for size variations in the units by providing a bed to accommodate tolerances of the blocks.

Good mortar is necessary for good workmanship and proper structural performance of masonry construction. The main properties of mortar are:

(a) **Workability.** The workability of the fresh mortar must be such that the mason can fill all the joints easily; it should ease placing of the unit without subsequent shifting due to its weight or the weight of successive courses.

(b) **Water retentivity.** This property is related to workability. Rapid loss of water might cause the mortar to stiffen too fast preventing the achievement of good bond and water-tight joints. The properties of the blocks, especially the suction, play an important role in water retentivity.

(c) **The Rate of Hardening.** The rate of hardening of mortar due to hydration, if too rapid, may reduce the workability and bond strength; very slow hardening may cause the mortar to flow.
(d) **Bond.** This property is influenced by: (1) extent of bond or degree of contact of the mortar with the masonry units, and (2) tensile bond strength, which is both a chemical and mechanical action. Bond is affected by: the mortar components and their proportions, characteristics of the masonry units, workmanship, and curing conditions. The bond strength of the mortar increases as the cement content increases and also as the water content increases (though mortar compressive strength decreases as the water cement ratio increases).

(e) **Compressive Strength.** The compressive strength of a masonry assemblage may be increased with a stronger mortar, but this increase is not proportional to the increase in the compressive strength of the mortar. It has been found experimentally that an increase of 130% in the mortar compressive strength results in only a 10% increase in the compressive strength of concrete masonry walls. Compressive strength of mortar increases with an increase in cement content and decreases with an increase in air, lime or water content. Compressive strength measurement involves casting, curing and testing 2-in. (50 mm) cubes in compression (CSA A179M, CSA A8, ASTM C270).

The current specifications for mortars (ASTM C270, CSA A179M) classify five types of mortars: M, S, N, O and K. Mortar types are identified by property or proportion specification, but not both. Mortar type classification under the property specifications is dependent solely on the compressive strength. The proportion specification identifies mortar type through various combinations of portland cement with masonry cement. When not otherwise specified the proportion specification governs.
In Canada the selection of mortars depends on whether or not the masonry is design using engineering analysis. When this approach is used, types M, S or N are required (CSA-CAN3-S304).

4.2.1 Mortar Tests at the University of British Columbia

The mortar used at the University of British Columbia was proportioned to meet the proportions by volume specification corresponding to portland cement-lime type S. A correction was made in the aggregate portion (sand) in order to compensate for bulking as required in clause 9.3 of the CSA Standard A179M (Ref. 33). The mortar contained by volume 1 part of type 10 portland cement, 1/2 part of hydrated lime, and between 4 1/2 and 5 parts of masonry sand (fine sand).

The mortar was mixed in a rotating drum concrete mixer. Retempering was permitted but no mortar was used that was more than 2 hours old.

4.2.1(a) Compression Tests

Mortar cubes 2 x 2 x 2 in. (50 x 50 x 50 mm) were cast and cured in accordance with CSA-Standard A179M (Ref. 33), and CSA-Standard A8 (Ref. 34). Later in our experimental work, the following changes were introduced in the cubes casting procedure as described in CSA-A8 in order to reproduce more accurately the conditions of the mortar in the joints:

(a) The sample mortar was taken from the batch and spread on a wooden board where it rested for a couple of minutes.

(b) It was then spread on the face of a concrete block, in an attempt to reproduce the water suction by the masonry units in the walls.

(c) After a minute it was then placed in the moulds.
The results are summarized on Table 4.III.

It can be seen that although the materials were proportioned in order to obtain a portland cement-lime mortar of type S the compression strength never reached the required minimum compressive strength of 1800 psi (12.4 MPa); in other words we obtained a strong type N mortar. Two factors that might have influenced this strength reduction are: (a) the increased proportion of masonry sand (after adjustment for bulking) although in one case the mortar was proportioned with 4 1/2 parts of sand (b) the water cement ratio. The range of strengths was wide, but there was consistency in the values obtained for mortar-cubes sampled on the same day. There was no data available to explain the variation in measured strengths, although the water cement ratio and the change in experimental procedure could be mentioned as two of the possible factors causing that variation.

4.2.1(b) Tensile-Bond Strength Test at the University of British Columbia

The bond strength is usually assessed on the basis of compressive strength values obtained from 2-inch cubes. At U.B.C. the tensile-bond strength was measured by a direct tension test, on samples made out of two teeth of knock-out blocks joined together by the mortar. The samples were cast at the time of the wall construction. They were lined up and leveled on a wooden board where they kept until they were tested. During that time, they were covered with a polyethylene plastic sheet in order to reproduce the moisture conditions of the mortar in the joints (mainly in the inner part of the joint). In order to test the specimens a special grip device that clamped the teeth at their ends was developed (and used also for the tension tests of concrete blocks). Special care was taken in order to minimize the bending of the specimens at the time of the tests.
Test results are shown on Table 4.IV.

It is obvious that there is a lot of scatter or variability in the results, some of which can be attributed to (a) the rate of loading (sometimes it was too fast, and it was all over before the speed could be adjusted) (b) eccentricity of the load resulting in a premature failure.

4.3 Grout

Grout is a high-slump concrete made with small aggregate. It must be fluid enough to fill all voids without segregation and completely encase the reinforcement. Its main function is to bond (a) the wythes together in a composite wall (b) the reinforcing steel to the masonry.

Grout for use in concrete masonry walls shall comply with requirements of CSA Standard A179M (Ref. 33) [Similar to ASTM C476 (Ref. 35)].

Admixtures are used in cases where it is necessary to reduce early water loss by absorption by the masonry blocks, to promote bonding of the grout to all interior surfaces of the units, and to produce a slight expansion sufficient to help ensure complete filling of the cavities. The excessive water in the fluid grout is absorbed by the masonry blocks, thus reducing the apparently high water-cement ratio.

The consistency of the grout is measured using a slump test (ASTM C143). Slump is not specified in most codes, however, when slump is measured using ASTM C143 Standard Method of Test for Slump of Portland Cement Concrete, the desired slump is 8 in. (20 mm) for units with low absorption and up to 10 in. (250 mm) for units with high absorption (e.g.: the Commentary-ACI-531R-79 recommends a minimum of 8 in.).

Some building codes (e.g. UBC code Ref. 23) require a minimum compressive strength of 2000 psi (1.4 MPa) for grout at 28 days, when
tested according to UBC-No. 24-23 (or CSA-A179M). The Commentary on ACI-531 (Ref. 21) requires that the grout compressive strength be at least equal to that of the required strength of the masonry to ensure a higher working relationship (and so implies clause 4.3.3.7 in CSA-S304).

4.3.1 Grout Compression Tests at the University of British Columbia

The compressive strength of the grout used in grouting the reinforced walls tested at U.B.C. was evaluated experimentally by testing specimens prepared in accordance with the general guidelines established by CSA Standard A179M, except that they were not cured with an impermeable sheet in order to retain their moisture content. A change was also introduced regarding the capping of the specimens: instead of being capped 48 hours after their casting, they were capped on the same day of the compression test, as follows:

a) all the specimens were air dried for 1 hour
b) they were then capped with sulfur
c) they were then returned to the moisture room and tested wet after 2 hours.

The capping was done according to ASTM-C617. The results are summarized on Table 4.V.

4.4 Determination of the compressive Strength of Masonry ($f'_m$).

The compressive strength of masonry is the most important parameter in the design of masonry structures. In order to establish the ultimate design strength of the masonry assemblage ($f'_m$), the UBC Code (Ref. 23) and CSA-S304 (Ref. 20) allow two methods: (1) an estimate based on the masonry unit strength and mortar type, which is purposely conservative,
(2) prism tests, which are relatively difficult to perform but will generally provide the designer with higher allowable stress.

4.4.1 Prism Tests

When $f'_m$ is to be established by tests, the specimens consist of prisms made from the wall materials. The moisture content, the consistency at the time of laying, the mortar joint thickness as well as the workmanship, should be the same as for the actual walls. The unit compressive stress for each specimen is obtained by dividing the ultimate load by the net area of the blocks and multiplying the result by a correction factor depending on the thickness to height ratio of the prism. (See Table 1 in CSA-S304, Ref. 20). The code specifications (Ref. 20, and 23), require at least five specimens to be tested and the compressive strength to be taken as the average failure stress less one and a half standard deviations.

4.4.1(a) Variables Influencing Prism Compressive Strength

The compressive strength of an assemblage usually lies between the compressive strength of the mortar and that of the masonry block.

The modulus of elasticity of the mortar is usually smaller than that of the masonry unit, resulting in larger lateral deformations in the mortar than in the blocks. If the Poisson's ratio of the mortar is greater than of the blocks, this will result in even bigger differences in the lateral strains. Because of friction and bond, the blocks restrain the lateral expansion of the mortar, producing tension in the blocks and sometimes resulting in tensile failure before compressive failure.
The influence of the mortar composition on the prism strength, has been found by some investigators to be of some significance. Ref. 36 notes a reduction in prism strength of more than half as one goes from type M to Type O mortar which is a reduction in mortar strength of a factor of 12. The strength of prisms built with high bond mortar was found to be about 37% greater than those built with conventional mortar. Drysdale and Hamid (Ref. 37) found that a decrease in mortar strength of about 70% resulted in a corresponding decrease in prism strength of less than 10%. Sahlin (Ref. 4) quotes an investigation by Nylander in which sand filled joints produced a masonry with a strength of about 60% of masonry with medium strength mortar.

