New Zealand Concrete Masonry Manual

concretenz
MASONRY

Patrick Reynolds Photography / Mercer and Mercer Architects

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1.1 What is Concrete Masonry?

Introduction

Concrete masonry construction (or as it is more commonly called, concrete blockwork) is based on thousands of years experience in building structures of stone, mud and clay bricks. Blockwork masonry units are hollow and are filled with concrete and allow for the integration of reinforcing steel, a feature essential for earthquake resistant design.

Concrete blockwork provides a structural and architectural advantage in one material and is recognised worldwide as a major contributor to the construction and building industry.

Types of Concrete Blockwork

The workhorse of concrete masonry has traditionally been stretcher bond blockwork forming structural, fire and acoustic functions from residential to large commercial buildings as well as the special use in retaining walls.

With the introduction of coloured masonry Architects and Specifiers are using Architectural Masonry in more commercial and residential applications.

Using the various textures of Fair Face, Honed and Splitface is adding a lot more variety to the features of the wall.

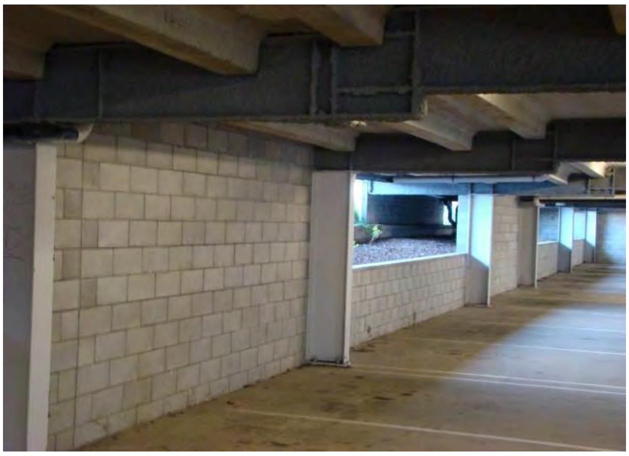


Figure 1: Commercial Stretcher Bond Blockwork

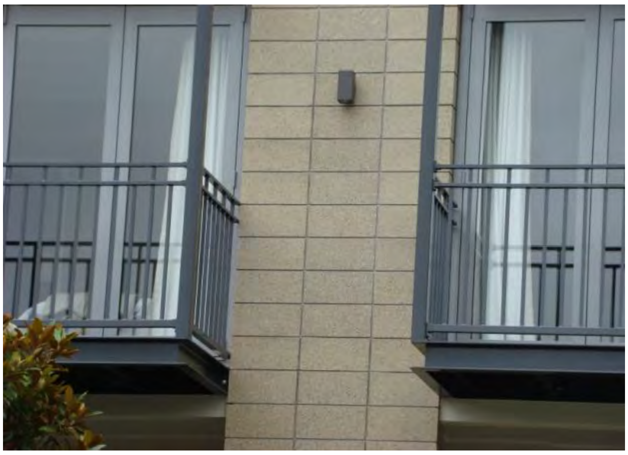


Figure 2: Coloured Honed Blockwork

Figure 3: Coloured Honed Blockwork

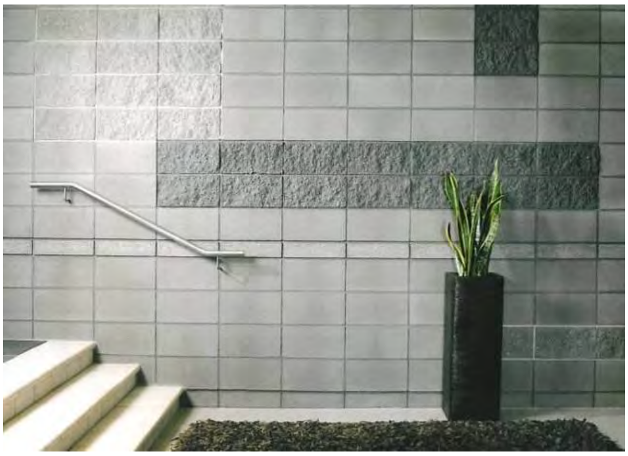


Figure 4: Coloured Fairface, Honed and Splitface Blockwork

