

**SEISMIC DESIGN OF MASONRY BUILDINGS –
BACKGROUND TO THE DRAFT MASONRY DESIGN CODE DZ4210**

Dedicated to the late O.A. Glogau

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SUMMARY:

Background material to seismic design aspects of the draft masonry design code DZ4210 is presented. The design approach is based on specified lateral force levels appropriate to the available but limited ductility, and on the principles of reinforced concrete section analyses adapted for low material strengths. Ultimate masonry strengths for compression, shear and flexure are based on the construction techniques and extent of supervision rather than on the strengths of the masonry constituents. Design lateral force coefficients for flexural strength depend on the characteristics of the structural system adopted and Structural Type factors (S) are proposed that are more appropriate to masonry structures than current values incorporated in the Loadings Code NZS4203. Shear failure is proscribed by the implementation of capacity design principles, though simplified procedures are allowed for structures with high flexural S factors. Brief discussion is made of so-called non-structural masonry, including veneers, partition, infill and secondary walls.

1. INTRODUCTION:

Masonry construction has in New Zealand as in many countries, a poor reputation for seismic performance. It is nevertheless widely used in housing construction in the form of unreinforced veneer walls, and for commercial buildings in the forms of reinforced hollow masonry and the concern felt about seismic performance of masonry structures, until recently structural masonry has enjoyed little seismic related research activity. This has been reflected in New Zealand and elsewhere in design methods and codes of practice which have generally been based on traditional rather than modern approaches, specifying elastic design, and treating structural masonry as a low grade concrete, without recognition of the special limitations imposed by material behaviour and the structural forms utilised in masonry construction. The current New Zealand masonry design code (NZS1900 ch. 9.2¹) is so out of date as to have been practically useless as a design document for some years, leaving the design profession without a viable basis for masonry design.

In recent years an attempt has been made to rectify this situation. Considerable research activity has focused on the strength and ductility of structural masonry under simulated seismic loading (e.g. Refs. 2-6), and after an unfortunately long gestation period, a draft masonry design code⁽⁷⁾ has been prepared whose seismic design provisions are based on a detailed assessment of the probable performance of masonry structures under seismic attack. This paper will discuss the background to this document.

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2. FORMS OF MASONRY CONSTRUCTION COMMON IN NEW ZEALAND:

Fig. 1 illustrates the four main forms of masonry construction used in New Zealand. Reinforced Cavity Masonry (RCM, Fig. 1a) consists of two wythes, generally of clay brick, separated by a reinforced Hollow Masonry (RHM) using concrete masonry units with elevation dimensions 390 x 190 mm, and thickness from 90 mm to 290 mm is the most common structural use of masonry. Fig. 1b illustrates open-end bond-beam RHM construction preferred for construction in regions of high seismicity because of the improved continuity within the grout core, and the ease of placing horizontal reinforcement. Masonry infill (Fig. 1c) in reinforced concrete or structural steel frames is not recommended for seismic resistance unless adequately reinforced, and designed for very limited ductility. Fig. 1d illustrates veneer construction which is common for housing and light buildings. The veneers may be unreinforced or reinforced, and tied back to a variety of structural backings.

3. MATERIAL STRENGTHS AND GRADES OF MASONRY:

Masonry consists of four separate materials: masonry unit, mortar, grout and reinforcement. Minimum specified constituent strengths are^(7,8) Concrete Masonry units: 14 MPa on net area, Clay Brick units: 20 MPa, Mortar 12.5 MPa, Grout 17.5 MPa, Reinforcement 275 MPa yield strength. However, though masonry constituent strengths are specified, it is difficult to translate this into a minimum effective strength of masonry in-situ. Masonry compression strength can be dictated by a number of different failure mechanisms, dependent on the

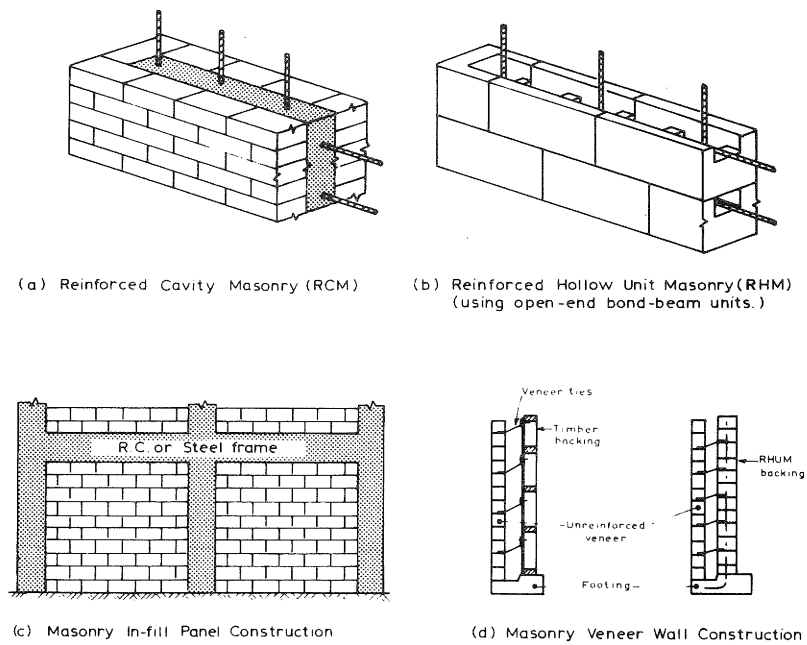


Fig. 1 – Common Forms of Masonry Construction

TABLE 1 - DESIGN STRESSES FOR STRENGTH DESIGN
(MPa)

Type of Stress	Grade of Masonry		
	C	B	A
Max. f'_m to be assumed for design	4	8	8 **
Shear:			
Max shear stress provided by masonry	0.12	0.24	0.30
Max total shear stress:	0.50	1.00	1.20
or where reduced ductility of ductile cantilever walls is assured:	N.A.	N.A.	$1.2 + 0.4(R-1) \leq 1.6$
Modulus of Elasticity, E	6,000	6,000	6,000
Modulus of Rigidity, G	2,500	2,500	2,500

* $R = \frac{\text{dependable flexural strength provided in design}}{\text{dependable flexural strength required by code}}$

** A higher value for f'_m may be assumed provided the value is substantiated by prism tests.

relative stiffness and brittleness of the grout and the masonry-unit/mortar system⁽⁹⁾. The strength and brittleness of the masonry-unit/mortar system is a function of the mismatch of strengths of the two constituents, and is generally dictated by the biaxial tension strength of the masonry unit rather than by any compression strength parameters⁽¹⁰⁾.

More important, however, is the influence of workmanship and supervision. In particular, a masonry element constructed without cleaning out mortar droppings and with inadequate or no vibration of grout is likely to have low strengths in critical regions at the base where seismic moments and shears are likely to be a maximum. Unlike reinforced concrete, where such defects are likely to be apparent on stripping the formwork, the masonry units hide the state of the grout unless cores are taken. Limited testing of walls constructed without supervision, and poorly vibrated as a consequence, indicated significant loss of strength and capacity for energy dissipation under simulated seismic loading when compared with sound walls⁽⁵⁾. Fig. 2 shows the grout core of one such wall after face shells had spalled during inelastic cyclic testing. Clearly compression strength of such a wall will be lower than indicated by constituent properties, and corrosion of reinforcement would be a major problem. Substandard positioning of starter bars and wall reinforcement have frequently been noted in masonry construction, and are very difficult to determine unless the wall is frequently inspected during construction.

Consideration of these factors has led to the specification in the draft masonry design code DZ4210⁽⁷⁾ (hereinafter called 'the draft code') of design levels of strength dependent on the degree of inspection and the construction method adopted, rather than on the constituent properties. Three grades of masonry construction are identified.

Grade C masonry, the minimum standard, is unsupervised and is intended only for structures not requiring specific design, or in special circumstances where inspection is impractical.

Grade B masonry, the standard grade, will be designed and inspected by an engineer experienced in this form of construction. Inspection of reinforcement placing, and grouting, is emphasised. End zones (potential plastic hinge regions) of Grade B masonry must be all-cells filled.

Grade A masonry, recognises that reliability and structural performance are improved when all cells are completely filled and work is closely supervised at all critical stages. More stringent inspection is required than for Grade B masonry to ensure quality construction throughout.

Table 1 lists design strengths specified in the code for the three grades of masonry. The standard value of $f'_m = 8 \text{ MPa}$ for Grade B masonry also applies to Grade A,

unless a higher value is supported by results from the testing of prisms laid at the same time as critical wall elements. In general the low compression strength is not a restriction, as masonry elements are typically under-reinforced, with low axial load levels. This results in small flexural compression stress blocks, and hence a lever arm for flexure that is comparatively insensitive to strength. Shear strengths specified in Table 1 are discussed in a later section.

4. DESIGN LATERAL SEISMIC LOADS FOR STRUCTURAL MASONRY:

Although many material codes still specify elastic design procedures for structural masonry under seismic loads, the levels of lateral loads specified are generally inadequate to ensure that behaviour remains elastic under the design level earthquake. Fig. 3a shows smoothed composite acceleration response spectra developed by Skinner⁽¹¹⁾ from eight California accelerograms scaled to El Centro 1940 N-S intensity, an accelerogram which is now accepted as having only moderate damage potential. Masonry structures, being stiff, typically have fundamental periods in the range 0.1 - 0.8s, thus spanning the frequency range of maximum response. Assuming a 5% equivalent viscous damping, peak elastic response of the order of 0.8g can be anticipated.