The influence of the joint thickness relative to the height of the masonry unit has also been recognized as a significant parameter of the prism strength. Since the mortar is usually the weakest part of the assemblage, the highest strengths are obtained with thin bed joints. The results obtained in Ref. 37 showed that increasing the joint thickness from 3/8 to 3/4 in. (~10 mm to ~20 mm) resulted in the prism strength decreasing by 16% for ungrouted masonry (but only 3% for grouted masonry). Sahlin (Ref. 4) quotes some investigators pointing out that if the joints are thin, mortar strength has little influence on the strength of the masonry.

Masonry walls are usually laid with mortar along the face shell but not along the webs (face-shell bedding). Maurenbrecher (Ref. 38) pointed out the need for prisms to reflect this practice because prisms with face-shell bedding fail at an apparent lower stress than do those with full bedding. He recommended the use of the bedded area instead of the net
area of the blocks in order to obtain the correct stresses.

If a two block prism is tested in compression between plates or capping material of higher elastic modulus than the specimen, there is an end restraint and the tensile stresses induced in the blocks are reduced. The failure load is increased and failure usually takes place in a shear mode similar to that observed in the individual block test but not observed in walls or prisms composed of more courses. It can be expected that by increasing the number of courses, the blocks in the center of the prism would be free from the end restraint and be more representative of the conditions in an actual wall. However, experimental data does not seem to confirm this expectation. B.F. Boult (Ref. 39) determined that the reduction in strength with height appears to be insignificant for 3 to 12 course grouted prisms. Maurenbrecher (Ref. 38) found that the reduction in strength for prisms built with a height-to-thickness ratio higher than two (up to h/t = 6) was very small. He suggested that correction factors applied to hollow concrete block prisms with h/t > 2 should not be larger than one (the factor for h/t = 2).

Prisms are usually capped to ensure a more uniform load distribution. The masonry standards (eg. ASTM-C447) specify a sulfur or a dental plaster capping. Maurenbrecher (Ref. 38) compared the results for prisms tested with fibre board capping to those obtained for prisms tested with plaster capping. The mean strengths were similar, with fibre board capping giving slightly lower strengths. Similar results were obtained by Roberts (Ref. 31) when comparing fibre board and mortar caps.

Drysdale and Hamid (Ref. 37) investigated the influence of grouting the prisms. They determined that the grout, which occupied approximately
40% of the gross are, did not contribute proportionally to the prism strength. Large increases in the grout strength resulted in only relatively small increases in prism strength. The test results indicated that at failure the block itself was stressed to about 80% of its compressive strength for ungrouted prisms and to about 60% in grouted prisms. The authors suggested that matching the deformational characteristics of the grout and block may be more efficient than increasing the grout strength as proposed by some codes (e.g.: ACI-531).

Drysdale and Hamid (Ref. 37) also studied the influence of joint reinforcement in grouted and ungrouted specimens. Use of No. 9 gage wire joint reinforcement resulted in an increase of 2% and 5% for the ungrouted and grouted prisms respectively. On the other hand, Hatzinikolas et al. (Ref. 13 and 40) reported a 19% decrease in the compressive strength of two-block prisms with joint reinforcement.

Workmanship has a big influence on the compressive strength of masonry. The detrimental effect of poor workmanship is due to improper filling and tooing of the joints.

Other factors influencing the strength of the masonry are: coring (Refs. 4, 39, 37), age (Refs. 4, 38), loading rate (Ref. 38), and initial ratio of absorption (Ref. 4).

4.4.2 Unit Test Method

Rather than conduct expensive prism tests to determine the masonry design strength, a value of \( f' \) may be determined using the unit test method. In this case, \( f'_m \) is obtained from a table, as a function of the masonry unit strength and the type of mortar, and generally produces a
conservative estimate of $f'_m$. It assumes that mortar bed joints will be 3/8 in. (~10 mm) (± 1/8 in. [±3 mm]) in thickness. The masonry units and other materials must be tested in accordance with applicable CSA Standards [or ASTM when using other Codes: ACI (Ref. 21), UBC (Ref. 23)]. The $f'_m$ value obtained from those tables is not affected by whether the block is hollow or grouted, but the code requires for the last case that the grout be at least as strong as the blocks. (CSA-CAN3-S304, Cl. 4.3.3.7).

4.4.3 Prism Test at the University of British Columbia

Sixteen two-block prisms were built at U.B.C. for the first series of tests. One prism built with half block units was grouted, the others, built with stretcher or end block units, were all ungrouted. Some of the ungrouted prisms were fully bedded, the rest were constructed with face-shell mortar only. The units used for the construction of the prisms were selected randomly from the blocks used in the walls specimens, and the prisms were laid by the masons who constructed the walls. The prism joints were tooled identically to the wall joints. The prisms were levelled and plumbed. For ease of levelling each prism was built on a mortar bed, and placed inside a polyethylene bag to preserve the moisture and reproduce the curing conditions in the wall joints. The plastic bags were opened just before capping. The prisms were capped on or the day before the test day with Hydrostone Gypsum cement. Although care was taken during the handling of the prisms, the bond between the blocks and the mortar was broken in two of the earlier specimens.

Table 4.IV(a) shows the strengths obtained.

If the value of $f'_m$ was to be determined by the unit and mortar tests
method using Table 12 in CSA-CAN3-S304, we would have obtained:

\[ f' = 1235 \text{ psi} = 8.5 \text{ MPa} \quad \text{for mortar type N, and block compressive strength of 3520 psi} = 24.3 \text{ MPa}. \]

which is much smaller than the 2000 psi(13.8 MPa) obtained experimentally.

In an attempt to investigate the influence of alternate methods of capping on the prism test results, three exploratory tests were carried out using Dona Cona boards, a moderately soft fibre board. In two tests the capping consisted of 1/2" Dona Cona board covering the entire contact surface, and one test used 3/8" Dona Cona on the face shells only. In all three cases a tensile splitting failure mode with vertical cracks running through the block webs was observed. Test results shown in Table 4.VI(b) confirm the observation made by Maurenbrecher (Ref. 38) that full capping causes premature cracking as a result of bending in the web sections not bedded in mortar, although in our case the thinner capping board may have contributed to the higher strength.

4.5 Reinforcing Steel

4.5.1 Reinforcing Bars

Bars used to reinforce masonry are normally the same as used in reinforced concrete, and thus must comply with ASTM Standard A615 - Grade 40 or 60. As the reinforcement is placed mainly to resist tensile forces and provide ductility, its most important parameters are the yield stress, the ultimate strength and its elongation. Generally grade 40 is recommended for its greater ductility but in circumstances where there are very large loads, grade 60 might be used.
4.5.1(a) Tests at the University of British Columbia

The reinforcement bars used at U.B.C. consisted of #4, and #6 grade 60 bars. The test specimens consisted of segments of the #4 and #6 bars actually used in the walls. Table 4.VII presents the results.

4.5.2 Joint Reinforcement

The Canadian Code (CSA-CAN3-S304 in articles 4.6.8.1.2. and 4.6.8.2.4) specifies that wire reinforcement in the mortar joints may be considered as required horizontal steel. Joint reinforcement consists of longitudinal wires joined with intermittent wires in either a ladder or truss type arrangement.

4.5.2(a) Tests at the University of British Columbia

The joint reinforcement used at U.B.C was galvanized #9 gauge wire ladder type, with the wires bent into a slightly corrugated shape. Tensile tests were performed on a single wire, without any surrounding mortar, and so the corrugations would tend to straighten out more than in an actual wall. Test results are summarized in Table 4.VIII.
### Table 4.1

Compression Strength of Concrete Blocks

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Max. Load (lbs.)</th>
<th>Stress on Gross Area (119 in²)</th>
<th>Stress on Net Area (63 in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>272,000</td>
<td>2283</td>
<td>4317</td>
</tr>
<tr>
<td>2</td>
<td>257,000</td>
<td>2157</td>
<td>4079</td>
</tr>
<tr>
<td>3</td>
<td>220,000</td>
<td>1847</td>
<td>3492</td>
</tr>
<tr>
<td>4</td>
<td>276,500</td>
<td>2321</td>
<td>4381</td>
</tr>
<tr>
<td>5</td>
<td>231,000</td>
<td>2107</td>
<td>3984</td>
</tr>
</tbody>
</table>

**AVERAGE**

<table>
<thead>
<tr>
<th></th>
<th>Stress on Gross Area (119 in²)</th>
<th>Stress on Net Area (63 in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2143 psi (14.8 MPa)</td>
<td>4051 (27.9 MPa)</td>
</tr>
<tr>
<td><strong>STANDARD DEVIATION</strong></td>
<td>187 psi (1.3 MPa)</td>
<td>352 psi (2.4 MPa)</td>
</tr>
<tr>
<td><strong>COMPRESSIVE BLOCK STRENGTH</strong></td>
<td>1865 psi (12.9 MPa)</td>
<td>3520 psi (24.3 MPa)</td>
</tr>
</tbody>
</table>

### Table 4.2

Tension Test of Blocks

<table>
<thead>
<tr>
<th>Test.</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
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<tr>
<td>2</td>
<td>152</td>
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<tr>
<td>3</td>
<td>180</td>
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<tr>
<td>4</td>
<td>225</td>
</tr>
<tr>
<td>5</td>
<td>209</td>
</tr>
</tbody>
</table>