Figure 5: Splitface Blockwork

Figure 6: Honed Half High Blockwork

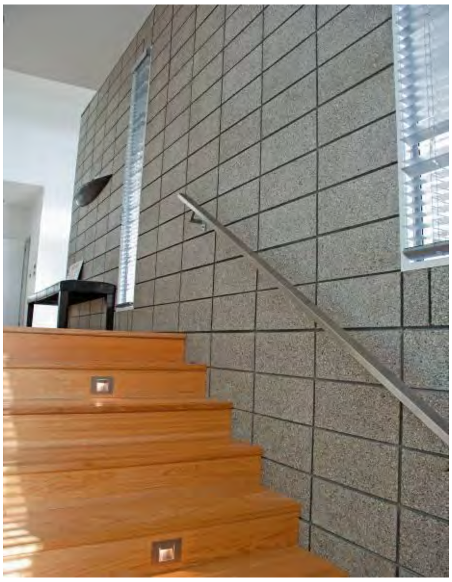


Figure 7: Honed Natural Blockwork

There are also masonry blocks that include polystyrene inserts which provide all the structural benefits of a normal masonry block with the added advantage of built-in insulation.

Building with these blocks removes the need for additional insulation - providing the added design flexibility of a solid plastered finish both inside and out.

The word "Concrete Masonry" also encompasses a wide variety of products such as, brick veneers, retaining walls, paving and kerbs.

Brick veneers are available in a range of sizes and colours and also have a large module range and can be splitface or rumbled.

Figure 8: Masonry Block with Polystyrene Insert

Figure 9: Rumbled Finish

Figure 10: Splitface Finish

Figure 11: Splitface Finish Retaining Wall

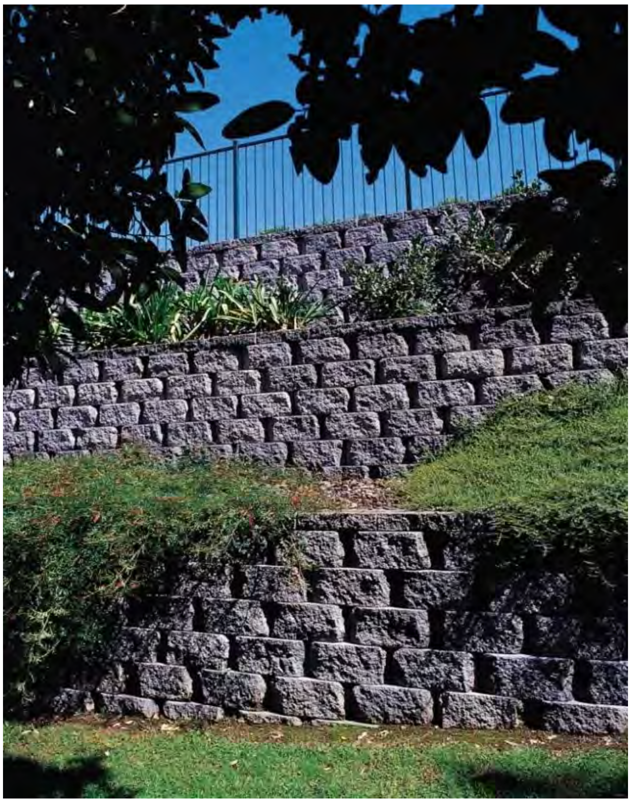


Figure 12: Splitface Finish Retaining Wall

Figure 13: Splitface Finish

Figure 14: Residential Paving

Retaining wall blocks cater from the heavy duty type applications down to the DIY ornamental walls around garden beds. The splitface types give the appearance of natural stone.

There is a wide variety of paving available in different colours and shapes and textures for both residential and commercial applications. Bush hammering and honing are also options as a secondary process which adds variety to the textures.

Flagstone paving brings additional terrazzo style finishes to the range of products the concrete industry can supply.