Seismic coefficients included in most loadings codes are based on ductile response, with structure ductility in the vicinity of four. Reference to Fig. 3b shows that elastic design to these load levels will still result in the ultimate capacity being attained, but with a reduction in the required structure ductility. Unless masonry structures are designed for the true elastic force levels likely to occur during the anticipated structural life, it is essential to recognise that ductile response will be required, and to design accordingly by ensuring that the materials and structural systems adopted are capable of sustaining the required ductility without excessive strength and stiffness degradation. This is inherent in the design approach specified by the Loadings Code NZS4203⁽¹²⁾, which gives the design lateral base shear as

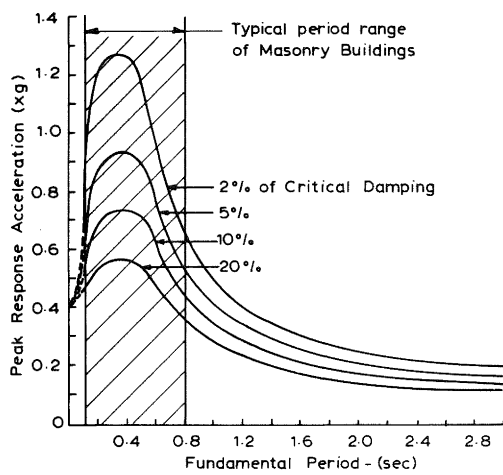
$$V = C.S.M.I.R. W \quad (1)$$

where C is the basic coefficient, obtained by dividing the design elastic response spectrum for 10% damping by a factor of 4, and then simplifying. Thus C corresponds to the design force for a displacement ductility factor of $\mu = 4$, based on the equal displacement principle, as illustrated in Fig. 3. S, M, I and R are structural type, material, importance and risk factors respectively and W is the seismic weight.

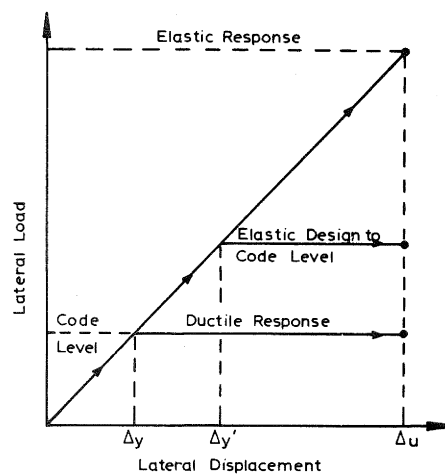
The two factors in Eq. 1 most relevant to masonry design are M and S. Because of the brittle nature of masonry compression failure, and reduced energy dissipation apparent in hysteresis loops of cyclically loaded masonry walls, a



Fig. 2 – Grout core of wall with unsupervised construction, after cyclic in-plane loading.



(a) Composite Response Spectra from Eight Accelerograms Scaled to El Centro 1940 N-S (after Skinner)



(b) Peak Lateral Load using "Equal-displacement" Principle.

Fig. 3 – Seismic loads for masonry walls.

materials factor of $M = 1.2$ is required by NZS 4203, requiring design for lateral loads 20% higher than for an equivalent reinforced concrete structure.

In reviewing the seismic design of masonry structures, the committee charged by SANZ with the responsibility of drafting the code found some anomalies when applying S factors specified by the Loadings Code. It became apparent that though reinforced masonry and reinforced concrete may be each built into similar structural forms, these would not necessarily perform equally well under seismic loading. For example, ductile frame structures are specified as $S = 1.0$ by NZS4203. Although this is quite satisfactory for a reinforced concrete frame, it would be extremely difficult to obtain the required plastic rotations implied by $S = 1.0$ from masonry beam members. The approach adopted in the draft code has been to list a series of over-riding S factors for use with masonry. While the draft is being reviewed by the design profession, discussions with the loadings code committee will hopefully resolve the discrepancies, and either list the masonry S factors in some future revision to NZS4203, or permit incorporation of S factors within materials codes. The writer's viewpoint is that structural type factors cannot be considered to be independent of material. Because this is a likely source of controversy, the basis for the proposed S factors is developed in some detail below, and discussion is invited.

It is emphasised that listing new S factors for masonry in the draft code does not imply that the committee felt that the material codes should be the source of specified S factors. The purpose of inclusion within the draft code has been to facilitate early publication of the draft for comment in the knowledge that it would have been incomplete without this information.

4.1 Cantilever Shear Walls -

Fig. 4 illustrates the masonry structural system preferred for ductile seismic response. Seismic loads are carried by simple cantilever shear walls. Where two or more such walls occur in the same plane, linkage between them is provided by flexible floor slabs (Fig. 4a) to ensure moment transfer between the walls is minimised. Openings within the wall elevation are kept small enough to ensure that the basic vertical cantilever action is not affected. Energy dissipation occurs only in carefully detailed plastic hinges at the base of each wall.

As Fig. 4 shows, the code divides cantilever shear walls into three categories: 'ductile' walls, squat walls, and a transition zone between ductile and squat walls. Ductile walls (Fig. 4a) are defined by an aspect ratio of wall height to wall length of $h_w/l_w > 2.0$. Such walls are expected, if properly detailed, to dissipate considerable amounts

of energy by flexural yielding in plastic hinge zones. Consequently structural type factors are set at the comparatively low levels of $S = 1.2$ for single wall in the principal direction being considered and $S = 1.0$ when more than one wall contributes to the strength in the direction considered. The 20% increase in S for a single wall is adopted to compensate for the lack of redundancy in the lateral load resisting system.

Squat walls (Fig. 4c) are defined by $h_w/l_w < 1.0$. Structures containing squat walls typically have low periods, and tests on masonry⁽³⁾ and concrete⁽¹³⁾ walls of low aspect ratios indicate that while flexural failure modes can be obtained, considerable strength and stiffness degradation results from a tendency for the walls to slide on their bases during inelastic cycling, increasing ductility demand. Squat walls are thus to be designed for the higher structural type factor of $S = 1.6$ regardless of whether there are one or more walls in the direction considered.

Walls in the transition zone with $1.0 \leq h_w/l_w \leq 2.0$ are to be designed for a linear variation of S between the 'ductile' and 'squat' limits. Thus for single walls

$$1.2 \leq S = 2.0 - 0.4 \frac{h_w}{l_w} \leq 1.6 \quad (2a)$$

for multiple walls

$$1.0 \leq S = 2.2 - 0.6 \frac{h_w}{l_w} \leq 1.6 \quad (2b)$$

This removes the step function in S at $\frac{h_w}{l_w} = 2.0$ currently in the loadings code, and is in accordance with changes adopted in the draft concrete design code (14).

4.2 Shear Walls with Large Openings -

Traditional masonry construction has generally consisted of peripheral masonry shear walls pierced by window and door openings, as idealised in Figs. 5a and 6a. Under lateral loading hinging may initiate in the piers (Fig. 5a) or the spandrels (Fig. 6a). In the former, and more common case, the piers will be required to exhibit substantial ductility unless designed to resist elastically the displacements resulting from the design earthquake. Plastic displacement (flexural or shear) will inevitably be concentrated in the piers of one storey, generally the lowest, with consequential extremely high ductility demand at that level.

Consider the deflection profiles at first yield, and ultimate illustrated in Fig. 5b. If design is on the basis of a specified or implied ductility μ at roof level, plastic displacement

$$\Delta_p = (\mu - 1) \Delta_y \quad (3)$$

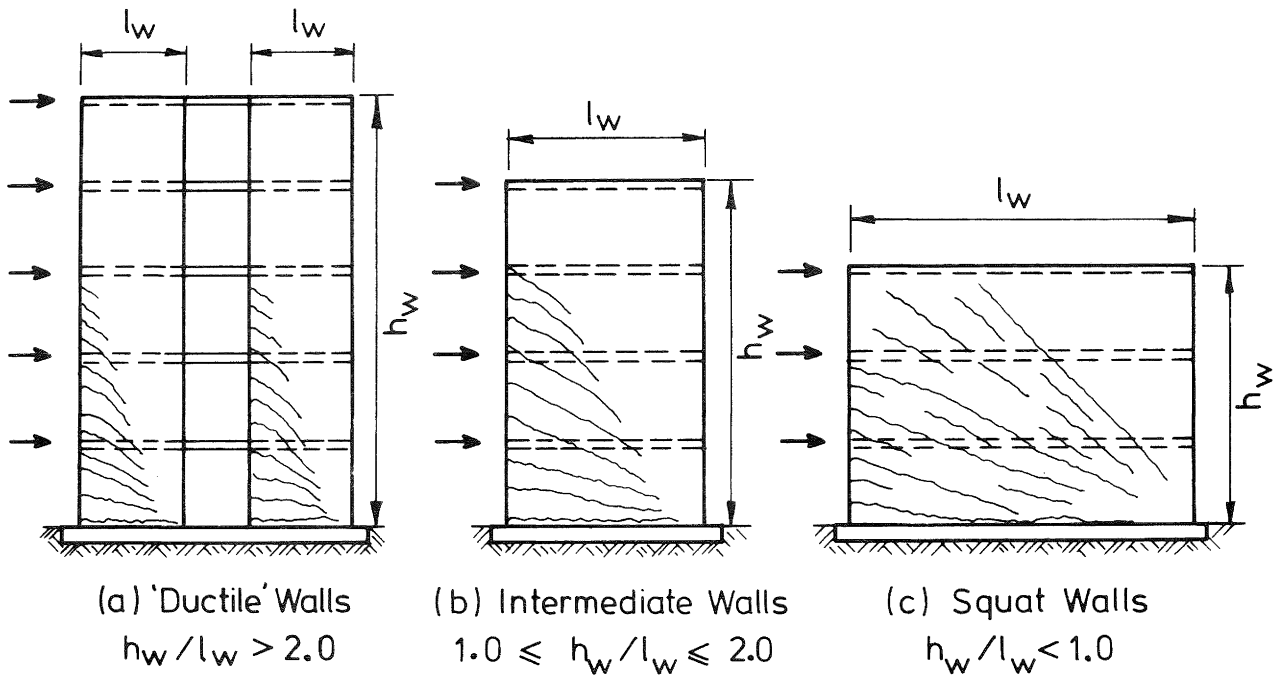


Fig. 4 – Aspect ratios of simple cantilever shear walls.