**Average**

<table>
<thead>
<tr>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>207 (1.4 MPa)</td>
</tr>
</tbody>
</table>
Table 4.III - Part I
Mortar Compressive Strength

<table>
<thead>
<tr>
<th>Test #</th>
<th>Age (Days)</th>
<th>Compressive Strength (psi)</th>
<th>Mortar Components cement:lime:sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7 days</td>
<td>1090</td>
<td>1 : 1/2 : 5</td>
</tr>
<tr>
<td>2</td>
<td>&quot; &quot;</td>
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<td>&quot; &quot;</td>
</tr>
<tr>
<td>3</td>
<td>28 days</td>
<td>1665</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>4</td>
<td>&quot; &quot;</td>
<td>1660</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>5</td>
<td>&quot; &quot;</td>
<td>1430</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>6</td>
<td>&quot; &quot;</td>
<td>1530</td>
<td>1 : 1/2 : 5</td>
</tr>
<tr>
<td>7</td>
<td>&quot; &quot;</td>
<td>1550</td>
<td>1 : 1/2 : 4 1/2</td>
</tr>
<tr>
<td>8</td>
<td>&quot; &quot;</td>
<td>1515</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>9</td>
<td>&quot; &quot;</td>
<td>1415</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>10</td>
<td>&quot; &quot;</td>
<td>1020</td>
<td>&quot; &quot;</td>
</tr>
<tr>
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<td>&quot; &quot;</td>
<td>995</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>12</td>
<td>&quot; &quot;</td>
<td>1000</td>
<td>1 : 1/2 : 4 1/2</td>
</tr>
<tr>
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<td>&quot; &quot;</td>
<td>635</td>
<td>1 : 1/2 : 5 1/2</td>
</tr>
<tr>
<td>14</td>
<td>&quot; &quot;</td>
<td>650</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>15</td>
<td>&quot; &quot;</td>
<td>850</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>16</td>
<td>&quot; &quot;</td>
<td>850</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>17</td>
<td>&quot; &quot;</td>
<td>890</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>18</td>
<td>&quot; &quot;</td>
<td>895</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>19</td>
<td>&quot; &quot;</td>
<td>900</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>20</td>
<td>&quot; &quot;</td>
<td>885</td>
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</tr>
<tr>
<td>22</td>
<td>&quot; &quot;</td>
<td>855</td>
<td>&quot; &quot;</td>
</tr>
<tr>
<td>23</td>
<td>&quot; &quot;</td>
<td>910</td>
<td>1 : 1/2 : 5 1/2</td>
</tr>
</tbody>
</table>

Average of 21 tests at 28 days: 1095 psi (7.6 MPa)

Note: Changes in the sampling procedure were carried out from sample 7 onwards.
Table 4.III - Part II
Mortar Compressive Strength (Samples Corresponding to Wall Series Not Tested in This Program)

<table>
<thead>
<tr>
<th>Test #</th>
<th>Age (Days)</th>
<th>Compressive Strength (psi)</th>
<th>Mortar Components cement:lime:sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>37</td>
<td>1470</td>
<td>1 : 1/2 : 5</td>
</tr>
<tr>
<td>25</td>
<td>&quot;</td>
<td>1740</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>26</td>
<td>&quot;</td>
<td>1395</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>27</td>
<td>&quot;</td>
<td>1580</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>28</td>
<td>&quot;</td>
<td>1410</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>29</td>
<td>&quot;</td>
<td>1655</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>30</td>
<td>36</td>
<td>1085</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>31</td>
<td>&quot;</td>
<td>1045</td>
<td>&quot; &quot; &quot;</td>
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</tr>
<tr>
<td>33</td>
<td>&quot;</td>
<td>1175</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>34</td>
<td>&quot;</td>
<td>1205</td>
<td>&quot; &quot; &quot;</td>
</tr>
<tr>
<td>35</td>
<td>&quot;</td>
<td>1220</td>
<td>1 : 1/2 : 5</td>
</tr>
</tbody>
</table>

Average of last 12 tests: 1335 psi (9.2 MPa)
### Table 4.IV - Part I

Mortar Tensile-Bond Strength

<table>
<thead>
<tr>
<th>Test #</th>
<th>Age (Days)</th>
<th>Tensile Strength (psi)</th>
<th>Mortar Composition Cement:Lime:Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45</td>
<td>26</td>
<td>1 : 1/2 : 4 1/2</td>
</tr>
<tr>
<td>2</td>
<td>&quot;</td>
<td>31</td>
<td>&quot;</td>
</tr>
<tr>
<td>3</td>
<td>&quot;</td>
<td>20</td>
<td>&quot;</td>
</tr>
<tr>
<td>4</td>
<td>&quot;</td>
<td>42</td>
<td>1 : 1/2 : 4 1/2</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>61</td>
<td>1 : 1/2 : 5 1/2</td>
</tr>
<tr>
<td>6</td>
<td>&quot;</td>
<td>48</td>
<td>&quot;</td>
</tr>
<tr>
<td>7</td>
<td>&quot;</td>
<td>73</td>
<td>&quot;</td>
</tr>
<tr>
<td>8</td>
<td>&quot;</td>
<td>80</td>
<td>&quot;</td>
</tr>
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<td>9</td>
<td>&quot;</td>
<td>96</td>
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</tr>
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<td>10</td>
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<td>112</td>
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</tr>
<tr>
<td>11</td>
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<td>82</td>
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</tr>
<tr>
<td>12</td>
<td>&quot;</td>
<td>71</td>
<td>&quot;</td>
</tr>
<tr>
<td>13</td>
<td>&quot;</td>
<td>63</td>
<td>&quot;</td>
</tr>
<tr>
<td>14</td>
<td>&quot;</td>
<td>48</td>
<td>&quot;</td>
</tr>
<tr>
<td>15</td>
<td>&quot;</td>
<td>60</td>
<td>1 : 1/2 : 5 1/2</td>
</tr>
</tbody>
</table>

Average of 15 tests: 61 psi (0.4 MPa)

### Table 4.IV - Part II

Mortar Tensile-Bond Strength (Samples Corresponding to Wall Series Not Tested in This Program)

<table>
<thead>
<tr>
<th>Test #</th>
<th>Age (Days)</th>
<th>Tensile Strength (psi)</th>
<th>Mortar Composition Cement:Lime:Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>37</td>
<td>28</td>
<td>1 : 1/2 : 5</td>
</tr>
<tr>
<td>2</td>
<td>&quot;</td>
<td>63</td>
<td>&quot;</td>
</tr>
<tr>
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</tr>
<tr>
<td>5</td>
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<tr>
<td>6</td>
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<td>65</td>
<td>&quot;</td>
</tr>
<tr>
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<td>&quot;</td>
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<td>8</td>
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<td>9</td>
<td>&quot;</td>
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<td>54</td>
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<tr>
<td>14</td>
<td>&quot;</td>
<td>46</td>
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Average of 14 tests: 60 psi (0.4 MPa).
Table 4.V – Part I
Grout Compressive Strength

<table>
<thead>
<tr>
<th>Test</th>
<th>Age (Days)</th>
<th>Compressive Strength (psi)</th>
<th>Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>2722</td>
<td>cement + 19 lbs</td>
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<tr>
<td>2</td>
<td>&quot;</td>
<td>2736</td>
<td>coarse sand + 88 lbs</td>
</tr>
<tr>
<td>3</td>
<td>&quot;</td>
<td>2815</td>
<td>3/8&quot; stone + 35 lbs</td>
</tr>
<tr>
<td>4</td>
<td>31</td>
<td>3006</td>
<td>Pozz + 20 ml</td>
</tr>
<tr>
<td>5</td>
<td>&quot;</td>
<td>3072</td>
<td></td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td>14</td>
<td>&quot;</td>
<td>3893</td>
<td></td>
</tr>
</tbody>
</table>

Average for 14 tests = 3214 psi = 22.2 MPa.
Slump for samples 1 to 3 = 7.5" = 190 mm.
" " " 4 to 8 + 6 " = 150 mm.

Note: the proportion of sand was originally 83 lbs, but was increased to 88 lbs in order to correct for bulking.