Figure 15: Commercial Paving

Figure 16: Bush Hammered Finished

Figure 17: Honed Finish

Figure 18: Commercial Flagstone Paving, Queenstown Airport

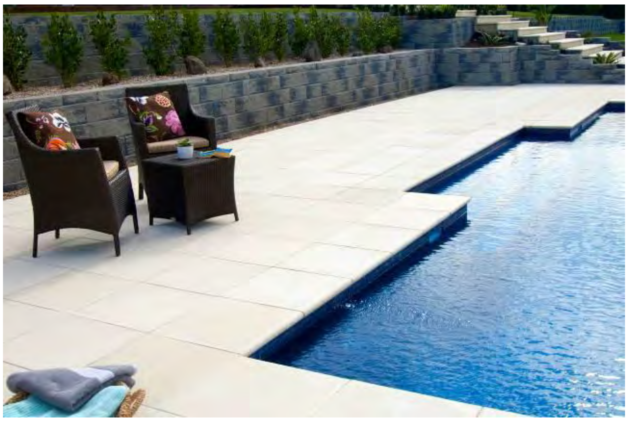


Figure 19: Terrazzo Flagstone Paving, Residential Pool, Auckland

Figure 20: Commercial Flagstone Paving, Richmond Town Centre

Figure 21: Commercial Flagstone Paving, Wharewaka Building, Wellington

Permeable paving and grass pavers are also available for use in residential and commercial applications.

There are some major advantages for permeable paving especially in a world which is far more conscious of the sustainability of our environment.

Advantages of Permeable Paving

- Improved water quality from runoff by filtering out contaminants.
- Improved hydrological response of stormwater peak flow by holding and releasing in a controlled manner.
- Providing amenity/landscape feature, especially when different coloured pavers are used.
- Allows utilizing the paving area for treating its own stormwater and therefore does not require ant additional land areas outside the paving area to treat stormwater runoff.
- Increases the impermeable footprint of a site as the driveway, carpark or patio is treated as a permeable surface.
- Can be manufactured in various formats and still maintain commercial strength and limited maintenance.

Formpave is a proprietary storm water source control system which allows heavy rain to infiltrate through a permeable concrete block paved surface into a unique sub-base before being released in a controlled manner into sewers or water courses.

Formpave Storm water source control systems and Aquaflow permeable paving products are suitable for use on carparks, industrial estates, retail centres, pedestrian areas, domestic drives, motorway services, airport service areas and aprons, garages and other heavy duty operations and are discussed more fully in Section 7 of this Manual.

Figure 22: Grass Pavers

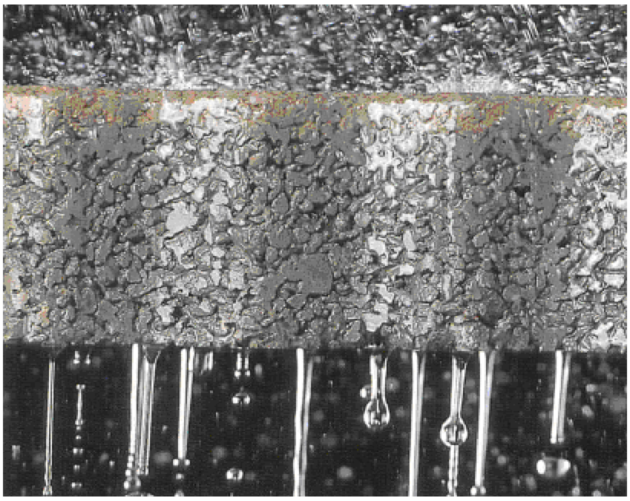


Figure 23: Permeable Paving

Figure 24: Infiltration Test on Permeable Pavers

Figure 25: Aquaflow Permeable Paving

Heavy Duty Commercial Paving

Commercial paving can be manufactured with increased strength to resist heavy axial and point loads in heavy duty applications such as Airport aprons, container yards, wharves and sawmill processing yards.

The product can also be delivered in a herringbone pattern to enable a machine laying process which is far quicker in areas between 5,000 m^2 to 20,000 m^2.

Figure 26: Tokoroa Sawmill 100 mm Pavers

Figure 27: Container and Truck Distribution Warehouses 80 mm Pavers

Figure 28: Paver Laying Machine

Figure 29: Paver Laying Machine

Figure 30: Christchurch Airport Apron Slab 80 mm Pavers

Figure 31: Christchurch Airport Apron Slab 80 mm Pavers

Masonry Block Manufacture

Almost without exception, concrete blocks are now made in fully automatic machines. Raw materials are stocked in separate bins near to the mixer which is so arranged that concrete is fed to the block making machine either by gravity or mechanically.