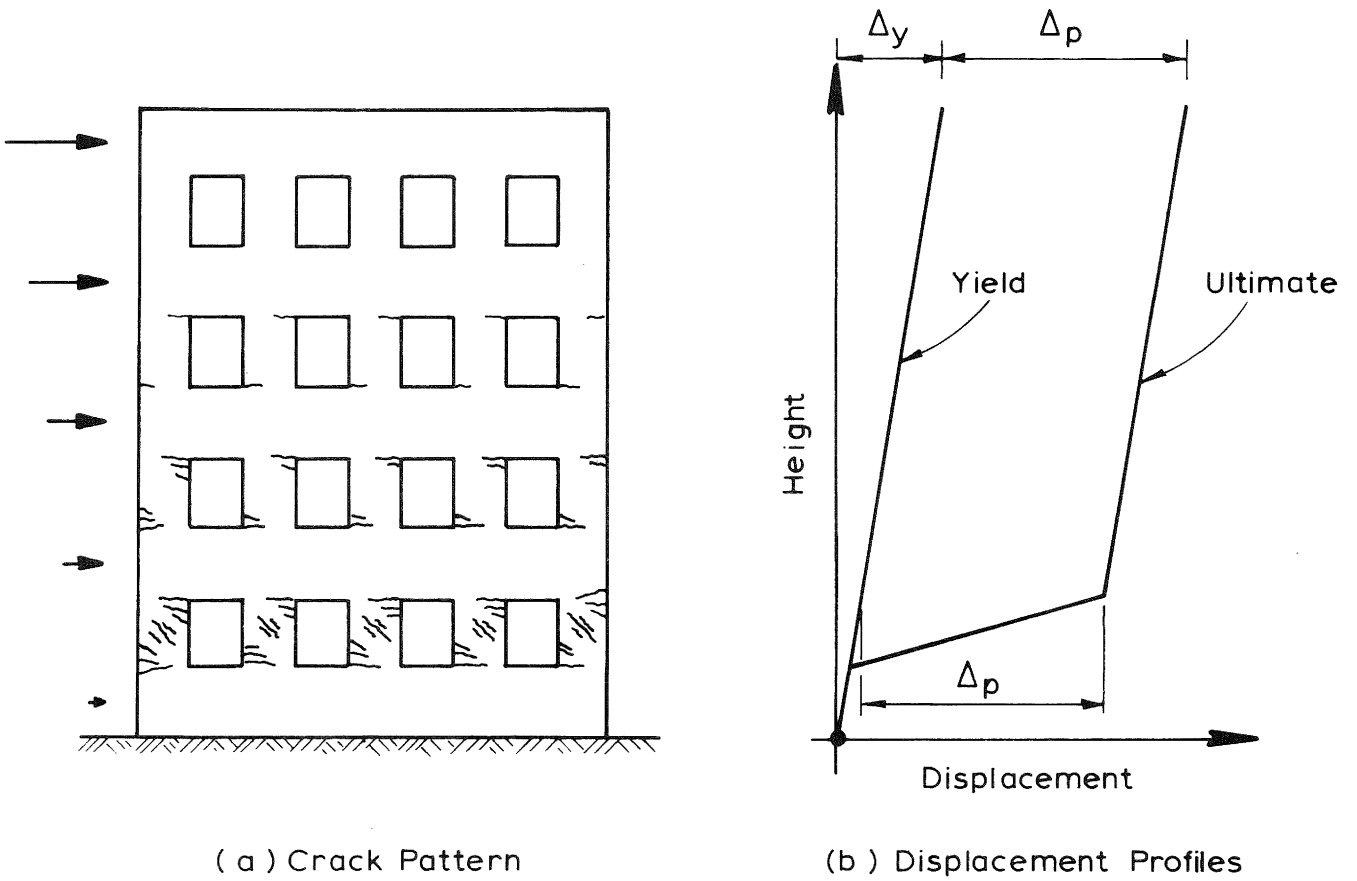


Fig. 5 – Shear wall with pier ductility.

is required. Under a typical triangular distribution of lateral seismic loads (Fig. 5a) the yield load will be attained when the flexural or shear strength of the bottom level piers is reached. Other elements of the structure will not have yielded at this stage. Subsequent deformation of the yielding piers will occur at constant or decreasing lateral load, thus ensuring that the other structural elements are protected from inelastic action, and concentrating the plastic deformation Δp in the yielding piers. If the structure has n storeys and pier height is half the storey height, then the elastic displacement over the height of the piers at yield will be

$$\Delta y_1 = \frac{\Delta y}{2n} \quad (4)$$

assuming a linear yield displacement profile. From Eqs. 3 and 4 the displacement ductility factor μ_1 required of the pier will then be

$$\mu_1 = 2n (\mu - 1) + 1 \quad (5)$$

Thus for a 10 storey masonry shear wall designed for a structural displacement ductility factor $\mu = 4$, the ductility required of the piers would be $\mu_1 = 61$. Extensive recent experimental research on pier units at the University of California, Berkeley⁽¹⁵⁾ has indicated extreme difficulty in obtaining reliable ductility levels an order of magnitude less than this value. It is concluded that the structural system of Fig. 5a, with ductile piers, is only suitable if very low structural ductilities are required.

Consequently, the draft masonry code⁽⁷⁾ requires pierced shear walls with potential pier ductility to be designed for a structural type factor $S = 4$, implying near-elastic response, unless a rational analysis is carried out limiting the ductility of the pier units to that implied by equation 2b. Where the pier aspect ratio exceeds 2, the example of n regular storeys discussed above would result in an S factor for the structure as a whole of

$$S = \frac{8n}{3 + 2n} \quad (6)$$

Occasionally openings in masonry walls will be of such proportions that spandrels will be relatively weaker than piers, and behaviour will approximate coupled shear walls, with crack patterns as illustrated in Fig 6a. Well designed coupled shear walls in reinforced concrete constitute an efficient structural system for seismic resistance⁽¹⁶⁾. However, to satisfy the high ductility demand generated in the spandrel beams diagonal reinforcement is generally required. Such a system is unsuitable for structural masonry and rapid strength and stiffness degradation is likely, resulting in an increase in wall moments towards those

appropriate to simple linked cantilevers (see Fig. 6b). If the wall moment capacities have been proportioned on the basis of ductile coupled shear wall action, then the moment increase implied by Fig. 6b will not be possible, and the consequence will be excessive ductility demand at the wall-base plastic hinges. To ensure satisfactory performance from coupled masonry shear walls the draft masonry code requires the strength of the spandrels to be ignored, and the walls to be proportioned for the wall moment capacities on the basis of the simple linked cantilever moments in Fig. 4a. To minimise damage to the spandrels, the flexural strength is based on a structural type factor of $S = 2.4$.

It will be noted that the high S factors specified for both categories of shear walls with openings make them unattractive to the designer, who is encouraged to adopt the structurally more dependable linked cantilever approach of the previous section.

4.3 Masonry Infilled Frames -

Masonry infill has been a common cause of poor performance of reinforced concrete frames under seismic loading, generally because the infill has been considered to be non-structural, and its effect on structural action ignored. The result has all too often been an unexpected shear failure of the columns, particularly when the infill has not extended the full storey height. It must be recognised that masonry infill panels modify the structural behaviour of the containing frame under lateral load, unless sufficient separation is provided at top and sides to allow free deformation of the frame to occur, in which case the panel is designed as a partition, to carry face-loads only. It should be noted however, that even where sufficient separation is provided at top and ends of a panel, the panel will still tend to stiffen the supporting beam considerably, concentrating frame plastic hinges in short hinge lengths at each end, or forcing migration of hinges into columns, with a breakdown of the weak-beam strong-column concept. Consequently the use of masonry partitions should be limited to shear wall buildings unless the partitions are supported on slabs at a sufficient distance from beams to ensure normal frame action is not affected.

The S factor specified in the draft code for masonry infilled structures depends on the expected failure mode and the number of storeys. If the infill is sufficiently reinforced for shear so that infill and frame act together in composite fashion as a shear wall with boundary elements, with a flexural hinging mode, a value of $S = 1.6$ is adopted. If, however, the more common failure mode involving shear failure or diagonal compression of the infill is predicted, a minimum S factor of 2.4 is specified to ensure ductility demand is low, and the consequences of the ensuing strength and stiffness degradation are not too severe.