Table 4.V – Part II Grout Compressive Strength
(Sample Corresponding to a Walls Series Not Tested Yet in the Program)

<table>
<thead>
<tr>
<th>Test</th>
<th>Age</th>
<th>Compressive Strength (psi)</th>
<th>Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>37</td>
<td>3560</td>
<td>cement + 19 lbs</td>
</tr>
<tr>
<td>16</td>
<td>37</td>
<td>3635</td>
<td>Coarse Sand + 83 lbs</td>
</tr>
<tr>
<td>17</td>
<td>36</td>
<td>2730</td>
<td>3/8&quot; + 35 lbs</td>
</tr>
<tr>
<td>18</td>
<td>&quot;</td>
<td>2575</td>
<td>Pozz + 20 ml</td>
</tr>
<tr>
<td>19</td>
<td>&quot;</td>
<td>3940</td>
<td></td>
</tr>
</tbody>
</table>

Average Compressive Strength for Samples 15 to 19 = 3288 psi = 22.7 MPa.
Slump for Samples 15 to 19 = 7 1/4" = 185 mm.
### Table 4.VI(a)

**Prism Tests**

<table>
<thead>
<tr>
<th>Test</th>
<th>Age (Days)</th>
<th>Prism Strength (psi)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36</td>
<td>2302</td>
<td>No bond between mortar and blocks</td>
</tr>
<tr>
<td>2</td>
<td>35</td>
<td>2381</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>35</td>
<td>2683</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>35</td>
<td>2349</td>
<td>No Bond between mortar and blocks</td>
</tr>
<tr>
<td>5</td>
<td>35</td>
<td>2167</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>&quot;</td>
<td>2294</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>&quot;</td>
<td>2516</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>&quot;</td>
<td>2079</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>&quot;</td>
<td>2603</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>47</td>
<td>2214</td>
<td>face-shell bedded *</td>
</tr>
<tr>
<td>11</td>
<td>&quot;</td>
<td>1913</td>
<td></td>
</tr>
</tbody>
</table>
| 12   | 46         | 2008                 |                              *
| 13   | "          | 2476                 |                              |
| 14   | "          | 2365                 |                              |
| 15   | "          | 2429                 |                              |
| 16   | 132        | 3,776 psi            | (was kept in moisture room) half block, grouted\(f'_m\) based on grouted area\(=\) 2 1/2 sand, 1 1/2 pea gravel 1 cement |

Average of 15 ungrouted tests: 2319 psi = 16 MPa. This would produce \(f'_m=2000\) psi = 13.8 MPa. The results are based on a net area of 63 in\(^2\).

*If based on the net bedded area (as recommended in Ref. 38) of 48 in\(^2\) we would obtain 2906 psi instead of 2214 psi, 2511 instead of 1913 psi and 2636 instead of 2008 psi.

### Table 4.VI(b)

**Prism Tests Capped with Dona Cona Boards**

<table>
<thead>
<tr>
<th>Test</th>
<th>Capping board/thickness (in.)</th>
<th>age (days)</th>
<th>Compressive strength (psi) (based on block net area 63 in(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1/2</td>
<td>92</td>
<td>626</td>
</tr>
<tr>
<td>2</td>
<td>1/2</td>
<td>&quot;</td>
<td>745</td>
</tr>
<tr>
<td>3</td>
<td>3/8</td>
<td>&quot;</td>
<td>1505*</td>
</tr>
</tbody>
</table>

Note: *When only face shell is taken into account: strength = 2230 psi
Table 4.VII
Tension Tests of Reinforcing Bars

| Test | Tensile Stress (psi) | | Tensile Stress (psi) |
|------|----------------------|----------------------|
|      | #4                  | #6                  |
|      | Yield Max. | Yield Max. |
| 1    | 66500     | 106750 |
| 2    | 66500     | 107200 |
| 3    | 67500     | 105800 |
| 4    |           | 70909   | 112045 |
| 5    |           | 70114   | 110000 |
| 6    |           | 71364   | 110227 |
| 7    | 68750     | 107000  |
| 8    | 69000     | 107500  |
| 9    |           | 64773   | 106477 |
| 10   |           | 66591   | 108409 |
| 11   |           | 65000   | 106364 |

The average yield stress of the eleven specimens was: 67900 psi (470 MPa)
The average ultimate strength of the eleven specimens was: 108000 psi (745 MPa)

Note: On average the #6 bars were slightly stronger than the #4 bars.

Table 4.VIII
Tension Tests of Wire Reinforcement

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Stress $\sigma_y$ (psi)</th>
<th>Ultimate Stress $\sigma_{ult}$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>58,365</td>
<td>60,699</td>
</tr>
<tr>
<td>2</td>
<td>--</td>
<td>78,500</td>
</tr>
<tr>
<td>3</td>
<td>49,610</td>
<td>64,200</td>
</tr>
<tr>
<td>4</td>
<td>55,450</td>
<td>67,120</td>
</tr>
<tr>
<td>5</td>
<td>74,120</td>
<td>75,290</td>
</tr>
<tr>
<td>Average</td>
<td>59,400</td>
<td>69,200</td>
</tr>
</tbody>
</table>
CHAPTER 5

Test Series at the University of British Columbia

At the University of British Columbia attention is being directed initially to the out of plane forces acting on non-loadbearing walls. The test series was designed to determine the parameters governing the spacing of principal reinforcement and the design of distribution steel. The main aims of this study are:

(a) To establish the maximum spacing of main steel without any transverse distribution steel for walls spanning either horizontally or vertically.

(b) To determine the effects of distribution steel on the spacing of the main steel.

(c) To determine the efficiency of joint reinforcement as horizontal distribution steel or as main steel for horizontal spanning walls.

(d) To determine a method of predicting the ability of the masonry to span between the main reinforcement or lateral supports.

The first objective was to determine an upper limit to the spacing without distribution steel. Thus, specimens having reinforcement steel in one direction only were tested, starting with walls spanning vertically with increasing spacing between reinforcement. Various arrangements of distribution steel in the form of bond beams or joint reinforcement were then introduced in order to study their influence in the behaviour of the walls. Tests were also performed on walls spanning horizontally with different arrangements of main reinforcement, including joint reinforcement, but with no distribution steel.
5.1 Test Set-Up

Test specimens consisted of 14 walls, 12 of which were 8 x 8 ft (2.44 x 2.44 m), one 8 inches (0.20 m) higher and one 16 inches (0.40 m) shorter. All were built of 8 x 8 x 16 in. (200 x 200 x 400 mm) ungrouted hollow concrete blocks in running bond construction. Reinforcement steel was grade 60 (60,000 psi or 415 MPa nominal yield stress) bars in grouted cores or bond beams and #9 gauge ladder-type joint reinforcement. The joint reinforcement was galvanized deformed wire bent in a slightly corrugated pattern.

The wall specimens were constructed and air cured in a laboratory environment. All walls were constructed by experienced masons using techniques typical of good workmanship and supervision. The mortar joints on both faces were tooled.

The walls were tested in the vertical position and loaded by means of an air bag placed between the panel and a reaction wall. In order that they could be moved around the laboratory they were all built on a wooden base beam. In an effort to prevent a lateral reaction from developing at the base of horizontally spanning walls they were, with two exceptions, supported vertically on sliding teflon base pads placed between the wall and the wooden beam. Hooked flat steel reaction bars placed at 8 inches (200 mm) on centres, and bearing on the unloaded face of the panel, provided a simple support reaction condition, except that in the case of the walls spanning vertically their self-weight gave some rotational resistance at the bottom.

Lateral displacements were measured at 9 points on a 3 x 3 grid using taut wires driving rotary potentiometers. The lateral pressure was
measured with a pressure transducer; and in most cases two strain gauges were placed on every main reinforcing bar. The recording of the data was done by a multi-channel Vidar "5200 Series D-DAS" recorder driving a paper tape. Figure 5.1 shows the test arrangement.

5.2 Test Series

To date the tests have involved monotonic quasi-static loads except for two walls subjected to one load reversal after yielding in the first direction. Eight walls spanned vertically and six horizontally. There were 15 tests as wall #1 was tested again after shearing off the upper course in the first test.

Table 5.1 shows the wall dimensions, reinforcing and boundary conditions along with the failure pressure, and a short description of the mode of failure. In some cases a load deflection plot is also shown. The steel ratio, was defined as $\rho = \frac{A_s}{tL}$, where $A_s$ is the total steel area in the length $L$ and $t$ is the wall thickness. All the walls with vertical main steel have 2#6 bars (19 mm diameter) for a steel ratio of 0.0011. The walls with horizontal main steel included #4 (13 mm diameter) and #6 (19 mm diameter) bars in bond beams, or joint reinforcement. The horizontal steel ratios varied from 0.00025 to 0.0014. All the reinforcing bars were placed in the middle of the walls.

5.3 Test Results

5.3.1 Vertical Spans

According to the Canadian Code, for non-loadbearing walls (CSA-CAN3-S304 - Clause 4.6.8.2.1.) in seismic zone 3, a minimum steel area of 0.001
A (or a steel ratio of 0.001) is required at a minimum spacing of 16 inches (400 mm) if there is reinforcement in one direction only. As a starting point this minimum steel area was provided but the spacing was increased to 4 feet (1.22 m), corresponding to the maximum spacing specified for walls having steel placed in two directions. No transverse distribution steel was included, as it was desired to find the maximum spacing of the bars that will bring about a failure of the plain masonry between the bars, thus establishing the maximum spacing of the main vertical steel without any transverse steel.

The recorded pressures were not particularly accurate for the first 3 tests which were exploratory in nature, but the results from test [1] and [2] with main vertical steel at 48 in. (1.22 m) showed that failure did not occur in the transverse direction in the blocks between the vertical bars.

Wall [1] failed in shear and bond at the top course at a maximum pressure of 310 psf (14.8 kPa). After removing the upper course, wall [1] was tested for a second time. The wall, which was already cracked after the first test, failed again along the top course at a pressure of 365 psf (17.5 kPa).

Wall [2] with the same arrangement as wall [1], failed at a pressure of 250 psf (12.0 kPa); it was a bending failure of one cantilever portion about the vertical reinforcement.

The spacing between vertical bars, was then increased in stages to a maximum of 72 inches (1.83 m). Wall [4] showed that the masonry was able to span at least 56 inches (1.42 m) between the main steel at the high load of 285 psf (13.6 kPa). The failure mode was similar to that of wall [2], a bending failure of one side about the vertical reinforcement.
Wall [5] with a steel spacing of 72 inches (1.83 m) and small edge cantilever finally failed by a bending mechanism in the masonry between the main reinforcing bars, although at a high load of 193 psf (9.2 kPa).