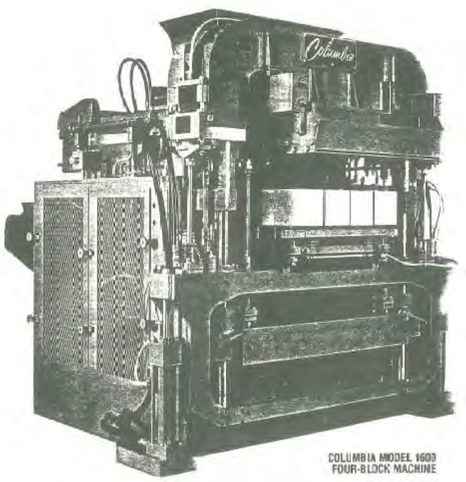


Figure 34: Columbia Model 1600 Four Block Machine

The concrete mix design is such as to produce the desired properties in the finished units as well as being properly workable for filling the moulds, pressing, vibration, stripping and moving of the fresh blocks to the curing chambers.

Concrete used in block making is of a very dry cohesive mix designed for moulding in conjunction with powerful vibration simultaneous with compression while in the mould.

In most systems the blocks are extruded from the moulds in a downwards direction, being pushed by a multi-plunger type head mechanism fitted with stripper shoes. These heads, like their matching moulds, are specially shaped to suit each respective block type.

The mould bases are heavy steel plates, or pallets. A steel pallet is used for each new moulding cycle and acts as a supporting tray for the newly moulded blocks in their removal from the moulds and the passage through the curing process. Blocks are removed from the steel pallet at the dispatch station which returns the steel pallet to the block making plant.

Curing

For structural masonry the 'green' blocks, after demoulding, are mechanically conveyed and loaded in racks for curing, usually in large insulated chambers.

Once the 'green' blocks are in the chamber, steam at atmospheric pressure and above 70°C is introduced into the chamber, thereby accelerating hydration - the chemical reaction between water and cement which causes hardening. This allows blocks to be packaged the day after moulding.

Other masonry products may have alternative curing processes which do not involve the use of steam or accelerated curing resulting in energy saving.

Secondary Processes

In some instances, blocks are passed on to other machines for further processing, e.g. concrete bricks for rumbling, which gives random rounded edges to the units, solid units for splitting into split face blocks or honing which removes the top 2-3 mm of the surface to expose the matrix and the aggregate, which gives the appearance similar to a "terrazzo" product.

Figure 32: Secondary Process in Block Manufacture

Figure 33: Secondary Process in Block Manufacture

Packaging and Handling

Following curing the blocks are mechanically conveyed to the “cuber", which strips the steel pallet, binds the blocks into "cubes" usually 1.2 x 1.0 in plan, on wooden pallets. These "cubed" pallets of blocks are transferred to yard storage and eventually to the construction site. Factory stocks are usually stored out of doors and moved by gantry crane, mobile crane, fork-lift truck or cranes mounted on delivery trucks. Such crane-fitted trucks are usually used for deliveries to the construction site.

Quality Control and Testing

All aggregates and cement are subject to checking and the design and proportioning of mixes are closely supervised. Machine controls, moulds, cores and height mechanisms are regularly checked to ensure dimensional accuracy.

Prior to cubing or further processing, units are subjected to a continuous visual inspection. Blocks of sub-standard appearance are rejected and removed. At this time blocks are taken at random from the conveyor run as specimens for testing in compliance with AS/NZS 4455. A suite of testing procedures for masonry is contained in AS/NZS 4456. The resulting consistency of product provides a dependable building material.

Advantages of Concrete Masonry

Concrete masonry stands almost alone as a material that can meet structural and architectural requirements within itself. It is predictable, and therefore reliable. It gives a sense of security.

The nature of concrete provides permanence, high fire and acoustic resistance and minimum maintenance.

It is available in a wide range of texture, shape and colour; and in units ideally suited for vertical, horizontal or sloping building elements and surfaces.

Concrete masonry lends itself to the development of exciting and efficient architectural and engineering forms and finishes. The versatility of concrete masonry, perhaps more than most other materials, enables it to meet the ever changing demands of a progressive society.