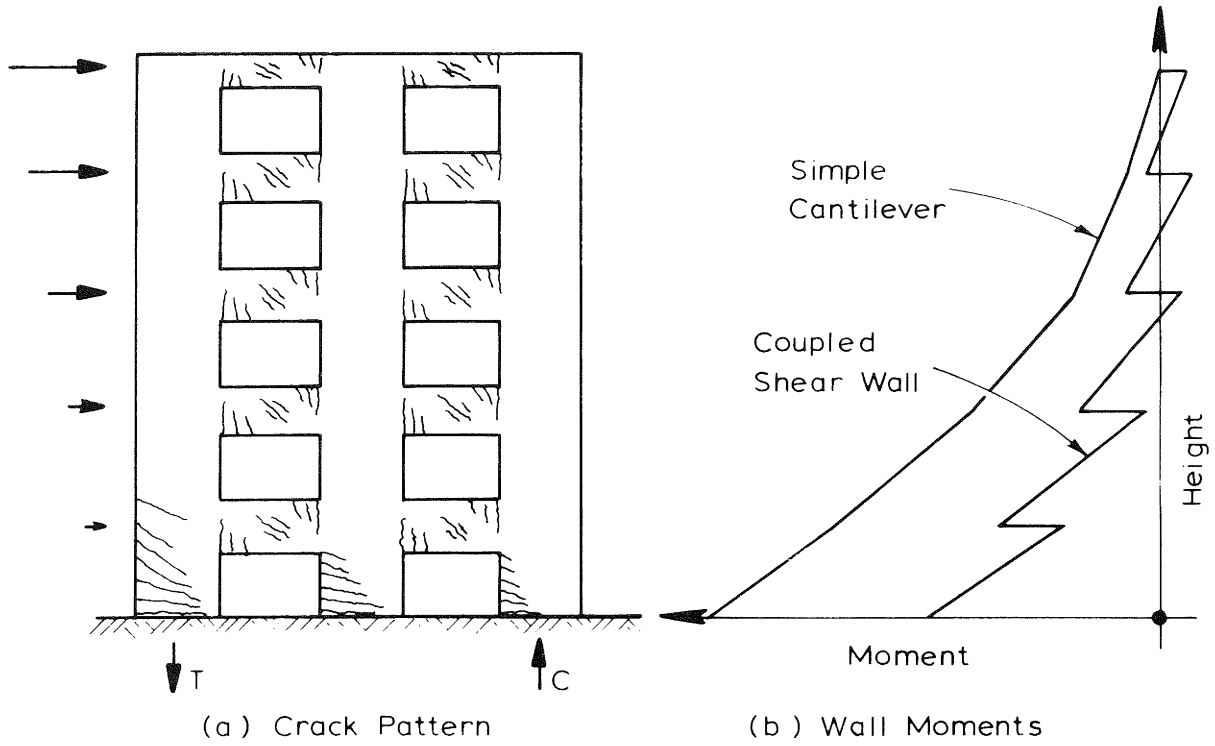


Fig. 6 – Coupled Masonry Shear Wall.

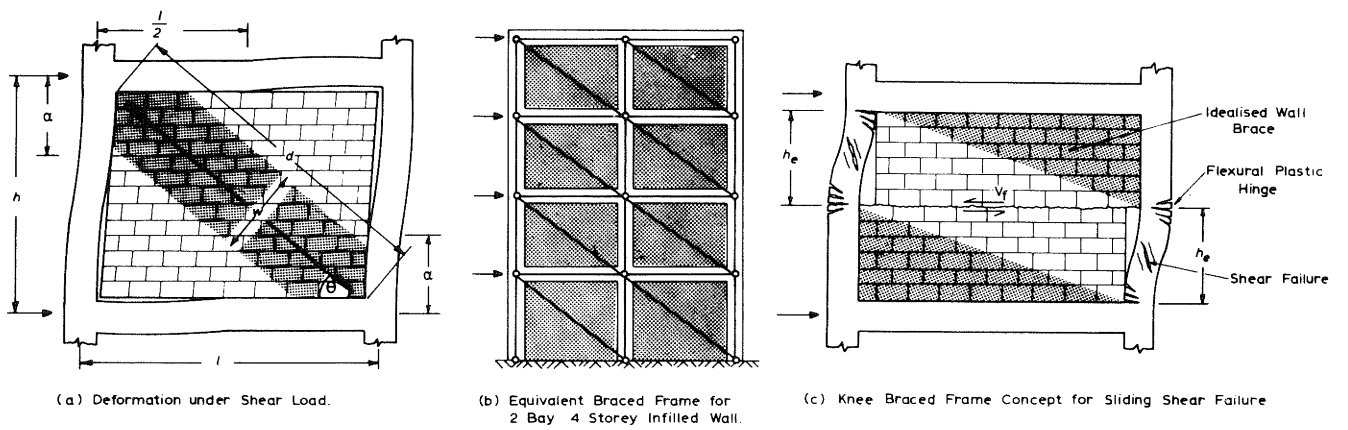


Fig. 7 – Equivalent diagonal bracing action of infill.

For multistorey structures, $S = 4$ is specified unless a rational analysis is carried out to limit ductility at any level to that implied by $S = 2.4$ for a single storey structure. As the failure mechanism invariably results in a soft-storey sway mechanism, the approach used above for pierced shear walls with pier ductility may be used to determine an S factor for a regular n storey structure as

$$S = \frac{4n}{n + 0.67} \quad (7)$$

In all cases the infill must be reinforced and designed to carry the face-loads resulting from seismic response. The composite action of frame and infill must be considered in analysis. Structural stiffness is greatly increased, and natural period reduced, which is significant to the determination of the appropriate basic seismic coefficient. For purposes of stiffness and force distribution calculations, the infilled frame may be represented by an equivalent diagonally braced frame (Fig. 7) where the effective width of the diagonal masonry strut is one quarter of its length. Reference 17 gives details on the design and analysis of masonry infilled frames.

4.4 Masonry Frames -

Although reinforced masonry is an accepted material for shear wall construction, it has never been promoted as a suitable material for ductile frames. It would be extremely difficult to detail masonry beams to sustain the curvature ductilities required for ductile frame response. Face shells could be expected to spall early in the response, with considerable hazard to building occupants, and flexural reinforcement would buckle as a result of a lack of adequate lateral support. It is expected that the strength of a ductile masonry frame would degrade rapidly under cyclic inelastic loading.

These considerations have resulted in a requirement that masonry frames be designed for $S = 4$, implying near elastic response. An exception is made for one and two storey masonry frames, which may be designed for $S = 2.4$, provided plastic hinging is designed to occur in columns and the beams are designed to remain elastic by a capacity design approach.

4.5 Upper Limit to S Factor

It will be noted that an upper level of $S = 4$ has been used by the draft code to represent elastic or near-elastic response. This is less than the level of $S = 6$ specified by the loadings code. This latter value was intended to apply to brittle elements with low elastic damping (2%), and is felt to be unnecessarily conservative for reinforced masonry structures, which can be expected to have comparatively high elastic damping, and at least a small reserve of potential for inelastic action. It should be noted that since the material factor $M = 1.2$ still applies, dependable strength will be based on $SM = 4.8$,

implying that probable strength, at about $1.35 \times 4.8 = 6.5$ is more than 50% higher than the expected elastic response for a 10% damped structure to the design level earthquake.

5. DESIGN FOR FLEXURE:

5.1 Flexural Strength -

The S factors developed in the previous section are applied to the design for dependable flexural strength, which is based on conventional ultimate strength theory, using a strength reduction factor varying between 0.85 and 0.65 depending on axial load level. Numerous tests⁽²⁻⁵⁾ have shown that reinforced masonry designed so that shear failures are inhibited will develop flexural failure modes which can be conservatively predicted using ultimate strength theory equations. For masonry shear walls the typically low axial load levels and reinforcement ratios result in high reinforcement tensile strains at ultimate with consequent strain-hardening and enhancement of ultimate capacity.

For walls with light reinforcement and low axial load level, there is little advantage in terms of ultimate flexural capacity in either concentrating a given quantity of flexural reinforcement in two bundles, one at each end of the wall, or distributing the same amount of reinforcement evenly along the wall. The latter, however, is to be preferred for a number of reasons. Reinforcement concentrated at wall ends causes bond and anchorage problems because of the limited grout space, and increases the tendency for splitting failure of the masonry compression zone as a result of compression bar buckling. If this occurs, strength and stiffness degrade rapidly. Distributed reinforcement is not subject to these faults to the same extent, and has the added advantage of enhancing shear performance. The distributed reinforcement increases the magnitude of the masonry flexural compression force, thus improving compression shear transfer, and also provides a clamping force along the wall base which helps to limit slip of the wall along the base, which has been shown^(3,5,13) to be the most significant cause of degradation of hysteresis loops in well designed walls.

Cardenas et al⁽¹⁸⁾ have shown that the dependable ultimate flexural capacity of a rectangular wall length l_w containing uniform distributed vertical reinforcement of total area A_{st} , and yield stress f_y , subjected to axial force N_u , can be approximated by

$$M_u = \phi_f 0.5 A_{st} f_y l_w \left(1 + \frac{N_u}{A_{st} f_y} \right) \left(1 - \frac{c}{l_w} \right) \quad (8)$$

where c is the distance from the neutral axis at ultimate to the extreme compression fibre, and ϕ_f is the strength reduction factor for flexure.