In an exploratory test to study the effects of repeated loading, an attempt was made to simulate the action of a cracked wall by reducing or destroying the bond between blocks and mortar through the application of a bond breaker. To this end wall [6] was built with the mortar faces of the blocks dipped in Sternson Bond Release, a compound commonly used to prevent bond between lift slabs poured on top of one another. The main steel consisted on 2#6 bars arranged similarly to wall [5]. A transverse #4 bar was placed in the top course to provide some containment capacity. It was hoped that the result of this test would give a lower bound on the behaviour under cyclic loading. However, the failure mechanism and load were essentially the same as for wall [5] (210 lb/ft$^2$ = 10.0 kPa). It is not known whether the bond breaker failed to perform or whether cracks have little effect on the load resistance.

Walls [5] and [6] showed that the masonry would resist a fairly high load, about 200 psf (9.6 kPa) over a span of 72 inches (1.83 m) between lines of main reinforcement. To investigate how much the span could be increased, and to learn how horizontal distribution reinforcement would affect the failure mode, several walls were built with increase distance between the vertical main steel plus horizontal distribution steel. The amount of distribution steel was designed so that the predicted failure mode would be a mechanism in the blocks or yielding of the distribution steel, but not a failure of the main steel. Distribution steel in the form of joint reinforcement or bond beams was provided with main steel at 72
inches (1.83 m) in walls [7] and [8] and 88 inches (2.24 m) in wall [14].

In wall [7], the joint reinforcement was placed in every course in order to obtain the same reinforcement ratio as #4 bars at 48 inches (1.22 m), which was the horizontal reinforcement for wall [8]. A #4 bar was placed at the top to prevent a premature shear or bond failure in the top course.

Wall [7] failed by a bending mechanism in the blocks between main bars. Wall [8] failed by bond on the horizontal #4 bar at mid-height. In both cases the primary bending capacity was also essentially fully developed. Wall [7] with the joint reinforcement performed better in the sense that there was less sign of distress before failure. After this good performance by wall [7], it was decided to build wall [14] with the main vertical steel at 88 inches (2.24 m) spacing and distribution steel consisting of joint reinforcement every second course. This wall failed in transverse bending between the main bars.

The failure load for walls [7] and [8] was more or less the same and about 1.75 times that of wall [5] which did not have the distribution steel. Wall [14] had about the same failure load as wall [5]. Figure 5.2 shows the deflection of the centre point with respect to the four corners for wall [14].

5.3.2 Horizontal Spans

The horizontally spanning walls included main steel spacing of 48 inches (1.22 m) (walls [3] and [9]) and 72 inches (1.83 m) (wall [10]), and walls with only joint reinforcement as main steel (walls [11], [12], [13]). No vertical distribution steel was used in these walls. In walls [12] and [13] joint reinforcement was placed in every course (ie. at 8 inches = 200
mm), whereas in wall [11] it was placed in every second course (at 16 inches). In order to see whether the teflon pads changed the mode of failure or failure load from that of a wall resting on the wooden beam (or the floor), they were omitted at the base of wall [13]. In all these cases (walls [9], [10], [11], [12], [13]) the failure load was very nearly the same: 125 to 130 psf (6.2 kPa). Wall [3] failed at a slightly higher load (160 psf = 7.7 kPa). In all cases, there was no cracking and very little deformations up to the failure load, whereafter the load capacity dropped off with increasing deflections. All the walls showed considerable ductility. Figures 5.3 to 5.7 show a series of plots of the load against the deflection of the centre point of the walls relative to their four corners.

Wall [3] suffered a bond failure of the top bar allowing a cantilever type of failure of the top portion. Wall [9] failed in shear along the bottom of the upper course and then failed in a bending mechanism in the blocks. Wall [10] failed in a bending mechanism of the blocks. The deflections at the centre point with respect to both a vertical line and a horizontal line showed close agreement which seems to confirm a two-way action (see Figure 5.8).

Walls [11], [12] and [13] appeared to fail in primary bending. The deflection of walls [11] and [12] confirmed the observation of one way bending (e.g.: see Fig. 5.9). Wall [13] exhibited more two way bending than wall [12] in that there was less damage to the lower part of the wall. However this did not appear to affect the load as the load deflection plots for the two walls were nearly identical out to about 0.4 inches (10 mm) deflection where wall [12] was reversed.
Walls [11] and [12] after being tested to failure in one direction were reversed and loaded from the other side. Figure 5.10 shows that post-cracking strength was only slightly reduced in the reverse cycle. Similar results were obtained by Scrivener (Ref. 8).

5.4 Analysis of Results

5.4.1 Bending

5.4.1(a) First Cracking

Of the walls spanning horizontally, specimens [11], [12] and [13] containing only joint reinforcement, reached a load of 120 - 130 psf (~ 6.2 kPa), when cracks formed in the vertical and connecting horizontal mortar joints. In each case the crack was confined to the mortar joints and did not pass through any blocks. After first cracking the load either remained nearly uniform or dropped off indicating an under-reinforced condition.

Walls [9] and [10] with bond beams, cracked at approximately the same load as walls [11], [12], [13]. In wall [10] first cracking was immediately followed by the formation of a mechanism between reinforcing bars, and loss of load capacity. Wall [9] had more closely spaced steel, but first cracking was followed by a shear failure along the top course followed by a bending failure in the top half of the panel. In all these walls, then, first cracking coincided with maximum load, and the cracking moment about a
vertical axis was about 1,000 lb/ft (4,450 Nm/m).*

In the walls spanning vertically, it was only possible to determine the cracking load for wall [14]. It was 55 psf (2.6 kPa), for a cracking moment of 440 lb/ft (~2,000 Nm/m). This moment corresponds to a tensile stress of about 63 psi (0.4 MPa) between block and mortar, which compares closely with the measured value of 61 psi (see Chapter 4). In the case of horizontal spans the cracking corresponds to a tensile stress of 140 to 150 psi (1.05 MPa) which is greater than twice the bond tensile strength of the mortar-block assemblage. This difference is attributed to the running bond construction which, if the crack is not a straight vertical crack through the mortar and blocks, forces the crack through the mortar in a combination of tension and shear failure. Other factors such as shear friction between blocks, self-weight of the walls, strength of the blocks, and restraint at the wall base all contribute to the moment capacity of the wall.

5.4.1(b) Capacity of Principal Reinforcement

The measured average strains in the bars were compared to the strains calculated on the basis of a linear stress-strain distribution of a cracked section. Figures 5.11 to 5.15 show the plots of strains vs. load obtained for walls with main vertical steel. In all cases the predicted theoretical strain values were very close to the experimental values. Based on the measured strength of the reinforcing steel the vertically spanning walls

*In the following discussion, moments will be defined in the vector sense. Thus, for example, a moment about a vertical axis, when the direction of the spanning is horizontal, will be referred as to vertical moment.
could resist a load of 285 psf (13.6 kPa) at yield \( f_y = 68 \text{ ksi} \) and 440 psf (21.1 kPa) at ultimate \( f_u = 108 \text{ ksi} \).

Figures 5.11 and 5.12 show the comparison plots for walls [4] and [6] respectively. In both cases the maximum load did not reach the theoretical yield value and this is reflected in the plots. It can be seen that the theoretical strain values are generally higher than the measured strain values.

Walls [7] and [8] (Figure 5.13 and 5.14) exceeded the theoretical yield load. In both cases the predicted yield load agreed closely with the load at which the measured average strains reached the yield strain. After yield, the bars showed the typical plastic deformation of steel. The unloading branch of the bar in wall [8] has not been plotted because the recorded strains showed an erratic behaviour, probably due to damage caused to one of its strain gages after failure in the bond beam.

Figure 5.15 shows the plot for wall [14]. In this case, up to a load of about 140 psf (6.7 kPa), the measured strains are smaller than the predicted values. For higher loads the opposite holds in a few cases but then the measured values remain within 3% of the theoretical values. Although this bar did not reach the yield stress some residual strains are observed in the unloading branch. They are probably due to a relative movement of the blocks not allowing a full recovery of the bar.

For walls [7], [8] and [14] only the average strain of one bar was plotted because the values obtained for the second bar were considered defective.

In general it can be observed that for the walls tested with vertical
main steel, the measured strain values follow the same trend as the theoretical values. The latter are bigger than the measured ones but the gap closes when approaching the yield load, which was very well predicted for walls [7] and [8]. Using ultimate strength theory, Scrivener (Ref. 8) obtained similar results for tests on reinforced brick walls. He assumed the stress-strain curve for the brick to be the same as for concrete and used the Whitney equation for reinforced concrete. That there is not much difference between the results obtained using either approach is due to the light reinforcement which causes a small compression zone in the masonry.