Concrete masonry walls have the ability, if positioned correctly, to capture free energy from the sun “thermal mass” and will help with the thermal efficiencies of a house or building.

Figure 35: Cubed Pallets of Concrete Blocks

Figure 36: Cubed Pallets of Concrete Blocks

1.2 Concrete Masonry Wall Units

Concrete Masonry Units are required to meet the requirements of Australian/New Zealand Standard *AS/NZS 4455 Part 1 Masonry units, pavers, flags and segmental retaining wall units - Masonry units*. The specification requirements of this document are contained in Section 1.7. Specification information for concrete bricks is presented in Section 5.3.

Definitions

The following definitions are in common usage.

- **Masonry** - any construction in units of concrete laid to a bond and joined together with mortar.
- **Face Shells** - walls of a masonry unit connected by a web and normally laid vertically.
- **Gross Cross-sectional Area** - total plan area parallel to the bedding surface including cells and re-entrant spaces.
- **Hollow Masonry Unit** – a unit with cores, intended to be laid with its faces vertical.
- **Lightweight Masonry Unit** – a unit with a dry density of less than 1,850 kg/m^3.
- **Net Cross-sectional Area** - the gross plan area less the area of cells and re-entrant spaces.
- **Nominal Dimensions** – unit dimensions defining block size inclusive of the 10 mm mortar joint.
- **Normal Weight Masonry Unit** - a unit with a dry density greater than 1,850 kg/m^3.
- **Solid Masonry Unit** – a unit with any recesses being not greater than 10% of gross volume.
- **Web** – a cross partition connecting face shells within a hollow masonry unit.

Block Types

- **Plain Face Blocks** - have a relatively smooth face texture and are typical of most standard masonry.
- **Split Face** – Veneer or structural units double moulded and mechanically split to form a rough textured face. Some structural units are available with split ends to form corners.
- **Rumbled Units** – veneers and some paving units with induced spalling at corners due to block to block contact in a rotating drum.
- **Screen Blocks** – units with large voids used as a wind break instead of a solid wall. They have decorative features arising from the pattern of voids.
- **Interlocking Paving** – units when laid provide a structural wearing surface for footpaths, residential driveways, patios, suburban roads and industrial applications. (See Section 7).
- **Flagstone Paving** – units with gross plan area greater than 0.08 m^2 laid to provide a wearing surface for footpaths and residential driveways. (See Section 7).
- **Turfed Paving** – units manufactured with voids to be filled with topsoil and grassed as an option for parking on a lawn. Also used as an option for permeable paving. (See Section 7).
- **Custom Masonry** – many masonry options are available offering alternative face shell effects with variations to texture and colour.
- **Pilaster and Column Blocks** – hollow units within a masonry wall which are filled with grout to form a vertical reinforced column.
- **Bondbeam** – a unit designed to include horizontal reinforcing and be grouted to form a beam.

Block Numbering System

Several different numbering or coding systems are used for concrete masonry units in many countries. The New Zealand Concrete Masonry Association has established the following national coding system for use by designers, specifiers, merchants, blocklayers and member companies of NZCMA.

The system is basically numerical with letters indicating variations of particular numerically coded units. Each code reference is in two numerical sections - the first refers to nominal unit width and the second to type of unit.

All units are nominal 200 mm height (190 mm actual) unless prefixed with T, H, or Q which respectively denote three-quarter height (150 nominal, 140 actual) or half height (100 nominal, 90 actual) units. When a block is suffixed H it indicates the special plan shape of the block which represents an H form with two open ends.

The following code covers standard block types currently in production in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.

Coding Examples:

From the above code, unit reference 20.05 denotes 200 mm nominal width, open end unit.

Reference 15.15 Left denotes 150 mm nominal width left handed corner bond beam.

Reference H20.04 denotes 200 mm nominal width half-height plain end whole unit.