5.2 Flexural Compression -

The values for c in Eq. 8 is dependent on the masonry crushing strength f'_m . Although the ultimate flexural capacity in Eq. 8 is not highly dependent on c , and hence f'_m , there is an incentive for the designer to aim for high quality work, and specify prism tests, particularly if N_u is large. This condition is only likely to occur in major structures, where the importance of high quality construction becomes even more pronounced. A further incentive to use of prism tests may be obtained in terms of allowable slenderness ratios, which are related to the length of the compression zone, in similar fashion to the provisions of the draft concrete code. For structures of more than 3 storeys height, the low default design value of $f'_m = 8$ MPa may result in a code requirement⁽⁷⁾ for $b_w/l_n \geq 0.075$. With a storey height $l_n \approx 2.8$ m, this could cause uneconomically wide walls. Under such circumstances it will be advantageous to the designer to use a higher f'_m value, verified by in-situ prism tests, to reduce the length of the compression block below the critical $0.3l_w$ or $4b_w$.

5.3 Ductility -

The design approach described in earlier sections implies that not only is the flexural strength dependable, but also that under inelastic cycling, displacement ductility factors of up to $\mu = 4$ can be depended upon without excessive strength or stiffness degradation occurring. Fig. 7 shows hysteresis loops for a reinforced hollow unit concrete masonry wall⁽³⁾ of dimension $h_w = 1.83$ m, $l_w = 2.43$ m and wall width $b_w = 140$ mm, reinforced vertically with 6 - 16mm deformed bars, and horizontally with 8 - 16mm deformed bars, all of yield strength 432 MPa. In Fig. 8, P_u and P_y represent the theoretical ultimate and first yield loads respectively, based on measured material properties, without incorporating a strength reduction factor, and μ is the displacement ductility factor. Despite the aspect ratio h_w/l_w of less than unity, a flexural failure mode was obtained, with peak loads at displacement ductility factors of $\mu = 2, 4$ and 6 exceeding P_u . Maximum shear stress on the gross cross-sectional area was 1.17 MPa. The stiffness degradation apparent in Fig. 8 is a result of the wall sliding along the base at low shear after inelastic cycling had opened up a continuous crack⁽³⁾ along the entire wall length at the base and is characteristic of squat shear walls⁽¹³⁾. It will be noted that the stiffness degradation was not accompanied by strength degradation. The inset photograph shows wall condition at $\mu = 6$. Note the diagonal shear cracks which remained small and controlled by the horizontal shear reinforcement.

The good behaviour displayed by the

wall is typical of squat walls designed to inhibit shear failure by the capacity design procedure. However few tests have been carried out on walls of high aspect ratio. Although it is expected that flexural failure modes will be easier to obtain from such walls, and that base-sliding shear degradation will be less significant than for shear walls, the available ductility may be limited by premature crushing failure of the masonry compression zone at the wall base.

Fig. 9 shows stress-strain curves for unconfined, and confined masonry prisms tested in a recent project at Canterbury University⁽¹⁹⁾. The unconfined curve is an average of six prisms, and indicates a low strain of about 0.0015 at peak stress, and a rather steep falling branch. Failure was initiated by vertical splitting of the masonry units, with cracks initiating at the mortar courses. Where high shear is being transmitted across the compression zone, the vertical splitting creates instability, and it appears that an ultimate compression strain no higher than $\epsilon_u = 0.0025$ should be adopted for masonry. Nevertheless, the shape of the curve is similar to that for unconfined concrete.

Consideration of deflection profiles at yield and ultimate for the linked cantilever wall system preferred for construction (Fig.10) will show that the available displacement ductility at an ultimate strain of $\epsilon_u = 0.0025$ is a function of the aspect ratio h_w/l_w . For the case of rigid foundations (Fig. 10b), and assuming linear elastic behaviour based on cracked-section analysis up to yield, the yield displacement Δ_y for a wall of height h_w depends solely on structural deformation (i.e. $\Delta_y = \Delta_s$) and can be expressed as

$$\Delta_y = \Delta_s = K\phi_y h_w^2 \quad (9)$$

where K is a constant depending on the lateral load distribution, and ϕ_y is the yield curvature. The plastic displacement Δ_p related to an ultimate curvature ϕ_u based on the limit strain of $\epsilon_u = 0.0025$ is

$$\Delta_p = (\phi_u - \phi_y) L_p (h_w - L_p/2) \quad (10)$$

where L_p is the length of the plastic hinge. Although experimental data defining L_p for shear walls are sparse, testing of rectangular columns^(20,21) indicates that it is reasonable to take $L_p = \frac{1}{2} l_w$ regardless of aspect ratio or axial load level. Thus Eq. 10 can be rewritten

$$\Delta_p = (\phi_u - \phi_y) \frac{l_w}{2} (h_w - l_w/4) \quad (11)$$

The available structure ductility is by definition

$$\mu = (\Delta_y + \Delta_p)/\Delta_y \quad (12)$$

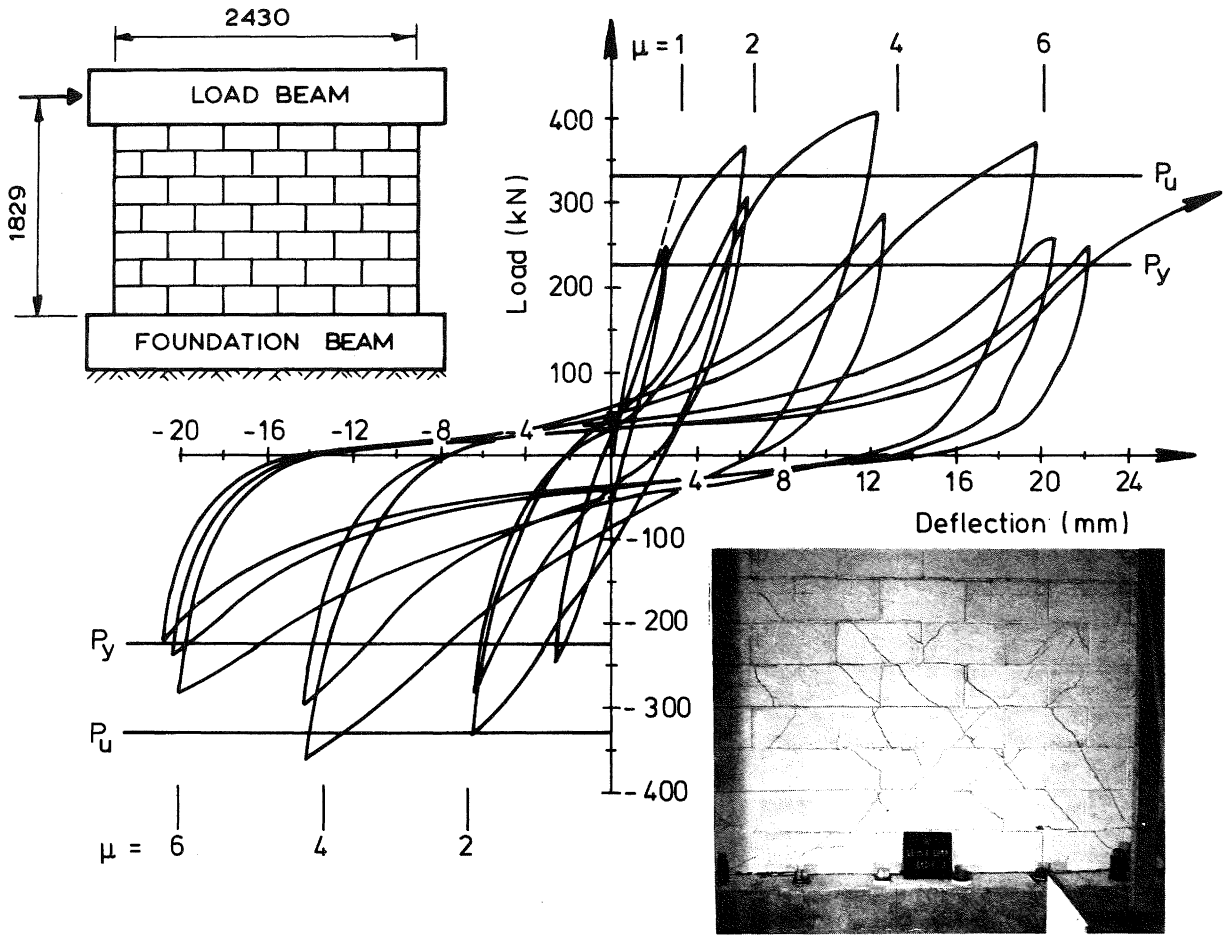


Fig. 8 – Load-Deflection behaviour of a squat RHM Wall

($V_u = 1.17\text{MPa}$)