In an attempt to analyze the behaviour of the horizontally spanning walls a similar comparison plot was drawn for walls [9] and [10] (Figures 5.16 and 5.17 respectively). The general behaviour of the reinforcing bars corresponds to the observed behaviour in the walls exhibiting small displacement until the cracking load was reached; thereafter, large deformations. In these cases the experimental points have little correspondence to the theoretical values obtained assuming a constant curvature across any vertical section. It is apparent that the distribution of moments across the height of the section is not uniform. For example, at maximum load the midspan moment of wall [9] (Figure 5.16), if proportioned on the basis of steel areas, would require a moment of 53,650 lb.in (6,060 Nm) to be resisted by the #6 bar and 26,825 lb.in (3,030 Nm) by each of the #4 bars. If we change our assumptions and assume that the wall behaves as a continuous slab in the vertical direction with the bond beams as supports, the moments in each bond beam would be distributed in the same proportion as the reactions on the supports. Thus, the #6 bar would resist a moment of 67,100 lb.in (7,580 Nm) and each one of the #4 bars a moment of 20,120
1 lb.in (2,275 Nm). According to the measured strains the moment distribution is as follows: 13,680 lb.in (1,545 Nm) by the upper #4 bar, 28,950 lb.in (3,270 Nm) by the lower bar, and 48,760 lb in (5,510 Nm) by the #6 bar. This moment distribution: (a) does not add up to the total applied moment and (b) contradicts the distribution based on the above two assumptions and the more likely moment distribution in which, the upper #4 bar would have higher strains than the lower bars because of increasing moment resistance to horizontal bending in the masonry from top to bottom due to the self-weight of the wall, and base resistance.

Inspection of the wall displacements and bar strains before failure, and the mechanism of failure, helps to explain the discrepancy. Before failure, the displacements and strains measured were very small with higher values being registered at the top. At failure (see Table 5.1) the upper course (i.e. the upper bond beam) separated from the rest of the wall by a horizontal crack running along the entire wall length. This could explain the low strain registered in the upper #4 bar. In the top part of the wall between the bond beams, a yield line type of mechanism developed which could have helped to reduce the part of the moment resisted by the middle bond beam. The one way bending mechanism observed in the lower part of the wall is perhaps the cause of the high strains registered in the bottom #4 bar, because in this type of mechanism the resistance will be concentrated at the bond beams. In trying to explain why such a high strain was measured in the bottom bar, we cannot discount the possibility that the bar was not centered. We can see how the rather complicated mechanism makes it exceedingly difficult to estimate the moment distribution along a vertical section.
Based on the measured strength of the wire, the joint reinforcement should sustain a yield load of 41 psf (~2.0 kPa) and an ultimate load of about 48 psf (2.3 kPa) for $\rho = 0.00025$ as in wall [11], and double this in walls [12] and [13] with $\rho = 0.0005$. The load carried by walls [11], [12] and [13] after cracking considerably exceeded the capacity of the joint reinforcement and points out the importance of masonry mechanisms in resisting the post cracking loads.

5.4.1(c) Bending Resistance of Blocks Between Reinforcement

In order to set limits on the spacing of the principal reinforcement, and to be able to design the distribution steel, one must know the behaviour of the masonry between the main reinforcing bars. Considerable efforts have been made to find a theory suitable for this purpose. Where failure did occur in this mode, a yield line pattern was observed to form in the panels inside a perimeter defined by the main steel and the supports (see Table 5.I). Yield line analyses were carried out on those walls that showed a well defined bending mechanism. Yield line theory assumes that the bending moment at a point along a line reaches a yield value and remains constant while other parts of the line reach that value; thus, a yield line pattern develops with constant moments along each line. This type of analysis implies relatively large deflections and ductile behaviour, which was observed in our walls.

Assuming that the yield lines follow the major crack patterns as dimensioned in Table 5.II, the relation between moment capacities and lateral load is given in column 5 of Table 5.II.

Let $M_{vb} = \text{vertical moment capacity of the blocks}$
\( M_H \) = horizontal moment capacity of the blocks

\( M_{vr} \) = vertical moment capacity of joint reinforcement every 2nd course.

\( M_V = M_{vr} + M_{vb} \) = total vertical moment capacity.

In extensively cracked walls the masonry can have little resistance to horizontal moments, only the dead load of the wall or the in-plane forces imposed by the vertical reinforcement can provide some resistance.

With respect to the vertical moment capacity in walls with running bond construction, where the cracks follow the mortar lines, curvature must cause relative rotation between the interlocking blocks. The consequent shear friction could be expected to provide resistance to vertical moments greater than the horizontal moments. The magnitude would be a function of the vertical forces arising from the dead weight or the clamping force caused by the tension in the vertical steel.

Previous works (Refs. 2, 3, 4 and 7) reported a modulus of rupture in horizontal bending (i.e. vertical moments) that was 3 to 6 times higher than the modulus of rupture in vertical bending (i.e. horizontal moments). The observations made in our series of test tended to confirm the above; the cracking load for wall \([14]\) was 55 psf (2.6 kPa) whereas wall \([9]\) to \([13]\) spanning horizontally had a cracking load of 120 to 130 psf (\(\sim 6.2\) kPa). It is important to note that whereas all the above mentioned references, and our own results, deal with cracking moments (or modulus of rupture), the yield line analysis deals with the ultimate moment capacity. We will assume that the same relationship applies for the ultimate moment capacity ratio, i.e.: \( M_{vb}/M_H = 3 \) to 6.
The data presented in Table 5.II suggest that $M_{vb}$ is considerably greater in walls with vertical main steel than it is in those with horizontal reinforcement only. This is consistent with the fact that there is an extra vertical compressive force on the blocks arising from the tension of the reinforcing bars.

If it is taken that

$$M_{vb} = 460 \text{ lbs.ft/ft (2.07 kNm/m)} \text{ in walls with vertical reinforcement}$$

$$M_{vb} = 190 \text{ lbs.ft/ft (0.86 kNm/m)} \text{ in walls without vertical reinforcement}$$

$$M_{vb} = 3 M_H$$

$$M_{vr} = 333 \text{ lbs.ft/ft} \text{ for } \rho = 0.00025$$

then the lateral loads that would be sustained by the assumed mechanisms are listed in column 7 of Table 5.II, and the discrepancy between these values and the observed loads are given in column 8. Except for walls [7] and [14] the agreement is good. Wall [7] may not have been a true test of the yield line theory as the failure load also exceeded the yield capacity of the main reinforcement. In wall [14] there was a shear failure below the top course; if the estimated force in this joint is included in the analysis, by assuming that at least 2/3 of the weight of the upper course bears on the wall with a friction coefficient of 1.0, the error is reduced to 13%.

If we change our initial assumption and assume $M_{vb} = 6 M_H$ the moment capacities that give reasonable agreement with the experimental results are:
M\textsubscript{vb} = 510 \text{ lb.ft/ft}(2.29 \text{ Nm/m}) in walls with vertical reinforcement

M\textsubscript{vb} = 235 \text{ lb.ft/ft}(1.06 \text{ Nm/m}) in walls without vertical reinforcement

M\textsubscript{vr} = 333 \text{ lb.ft/ft}(1.50 \text{ Nm/m}).

It is interesting to note that the estimated moment capacities do not differ much with a change in the \(\frac{M\textsubscript{vb}}{M\textsubscript{H}}\) ratio. (See Table 5.II column 9 and 10).

The question arises of what friction coefficient (\(\mu\)) would be required to achieve such moment ratios. If we assume that because of simultaneous horizontal bending the contact area between interlocking blocks will be reduced to the face shell, then the shear area will be as shown in Figure 5.18.

Assuming a plastic yield shear stress distribution (Ref. 4) as shown in Figure 5.19 the maximum moment carried on such an area is:

\[
M = \tau \frac{t_1^2}{2} (\ell - \frac{t_1}{3}) = \mu \sigma \frac{t_1^2}{2} (\ell - \frac{t_1}{3})
\]

(1)

where \(\tau\) is the shear stress, \(\sigma\) is the vertical compressive stress, and \(t_1\) and \(\ell\) are defined in Figure 5.19.

The moment capacity per unit height would be given by:

\[
M\textsubscript{vb} = \frac{M}{b} = \mu \sigma \frac{t_1^2}{2b} (\ell - \frac{t_1}{3})
\]

(2)

where \(b\) is the block height.

The moment capacity in vertical bending per unit length is given by (see Figure 5.20):

\[
M\textsubscript{H} = \sigma t_1 d_1
\]

(3)

where \(d_1\) is the lever arm. Thus, \(\mu = \frac{2bd_1}{t_1(\ell - t_1/3)} \frac{M\textsubscript{vb}}{M\textsubscript{H}}\)

(4)
In our case $t_1 = 1.5$ in.

$\lambda = 8$ in.

$d^1 = 3.5$ in.

$b = 8$ in.

Therefore the coefficient of friction $\mu$ required would be:

for $M_{vb} = 3 M_H$ $\mu = 14.9$ and for $M_{vb} = 6 M_H$ $\mu = 29.9$

These values are very large and obviously not realistic. If we elaborate a bit more on our assumption, we can argue that because of horizontal moments, the contact area is essentially reduced to a line, and the shear stresses act only in a direction perpendicular to the motion (Figure 5.21(a)). The moment capacities in this case will be given as:

$$M_H = q d^1$$

$$M_{vb} = \mu q \frac{\lambda^2}{4b}$$

where $q$ is the vertical compressive load per unit length arising from vertical bending.

Therefore $\mu = \frac{4d^1 b M_{vb}}{\lambda 2 M_H}$

and thus for $\frac{M_{vb}}{M_H} = 3$, $\mu = 5.3$

and $\frac{M_{vb}}{M_H} = 6$, $\mu = 10.5$

These are also very high values.