Prefix	Column 1	Column 2	Reference	Suffix
T			Three-quarter height	
H			Half-height	
F			Thick wall-fire rated block	
	10.		100 mm nominal width	
	15.		150 mm nominal width	
	20.		200 mm nominal width	
	25.		250 mm nominal width	
	30.		300 mm nominal width	
		01	Standard whole	
		02	Half	
		03	Corner	
		04	Plain ends	
		05	Open end	
		08	Sill	
		09	Rebate whole	
		10	Rebate half	
		11	Rebated lintel	
		12	Lintel & half end-closer	
		13	Deep lintel &full end-closer	
		14	Knock in bond beam	
		15	Corner bond beam	
		16	One open end bond depressed web/clean out	H two open end
		17	Solid whole	
		18	Quarter	
		19	Three-quarter	
		23	Channel bond beam	
		24	Channel open bond beam	
		26	Half single bull nose	
		27	Whole single bull nose	
		28	Half double bull nose	
		29	Whole double bull nose	
		30	Standard 200 mm pier	
		32	Control joint	
		33	L pier square end	
		34	Pilaster C type	
		35	Pilaster H type	
		44	Half knock-in bond beam	
		45	Header	
		64	Capping block	

Units available in alternative forms, (e.g. left or right) are to be defined by a suffix note.

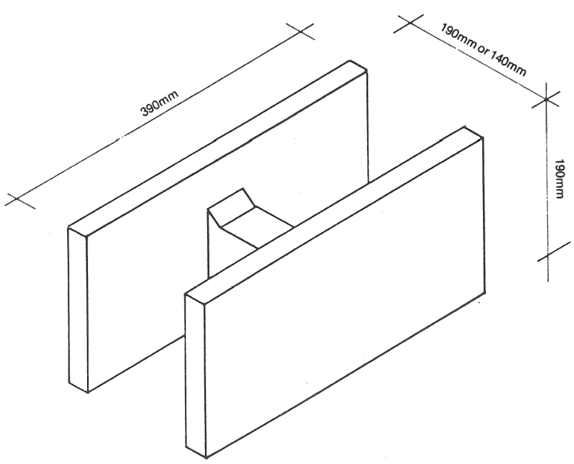
Concrete Block Series

The full graphic descriptions of the block types listed in the block number system are set out in this section. Many of the block types in the full listing are not normally in standard production, since utilisation may be very infrequent.

The blocks listed below are those which are predominantly in standard production. A restricted number of these types are available as half high units.

BEFORE SPECIFYING BLOCK TYPES, a check with potential supplies to the area of construction should be made. All manufacturers have a current production range publication.

Some manufacturers have introduced a block which is an H configuration for the construction which requires solid filling. It would be used as an alternative to 20.16. The advantages of these blocks relates to providing greater vertical cell areas and avoiding back to back problems when using the 20.16 units with reinforcement spaced at 600 mm centres. Solid fill structural masonry in the United States makes significant use of this block shape which is illustrated below.



2016 H or 1516 H
Depressed web solid fill H shape block

20 Series		
20.01	20.16	20.64*
20.02*	20.16H*#	
20.03*	20.17*	
20.04	20.20*	
20.05	20.23*	
20.08	20.24*	
20.09*	20.30	
20.10*	20.32*	
20.11*	20.33	
20.12	20.34	
20.13*	20.35	
20.14	20.44*	
20.15	20.45*	

15 Series	10 Series	25 Series
15.01	10.01	25.01*
15.02*	10.02	25.02*
15.03	10.03	25.04*
15.04	10.05	25.05*
15.05	10.08*	25.11*
15.12	10.14	25.12
15.14	10.17	25.14
15.15	10.30*	25.15*
15.16*		25.16*
15.16H*#		25.16H*#
15.19*		
15.30*		
15.35*		

Note: * Not all NZCMA members produce this block or all the variations of rebate, end shape, etc.

The 15.16H, 20.16H and 25.16H blocks are manufactured by Mitchell Concrete Limited. Bowers Brothers Concrete Limited and Firth Industries will manufacture 16H blocks to order.