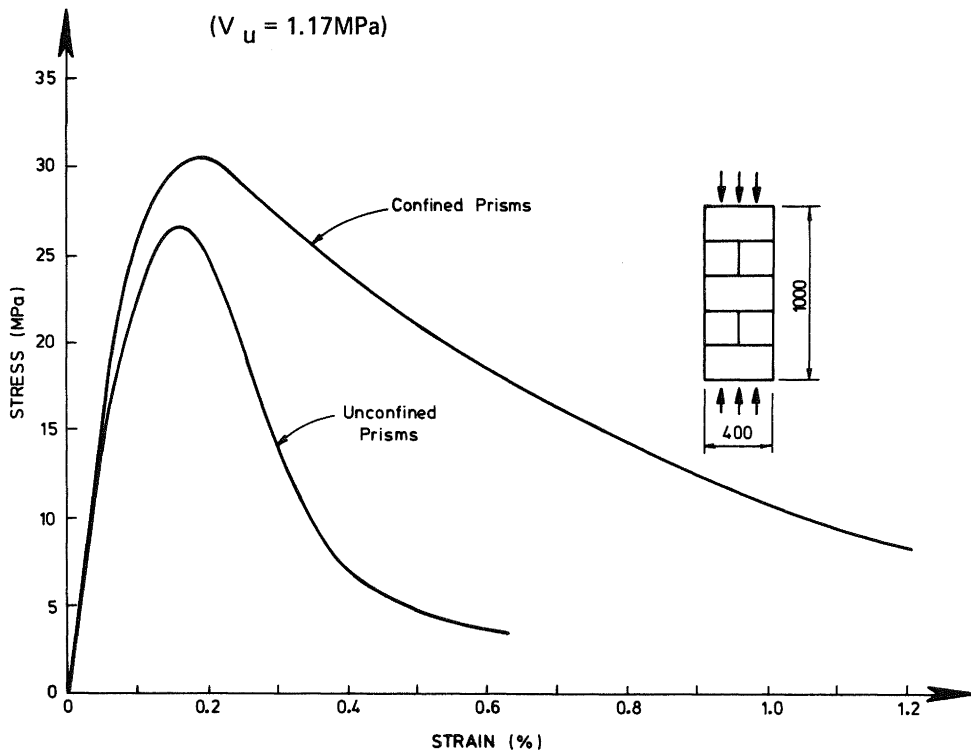


Fig. 9 – Stress-strain curves for concrete masonry prisms.

Substituting from Eqs. 9 and 11 into Eq. 12 and simplifying yields

$$\mu = 1 + \frac{1}{2} \left(\frac{\phi_u}{\phi_y} - 1 \right) \frac{(1 - 1/(4h_w/l_w))}{K(h_w/l_w)} \quad (13)$$

For a wall of given length l_w and axial load N_u , the curvature ductility factor ϕ_u/ϕ_y will be constant. It is thus apparent from Eq. 13 that the available displacement ductility decreases as the aspect ratio h_w/l_w increases.

When cantilever walls are constructed on a flexible foundation (see Fig. 10c), foundation compliance will increase the yield displacement by an amount Δ_f proportional to the foundation rotation. Thus the yield displacement can be expressed as

$$\Delta_y = \Delta_s + \Delta_f = C \cdot \Delta_s = C \cdot K \phi_y \cdot h_w^2 \quad (14)$$

where C is a constant expressing the increase in elastic flexibility. However, assuming elasto-plastic behaviour of the flexural hinge, all plastic displacement will occur at constant load by rotation at the plastic hinge. Eq. 13 is then modified to become

$$\mu = 1 + \frac{1}{2} \left(\frac{\phi_u}{\phi_y} - 1 \right) \frac{(1 - 1/(4h_w/l_w))}{CKh_w/l_w} \quad (15)$$

Thus for a specified ultimate compression strain ϵ_u , the available displacement ductility will be decreased by foundation flexibility. This effect is particularly important for very stiff structures where the increase in elastic flexibility does not result in a natural period shift to a region of the response spectrum of less intensive response.

In the event that Eqs. 13 or 15 indicate that available ductility is less than that implied by the structural type factor (i.e. $\mu < 4/S$), it is necessary to design for a higher S factor, or to enhance the ultimate compression strain by the use of confining steel. For reinforced concrete shear walls confining reinforcement takes the form of lateral ties at close centres vertically and horizontally, but this is generally impractical for masonry. A method that has been developed specifically for masonry construction incorporates 3mm thick stainless steel plates placed in the mortar beds of potential crushing zones. These plates are shaped to the outline of the wall, with an allowance for pointing, for the required length (at least half the distance from the extreme compression fibre to the neutral axis), and have holes corresponding to the size of the grout cavities. Secondary benefits of the plates are increased resistance against compression bar buckling or splitting failure of grout at the starter-bar laps. Fig. 9 includes an averaged stress-strain curves from (19). It will be noted that the plates have

resulted in a strength enhancement, and a substantial flattening of the falling-branch of the curve. The failure mechanism was changed from a vertical splitting failure involving at least the centre three courses, to a shear/compression failure within one course. On the basis of these tests it is felt that an ultimate strain for confined masonry of $\epsilon_u = 0.008$

may be conservatively assumed. Recent testing of tall masonry shear walls at the University of Canterbury⁽¹⁹⁾ indicated a substantial improvement to behaviour when confining plates were included in mortar beds over the bottom $1/6$ of the wall height at each end of the wall. Fig. 11 shows the condition at end of testing for equivalent unconfined and confined walls ($\mu = 5.5$ and 8.0 respectively). Note that considerable breakdown of the unconfined wall end has occurred, with grout crushing and splitting, but the confining plates have maintained the integrity of all but the bottom course of blocks, despite the higher ductility, and a significantly increased number of inelastic cycles of displacement.

It is not intended that the designer should be required to perform detailed calculations to assess the ductility capacity of each wall. Research under way will result in guidelines as to what combinations of aspect ratio, material strengths, reinforcement ratio and axial load levels should result in a requirement for confining. Present indications are that only a small fraction of designs will be affected. However, T-section walls with the flange in tension may be expected to be particularly susceptible.

6. DESIGN FOR SHEAR:

6.1 Shear Force -

A basic tenet in the seismic design philosophy for masonry structures is that shear failure should be avoided at all costs. Masonry walls failing in shear have very limited ductility and suffer rapid strength and stiffness degradation following formation of a diagonal shear crack. Consequently the draft code requires that the dependable shear strength exceeds the maximum feasible shear force. This is done by either requiring a capacity design approach, or by the specification of structural type factors for shear that exceed those for flexure by an adequate margin.

Cantilever walls designed for $S = 1.0$ to $S = 1.6$ are designed to capacity design principles⁽¹⁶⁾. Although the dependable flexural strength, based on a strength reduction factor ϕ_f and specified material strengths is matched to the appropriate S factor, it is expected that, with good construction procedures, the actual flexural strength will exceed the ideal strength ($\phi_f = 1.0$), as a result of strain-hardening of the tension reinforcement, and characteristic material strengths exceeding specified minima. This is clearly illustrated in

Fig. 7 where the maximum load exceeded the theoretical ultimate load, based on measured material properties, by 24%. To ensure shear failure does not occur, the dependable shear strength must exceed this enhanced flexural capacity.

The design shear force V_D is related to V_C , the code shear based on the specified S factor, by the relationship

$$V_D = \frac{\phi_o}{\phi_f} \cdot V_C \quad (16)$$

where ϕ_o is the ratio of maximum feasible flexural strength to ideal flexural strength. Commonly accepted⁽¹⁶⁾ values of ϕ_o vary between 1.25 and 1.40, dependent on flexural reinforcement yield strength. It should be noted that further overstrength in flexure can occur due to provision of more flexural reinforcement than required to satisfy the code loading, and that shears in multistorey shear walls can be increased due to higher mode response⁽²²⁾. These effects should be considered when designing for shear.

Structures where $S > 1.6$ for flexure, are designed so that the dependable shear strength exceeds the required flexural strength by 50%. This results in much the same level of protection as Eq. 16, but is simpler to apply as the flexural strength need not be calculated to the same precision. Although implementation of capacity design equations such as Eq. 16 might result in a requirement for a margin of more than 50%, this is not felt to be necessary, as the minimum effective S factor for shear would be 2.4, implying a structure ductility of about $\mu = 1.67$. At such a low ductility, strength enhancement resulting from strain-hardening of flexural reinforcement is likely to be low, justifying a lower level of shear protection.

6.2 Shear Carried by Masonry Shear Resisting Mechanisms -

Within potential plastic hinge regions, all shear force is required to be carried by truss action of horizontal reinforcement. As with reinforced concrete, masonry shear resisting mechanisms within the plastic hinge zone degrade rapidly as a result of formation of wide, crossed flexure-shear cracks under inelastic cyclic loading. The draft code is more conservative than the draft concrete code, as masonry shear capacity is ignored regardless of the level of axial load. The code committee felt there was insufficient experimental data for v_m to support a value increasing with axial load level.

In other areas, where plastic hinges will not occur, or where elastic response is assured by adopting a structural type factor $S = 4$, the draft code allows

shear stresses, based on the gross cross-section of 0.12, 0.24 and 0.30 MPa to be carried by masonry shear resisting mechanisms for Grades C, B and A masonry respectively. These figures have been obtained from a conservative assessment of results from monotonic tests of shear panels^(23,24).

6.3 Shear Carried by Reinforcement -

Two possible situations are identified in Fig. 12. When the aspect ratio exceeds unity (Fig. 12a), a potential shear failure crack, inclined at 45° crosses the entire width of the wall. Normal reinforced concrete theory gives the required steel area A_V , at vertical spacing s , as

$$A_V = \frac{V_D \cdot s}{\phi_s \cdot f_y \cdot d} \quad (17)$$

Where V_D is given by Eq. 16, ϕ_s is the strength reduction factor for shear ($\phi_s = 0.80$ in the draft code) and d is the effective depth, normally taken as $0.8l_w$ for cantilever shear walls.