We can also argue that because of the higher stiffness of the block around its webs, resistance to vertical bending ($M_H$) will be concentrated around those areas. Thus, most of the vertical load and the resulting
shear forces would be concentrated near the webs as indicated in Figure 5.21(b). Thus:

\[ M_H = 2f_v \frac{d^1}{l} \]

\[ M_{vb} = f_H \frac{b}{b} = \mu f_v \frac{b}{b} = \mu \frac{f_H}{2bd^1} \]

Therefore:

\[ \mu = \frac{2bd^1}{M_{vb}} \frac{M_{vb}}{M_H} \]

where \( f_v \) is the vertical compressive force arising from vertical bending as indicated in Figure 5.21(b).

For \( \frac{M_{vb}}{M_H} = 0.3 \), \( \mu = 4.7 \)

and for \( \frac{M_{vb}}{M_H} = 6 \), \( \mu = 9.4 \)

and again the values obtained are high.

From the above discussion, it would seem that a \( \frac{M_{vb}}{M_H} \) ratio of 6 would not be a realistic value, and that even a ratio of \( \frac{M_{vb}}{M_H} = 3 \) is high. The resulting stress distribution would probably be a combination of the last two cases discussed, i.e., a contact area reduced to a line and vertical and shear load concentration around the webs of the sliding blocks. We can expect an increase in the vertical compression in the areas where the blocks slide over one another, as the shear displacement would require dilation in the mortar or at the mortar block interface. (See Fig. 5.21(c)). This would produce a higher \( \frac{M_{vb}}{M_H} \) ratio for a given coefficient of friction. It is evident that we need a better assessment of the stress distribution across the wall section in order to estimate a value for the \( \frac{M_{vb}}{M_H} \) ratio.
On the other hand, the average self-weight of the walls at midheight is about 200 \( \text{lb/ft} \) \((2.92 \text{ kN/m})\) which would provide a horizontal resisting moment capacity of approximately 60 \( \text{lb/ft} \) \((0.27 \text{ kNm/m})\). Following the assumption that \( M_{vb} = 3 M_H \) this leads to \( M_{vb} = 180 \text{ lb/ft} \) \((0.81 \text{ kNm/m})\) which is close to the value used for calculations in Table 5.11. For those walls with vertical main steel, the assumed moment capacity \( M_{vb} = 460 \text{ lb/ft} \) \((2.07 \text{ kNm/m})\) would imply, using the same argument as in the previous paragraph, an average vertical force in the horizontal joints of 550 \( \text{lb/ft} \) \((8.03 \text{ kN/m})\), corresponding to an effective stress in the main steel of only 3000 psi \((20 \text{ MPa})\), which is not unreasonable.

5.5 Shear and Bond (Vertical Spans)

A. Hamid et al. (Ref. 41) found that the shear strength of ungrouted masonry was not strongly related to either the compressive strength of the mortar or the compressive strength of the masonry assemblage, but rather to the physical properties of both mortar and blocks such as surface roughness and initial rate of absorption. Grouting was shown to significantly increase the shear strength of masonry joints and the authors concluded that along with the level of the normal compressive stress, the shear strength of the grouted cores is the dominant factor in the shear strength of grouted concrete masonry.

Using a regression analysis the authors predicted the shear strength for concrete masonry as follows:
\[ \tau_{an} = 76 + 1.07 \sigma_n \text{ [psi] for ungrouted masonry (net area)} \]
\[ \tau_{ag} = 114 + 1.08 \sigma_n \text{ [psi] for grouted masonry with weak grout (gross area)} \]
\[ \tau_{ag} = 156 + 1.54 \sigma_n \text{ [psi] for grouted masonry with strong grout (gross area)} \]

where \( \sigma_n \) = normal compressive stress

In the University of B.C. series of tests, Wall [1] failed twice in what seemed to be a combination of bond and shear failure along the top course. In order to have an idea of the shear stresses sustained by this wall, an effective shear area as shown in Figure 5.22 was assumed. Table 5.III shows the shear stress values obtained for wall [1]. To give an idea of the stress level achieved, the values were compared to an ultimate shear stress of \( \tau_u = 3.5 \sqrt{f'_c} \text{ (psi)} \) as defined in Ref. 42 for the ultimate shear stress required for the formation of tension cracks in concrete beams in the region of high shear and low bending moments. In our case \( f'_c \) is assumed to equal the compressive strength of the grout. Comparison was also made with the minimum strength for strong grout of 156 psi (1.08 MPa) as predicted by Hamid et al. (Ref. 41).

The anchorage behaviour of grouted masonry depends on the bond-slip relationship of the bars and grout, and on the resistance of the grout to splitting. It also depends on the confining effects of the blocks in preventing splitting of the grout core. Cheema and Klinger (Ref. 42), recommend caution when applying concrete anchorage data to grouted masonry, particularly when large-size bars are used. Despite this, the bond stress was calculated using the methods of reinforced concrete (Refs. 26, 27, 42: \( u = V/\sum jd \)). Reference 42 gives the ultimate bond stress as \( u_{cr} = 11\sqrt{f'_c}/d \).
(psi), where again \( f' \) is taken as the compressive strength of the grout.

Table 5.IV shows the bond stresses calculated for wall [1] for the section between the top two courses and their comparison to the ultimate stress as defined above.

If we assume that the ultimate values used in Tables 5.III and 5.IV are correct, the fact that neither the shear nor the bond stresses reached these values seems to confirm the observed combined mode of failure.

Unfortunately we do not have enough data to develop an interaction equation to analyze our results in a more accurate way. Shear and bond failure are brittle types of failure, therefore they have to be prevented. Thus, walls [6], [7] and [8] were constructed with a bond beam in their upper course, and all of them failed in a different mode than wall [1].
### Table 5.1 DETAILS AND RESULTS OF TESTS ON MASONRY WALL PANELS

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Dimensions, reinforcing, supports</th>
<th>Maximum Pressure, Reinforcement ratio</th>
<th>Mode of Failure</th>
<th>Major crack pattern at failure</th>
<th>Central deflection (inches) vs Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. 96&quot; x 28&quot;</td>
<td>20&quot; x 48&quot; x 28&quot;</td>
<td>$P_{\text{max}} = 310$ psf, $\rho_v = 0.0011$</td>
<td>Sudden shear and bond failure at top course.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. same as 1.</td>
<td>same</td>
<td>$P_{\text{max}} = 250$ psf</td>
<td>Bending failure of cantilever portion about vertical reinforcing.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. 96&quot; x 29&quot;</td>
<td>19&quot; x 48&quot; x 29&quot;</td>
<td>$P_{\text{max}} = 160$ psf, $\rho_H = 0.0011$</td>
<td>Bond failure of top bar allowing cantilever type failure of top portion.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*After removing the top course, wall (1) was tested again. It failed in the same mode at a pressure of 365 psf.*
Table 5.1 (cont'd)

4. 96''
- Bending failure at one side cantilever.

5. 96''
- Bending failure of masonry between reinforcement.

6. 92''
- As in wall 5.

7. 92''
- Bending failure of masonry between reinforcement reached yield.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Description</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Table 5.I (cont'd)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bond failure of #4 bar near midheight. Vertical reinforcement reached yield.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Bond failure of #4 bar near midheight. Vertical reinforcement reached yield. |

| Shear failure along top course then bending mechanism in blocks. |

| Joint reinforcement |

| One way bending at top changing to bending mechanism near base. |
Table 5.1 (cont’d)

| 12. | 18” | \( P_{\text{max}} = 130 \text{ psf} \) |
|     |     | \( \rho_H = 0.0005 \) One way bending. |

| 13. | 16” | \( P_{\text{max}} = 120 \text{ psf} \) Mostly one way bending. Very extensive cracking. |

| 14. | 10” | \( P_{\text{max}} = 180 \text{ psf} \) Bending mechanism. Very extensive cracking. |

\( \rho_y = 0.0011 \)
\( \rho_H = 0.00025 \)
TABLE 5.II - YIELD ANALYSIS RESULTS

<table>
<thead>
<tr>
<th>Wall</th>
<th>Yield Line</th>
<th>Direction of Main Reinforcing</th>
<th>Horizontal Joint Reinforcing</th>
<th>Yield Line Equation</th>
<th>Load at Large Deformation - psf</th>
<th>West†</th>
<th>Error+</th>
<th>West***</th>
<th>Error***</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>3/8&quot;</td>
<td>vertical</td>
<td>none</td>
<td>( W = \frac{1}{3.725}(M_v + 0.25M_H) = W )</td>
<td>190</td>
<td>200</td>
<td>+5%</td>
<td>199</td>
<td>+5%</td>
</tr>
<tr>
<td>6</td>
<td>3/8&quot;</td>
<td>horizontal</td>
<td>none</td>
<td>( W = \frac{1}{5.22}(M_v + 2M_H) = W )</td>
<td>210</td>
<td>200</td>
<td>-5%</td>
<td>199</td>
<td>-5%</td>
</tr>
<tr>
<td>10</td>
<td>3/8&quot;</td>
<td>horizontal</td>
<td>none</td>
<td>( W = \frac{1}{6.67}(M_v + 0.5M_H) = W )</td>
<td>83</td>
<td>83</td>
<td>0</td>
<td>88</td>
<td>+6%</td>
</tr>
<tr>
<td>11</td>
<td>3/8&quot;</td>
<td>horizontal</td>
<td>( M_{VR} )</td>
<td>( W = \frac{1}{6.67}(M_v + 0.5M_H) = W )</td>
<td>83</td>
<td>83</td>
<td>0</td>
<td>88</td>
<td>+6%</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>vertical</td>
<td>( \frac{1}{4}(M_v + 2M_H) = W )</td>
<td>130</td>
<td>125</td>
<td>-4%</td>
<td>130</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>vertical</td>
<td>( \frac{1}{4}(M_v + 2M_H) = W )</td>
<td>130</td>
<td>125</td>
<td>+4%</td>
<td>130</td>
<td>+4%</td>
<td>+4%</td>
</tr>
<tr>
<td>7</td>
<td>same as 5</td>
<td>vertical</td>
<td>( \frac{1}{4}(M_v + 2M_H) = W )</td>
<td>130</td>
<td>125</td>
<td>-4%</td>
<td>130</td>
<td>-4%</td>
<td>-4%</td>
</tr>
<tr>
<td>14</td>
<td>3/8&quot;</td>
<td>vertical</td>
<td>( \frac{1}{4}(M_v + 2M_H) = W )</td>
<td>130</td>
<td>125</td>
<td>-4%</td>
<td>130</td>
<td>-4%</td>
<td>-4%</td>
</tr>
</tbody>
</table>