When the aspect ratio is less than unity (Fig. 12b), the critical 45° crack intersects the wall top. Recent research on squat concrete shear walls⁽¹³⁾ indicates that the shear entering the wall on the tension side of the 45° line from the compression toe can be transmitted by arch action involving the vertical flexural reinforcement and inclined masonry compression struts. Shear reinforcement needs to be provided to transfer the shear entering the wall on the compression side of the potential initial crack, back across the crack into the body of the wall. Thus in Fig. 11b, assuming that the design shear V_D is distributed evenly across the wall length, the required steel area A_V at spacing s is

$$A_V = \frac{1}{\phi_s} \cdot V_D \cdot \frac{h_w}{l_w} \cdot \frac{s}{f_y h_w} = \frac{V_D \cdot s}{\phi_s f_y l_w} \quad (18)$$

It will be noted that Eqs. 17 and 18 are inconsistent for an aspect ratio of 1.0, when both theoretically apply. This is because of the different mechanisms involved, and clearly some judgement is required in determining which mechanism applies. For aspect ratios significantly less than unity, Eq. 18 is likely to be conservative, as some shear will be transmitted back across the 45° crack at the top of the wall by a beam or roof slab.

6.4 Maximum Allowed Shear Stress -

It has been established⁽³⁾ that masonry shear walls detailed to capacity design principles can develop shear stresses as high as 2.4 MPa without shear failure, while sustaining controlled flexural failure. However, when the displacement ductility factor μ is increased at high shear stress levels, severe stiffness and load degradation results as the wall starts

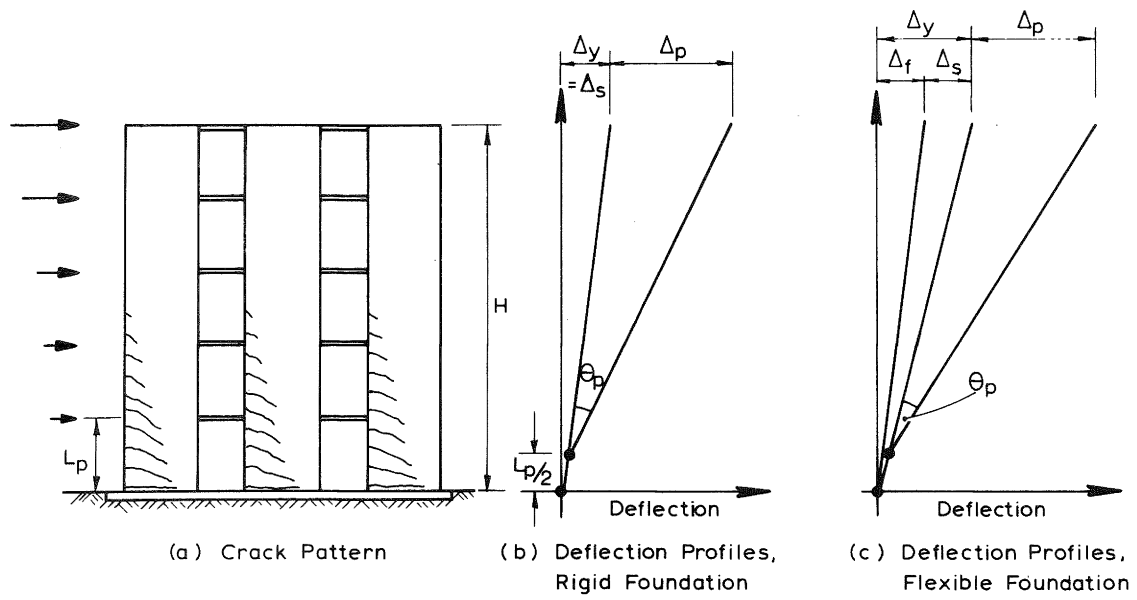
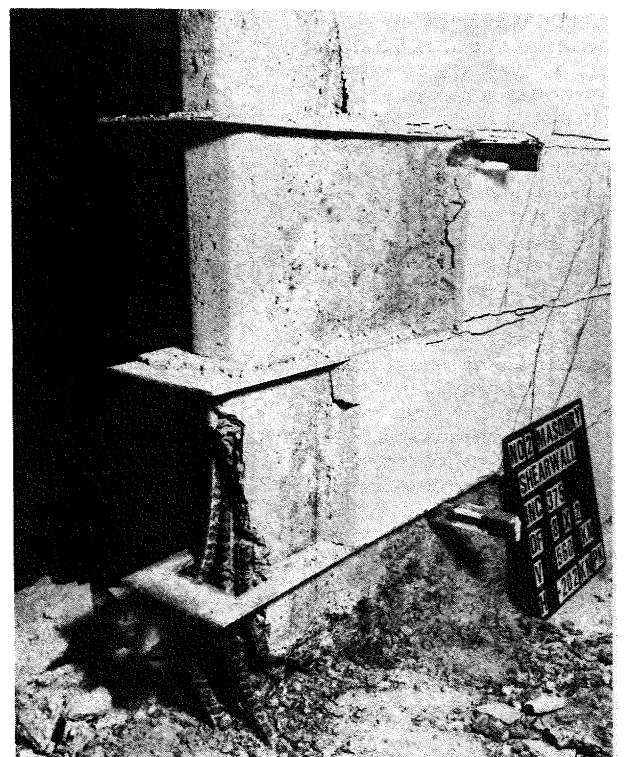


Fig. 10 – Ductility of slender masonry shear walls.



(a) Unconfined Toe, 2 cycles to $\mu = 5.5$
(Note compression and bond failures)



(b) Confined Toe, 2 cycles to $\mu = 8.0$
(end bar buckling, no bond failure)

Fig. 11 – Influence of confining plates on behaviour of RHM shear walls.

to slide on its base. Tests of walls deliberately built to substandard construction practice⁽⁵⁾ showed that degradation occurred at lower values of μ , and was more rapid for these walls, as the poor condition of inadequately vibrated grout at the wall base decreased the ability of the walls to transmit shear by shear friction and dowel action of the vertical reinforcement. This potential influence is recognised in the draft design code which specifies maximum design loads for ultimate shear of 0.5, 1.0 and 1.2 MPa for grades C, B and A respectively. The significance of ductility level to sliding shear resistance is further recognised by allowing increases for Grade A masonry above 1.2 MPa where excess flexural capacity is available, according to the relationship

$$v_u = 1.2 + 0.4 (R - 1.0) \leq 1.6 \text{ MPa} \quad (19)$$

where R is the ratio of actual dependable flexural strength to code required dependable flexural strength. R factors higher than unity, which may result from code minimum reinforcement ratios exceeding seismic requirements, from a 'rational', rather than an 'ideal' distribution of flexural reinforcement governed by block module size, or as a result of design choice, will reduce the design level of ductility, and hence provide better base-sliding resistance.

7. DETAILING:

It could be argued that sound detailing is even more important to satisfactory seismic performance than the provision of adequate shear and flexural strength. The draft code has set fairly rigorous detailing standards which, it is hoped, will provide the integrity and ductility necessary for masonry walls to avoid the brittle failures of reinforced masonry structures evident in the 1971 San Fernando earthquake⁽²⁵⁾. Detailing requirements include stringent slenderness ratio requirements for plastic hinge regions to prevent buckling of the wall under flexural compression, close-spaced vertical and horizontal reinforcement for virtually all types of masonry construction, including infill panels, and conservative reinforcement lapping requirements. An exception to the requirement for two-way reinforcement is made for minor structures in Seismic Zones B and C which are designed to $S = 4$, and where the design shear stress does not exceed the appropriate value for masonry shear mechanisms in Table 1, in which case horizontal reinforcement may be omitted.

Although lapping of flexural reinforcement in potential plastic hinge zones is undesirable, it is permitted because of practical considerations. However, the draft code emphasises that wherever possible the designer should avoid the necessity for lapping by use of open-ended bond-beam units, and vertical reinforcement at 400 mm centres, allowing blocks to be moved laterally

into position. Space limitations do not permit a full description of the draft code detailing requirements.

8. NON-STRUCTURAL WALLS:

The draft code⁽⁷⁾ identifies three categories of so-called non-structural walls. In each case specified detailing requirements are intended to ensure collapse does not occur during seismic attack. It must be recognised that such walls, although perhaps not contributing to the primary seismic resistance of the structure, will still be subjected to inertia loads and deformation, and must be designed accordingly.

8.1 Partitions -

The minimum thickness is 90 mm or $0.035h$, where h is the unsupported height, increased to 120 mm where the partition is a wall of an exit way, or is adjacent to a public place, where consequences of failure are more serious. Sufficient separation between partitions and other parts of the structure must be provided to prevent contact during the design level earthquake. This is to avoid loading of the partition and structural modification of the primary seismic system. For frame structures, this implies that separations between frame and partition should be at least twice that specified by the loadings code.

Face loads resulting from inertial response of the partition mass must be calculated including the possibility of resonant response, when partition and structural natural frequencies are similar, particularly for upper floors. Minimum reinforcement contents, and spacing limitations are similar to non-hinging regions of structural walls.