*\( M_{VR} \) - vertical moment capacity of joint reinforcement spaced at 16 o.c. \( (p = 0.00025) \) lb. in/in.
†W from equation in col. (5) using \( M_{VB} = 460 \) or 190 lb., \( M_{VR} = 333 \) lb. and \( M_H = 0.33 M_{VB} \)
**W from equation in col. (5) using \( M_{VB} = 510 \) or 235 lb., \( M_{VR} = 333 \) lb. and \( M_H = 0.167 M_{VB} \)
### TABLE 5.III

Shear Stresses - Wall [1]

<table>
<thead>
<tr>
<th>Wall</th>
<th>Max. Load</th>
<th>Max. Shear Stress ( v = \frac{V}{A_H} )</th>
<th>( \frac{V}{3.5\sigma_{gr}} )</th>
<th>( \frac{V}{156 \text{ psi}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1a)</td>
<td>310 psf (14.8 kPa)</td>
<td>130 psi (0.38 MPa)</td>
<td>0.66</td>
<td>0.83</td>
</tr>
<tr>
<td>(1b)</td>
<td>365 psf (17.5 kPa)</td>
<td>135 psi (0.39 MPa)</td>
<td>0.69</td>
<td>0.87</td>
</tr>
</tbody>
</table>

\( A_H = \text{effective shear area, (as shown in Fig. 5.22).} \)

\( \sigma_{gr} = \text{compressive strength of grout (psi)} \)

### TABLE 5.IV

Bond Stresses - Wall [1]

<table>
<thead>
<tr>
<th>Wall</th>
<th>Load</th>
<th>Stress ( u(\text{psi}) )</th>
<th>( \frac{u}{u_{cr}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1a)</td>
<td>310 psf (14.50 kPa)</td>
<td>500 (3.45 MPa)</td>
<td>0.60</td>
</tr>
<tr>
<td>(1b)</td>
<td>365 psf (17.5 kPa)</td>
<td>530 (3.66 MPa)</td>
<td>0.64</td>
</tr>
</tbody>
</table>

\( u = \frac{V}{\Sigma_0 jd} \)

\( V = \text{shear force} \)

\( \Sigma_0 = \text{sum of reinforcing bars perimeter} \)

\( jd = \text{lever arm for bending internal forces} \)

\( u_{cr} = \text{ultimate bond stress} = \frac{11f'_c}{d_b} \) (psi)

\( f'_c = \text{taken as} \) \( \sigma_{gr} = \text{compressive strength of grout (psi)} \)

\( d_b = \text{bar diameter} \)
(a) Vertically Spanning Wall

- wood reaction wall
- anchors at 8 in (200 mm)
- rotary potentiometer
- taut wire
- air bag
- wooden support beam

(b) Horizontally Spanning Wall

- anchors at 8 in (200 mm)
- teflon base pads

Figure 5.1 Test Arrangement
Figure 5.2 Central Displacement vs Load - Wall (14)

Figure 5.3 Central Displacement vs Load - Wall (9)
Figure 5.4 Central Displacement vs. Load - Wall (10)

Figure 5.5 Central Displacement vs. Load - Wall (11)
Figure 5.6 Central Displacement vs. Load - Wall (12)

Figure 5.7 Central Displacement vs. Load - Wall (13)
Figure 5.8(a) Displacement of the Centre Point of the Wall relative to the top and bottom displacements.

Figure 5.8(b) Displacement of the centre point of the wall relative to the edge displacements.

Figure 5.8 Central point displacement in a horizontal and in a vertical plane (Wall 10).
Figure 5.9(a) Displacement of the centre point of the wall relative to the top and bottom displacement.

Figure 5.9(b) Displacement of the centre point of the wall relative to the edge displacements.

Figure 5.9 Central Point Displacement in a Horizontal and in a Vertical Plane (Wall 11).
Figure 5.10: Wall (11) Load vs. Central Displacement for one Complete Cycle.
Figure 5.11 Load-Strain Plot for Reinforcing Steel in Wall (4)  
(Test results for two bars plotted)
Figure 5.12 Load-Strain Plot for Reinforcing Steel in Wall (6)
(Test results for two bars plotted)
Figure 5.13 Load-Strain Plot for Reinforcing Steel in Wall (7)
(Test results for one bar plotted)
Figure 5.14 Load-Strain Plot for Reinforcing Steel in Wall (8)
(Test results for one bar plotted)

- $t = 7.625 \text{ in}$
- $H = B = 96 \text{ in}$
- $A_s = 0.88 \text{ in}^2$
- $f'_m = 2,000 \text{ psi}$
- $E_m = 1,000 f'_m$
- $E_s = 29,000,000 \text{ psi}$
Figure 5.15  Load-Strain Plot for Reinforcing Steel in Wall (14)
(Test results for one bar plotted)

- $t = 7.625$ in
- $H = B = 96$ in
- $A_s = 0.88$ in$^2$
- $f'_m = 2,000$ psi
- $E_m = 1,000 f'_m$
- $E_s = 29,000,000$ psi
Figure 5.16  Load-Strain Plot for Reinforcing Steel in Wall (9)
(Test results for three bars plotted)
Figure 5.17 Load-Strain plot for Reinforcing Steel in Wall (10)
(Test results for two bars plotted)

- t = 7.625 in
- B = 96 in
- H = 80 in
- As = 0.88 in^2
- f'_m = 2,000 psi
- E_m = 1,000 f'_m
- E_s = 29,000,000 psi
Figure 5.18 Mechanism of Failure for Masonry Wall Spanning Horizontally.

Figure 5.19 Shear Stress Distribution on Shear Area

Figure 5.20 Section in Vertical Bending
(a) Contact Area Reduced to a Line

(b) Force Concentration Around Webs

(c) Effects on vertical compression due to rotation of the blocks.

Figure 5.21 Horizontal Bending Mechanism
Figure 5.22. Effective Shear Area for Out-Of-Plane Forces.
In this thesis a study of the behaviour of masonry walls under combined in-plane and lateral loading was presented. It included a discussion of previous research work and the approaches of present codes of practice. The properties of masonry components and assemblages were examined, and the tests aimed at determining the properties of the materials for the walls tested at the University of British Columbia were reported. An experimental program on laterally loaded non-loadbearing wall panels has been described, which will eventually lead to the basis for the design of the spacing for the main steel and the appropriate amount and spacing of the distribution steel. Findings based on the results of the experimental program and their analysis, can be summarized as follows.

(1) Based on the results of the tests the present code requirements on spacing seem to be very conservative.

(2) The wall specimens showed a much greater strength than is required for earthquake code forces applied to eight foot (2.44 m) spans such as were tested.

(3) Joint reinforcement appears to be effective as distribution steel for vertical reinforcement, and also as main steel for horizontal spans. It appears to provide moment capacities corresponding to a stress nearly equal to its yield strength.

(4) Load-strain curves for the main steel bars in walls spanning vertically showed good agreement between current analysis methods and experimental data.
(5) Most wall panels failing in flexure showed ductile behavior. Yield line analysis, which can be justified because of the observed ductile behaviour of the walls, appears to be able to predict the lateral load capacity of masonry to span between lines of support or reinforcement. However, the moment capacity of the masonry is highly dependent on in-plane loads which arise from supports or confining steel, making an exact moment capacity prediction very difficult. Lateral loads predicted for moment capacity ratios $\frac{M_{vb}}{M_h}$ of 3 and 6 did not show any significant difference. However a friction analysis seems to indicate that a moment capacity ratio of 3 is a more feasible assumption.

(6) The code procedures to determine the properties of masonry materials and assemblages have been shown, in other research programs, to be unsatisfactory in reproducing the properties of masonry walls and their components. Although some variations in these procedures were introduced in the material tests at the University of B.C., further research is necessary in order to draw some conclusions as to their adequacy.

Further research in this program is being aimed at determining the behaviour of masonry walls under out-of-plane quasi-static cyclic loading. The results obtained in this second stage of the University of British Columbia research program will lead to a final phase in which masonry walls will be tested under simulated earthquake loading on a shaking table.


29. Canadian Standard Association, CSA Standard A.165.1, Concrete Masonry Units.


53. Recommended Testing Procedures for Concrete Masonry Units, Prisms, Grout and Mortar, California Concrete Masonry Technical Committee.