8.2 Veneers -

Recent dynamic testing has established that unreinforced veneers tied to structural backing can sustain high face-load inertial response⁽⁶⁾. Brick veneer panels 2.4 m high tied to timber backing were subjected to repeated sinusoidal response accelerations of more than $2.0g$ without failure. This level of response is approximately twice that implied under design earthquake loading by the smoothed response spectra shown in Fig. 3. However structural response accelerations may be much higher than code levels for veneers on upper storeys of multistorey buildings as a result of resonant response. Because of the severe consequences of veneer shedding into streets from upper floors, a height limitation is placed on unreinforced veneer, that depends on seismic zone. Where the veneer is at an egressway, a blanket height restriction of 2.4 m is made.

Reinforced veneers may be used without height restriction, provided adequate basketting reinforcement (vertical reinforcement plus horizontal joint reinforcement) or close spaced vertical

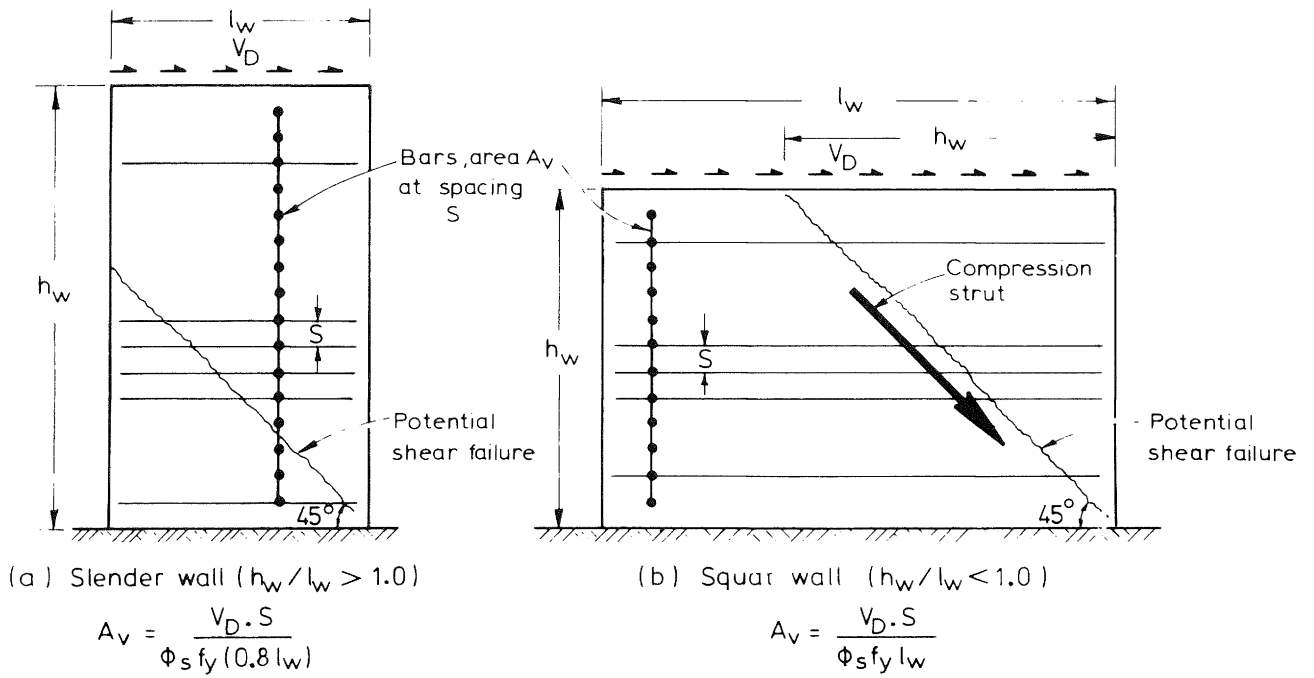


Fig. 12 – Shear reinforcement for cantilever shear walls.

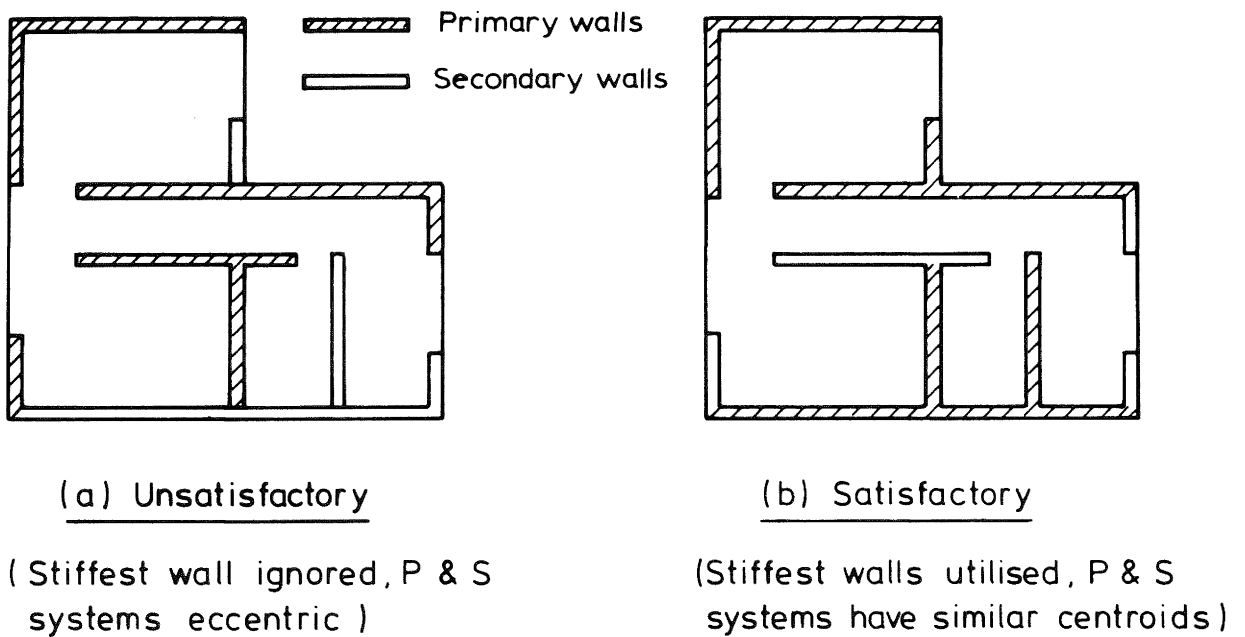


Fig. 13 – Example of subdivision of walls into primary and secondary systems.

reinforcement is provided, and vertical support is provided at foundations and not more than 10 m vertical spacing.

8.3 Secondary Walls -

Some shear wall structures do not lend themselves to rational analysis under lateral loading, as a consequence of the number, orientation and complexity of shape of the load bearing walls. In such cases it is permissible to consider the walls to consist of a primary system, which carries gravity loads and the entire seismic lateral load, and a secondary system which is designed to support gravity loads and face loads only. This allows simplification of the lateral load analysis in cases where the extent of wall area exceeds that necessary to carry the code seismic loads. However, although it is assumed in the analysis that the secondary walls do not carry any in-plane loads, it is clear that they will carry an albeit indeterminate proportion of the lateral load. Consequently they must be detailed to sustain the deformation to which they will be subjected, by specifying similar standards as for structural walls, though code-minimum requirements will normally be adopted. To ensure satisfactory behaviour results, the natural period should be based on an assessed stiffness of the composite primary/secondary system.

No secondary wall is permitted to have a stiffness greater than one-quarter that of the stiffest wall of the primary system. This is to ensure that the probability of significant inelastic deformation developing in secondary walls is minimised, and integrity of secondary walls for the role of gravity load support is maintained. Long stiff secondary walls may be divided into a series of more flexible walls by the incorporation of vertical control joints at regular centres. A further requirement in selecting the primary and secondary systems of walls is that the centres of rigidity of the two systems should be as close as possible to minimise unexpected torsional effects. Fig. 13 shows acceptable and unacceptable division of a complex system of walls into primary and secondary systems.

9. CONCLUSIONS:

Seismic design aspects of the draft masonry code⁽⁷⁾ have been based on the following premises.

- (1) Unreinforced masonry is unsuitable for seismically active areas and can only be used in veneer construction, under restricted circumstances.
- (2) Reinforced masonry can be made to perform adequately under design level shaking, provided lateral design loads are related to the structural type adopted, and stringent detailing requirements are met. Structural types suitable for aseismic reinforced concrete or steel construction

may be less suitable for reinforced masonry, and hence different structural type factors should be assumed.

- (3) The importance of good workmanship and inspection during construction is paramount, and is reflected in allowable stress levels for compression and shear which depend on construction details and extent of inspection rather than on constituent material strengths.
- (4) There is no such thing as 'non-structural' masonry. Elements which are not considered part of the primary structural system will still be subjected to deformation and inertia loading as a result of their own mass, and building deformation, and must be designed accordingly.

The draft masonry code sets more detailed design requirements than its predecessor, but is significantly more liberal in terms of allowable stress levels and general structural freedom.

10. ACKNOWLEDGEMENT:

The masonry design code committee has consisted of the following people:

Chairmen	: B. Chamberlain R.J. Fulford
Members	: B.W. Buchanan O.A. Glogau M. Jones E.B. Lapisch M.J.N. Priestley R.D. Sullivan D. Turner M.A. Wesseldine B.J. Wood

The writer wishes to acknowledge the work of all other members of the committee in developing the draft code, and in particular, the extensive input of the late Otto Glogau, without whose industry, drive, and detailed understanding of masonry structures, the present document could hardly have evolved. The writer believes that the paper represents the consensus viewpoint of the committee on the background to the code, but it should be interpreted as a personal statement rather than an official committee document.

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