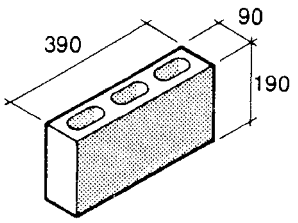
NZCMA Member Companies

To confirm availability of block types, NZCMA Members can be contacted at the following links:

Bowers Brothers Concrete Limited	Various Locations in Upper North Island	www.bowersbrothers.co.nz
Firth Industries	Various Locations	www.firth.co.nz
Mitchell Concrete Limited	Taranaki	www.mitchellconcrete.co.nz
The Block Shop New Zealand Ltd	Various Locations in the North Island	www.blockshop.co.nz
Viblock Limited	Christchurch and Alexandra	www.viblock.co.nz

10 Series

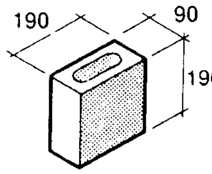
These diagrams cover standard block types in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



10.01

Standard whole

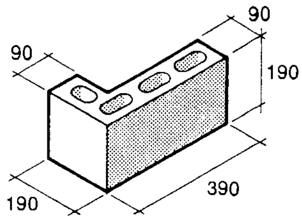
(also available with two cores)



10.02

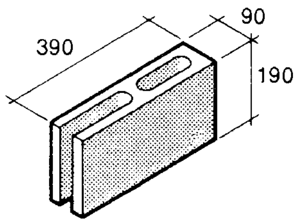
Half

(also known as 20.18)



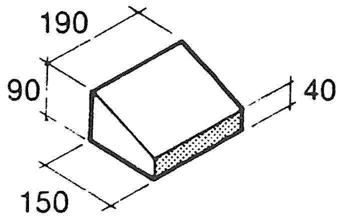
10.03

Corner



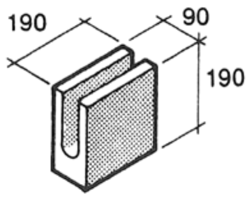
10.05

Open end



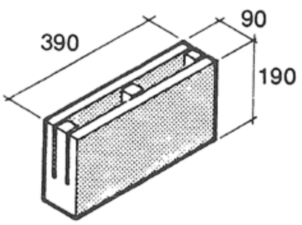
10.08*

Small



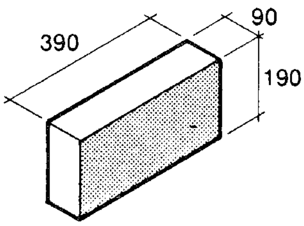
10.12

Lintel & half end-closer



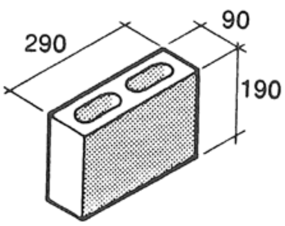
10.14

Knock-in bond beam



10.17

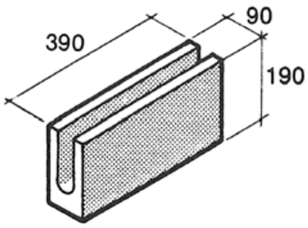
Solid whole



10.19*

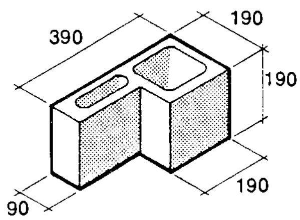
Three Quarter

(also known as 30.18)



10.23

Channel bond beam



10.30*

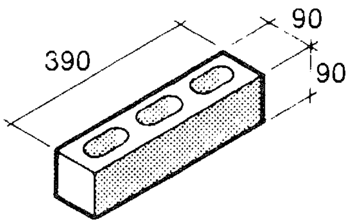
Standard 200 mm pier

Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

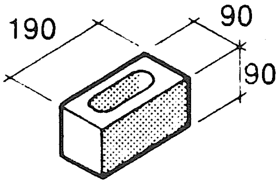
H10 Series

These diagrams cover standard block types in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



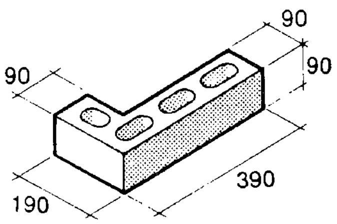
H 10.01*

Standard whole
(also available with two cores)



H 10.02*

Half
(also known as H 20.18)



H 10.03*

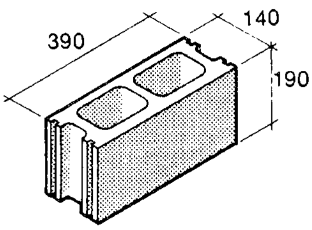
Corner

Shaded blocks are types not usually available ex-stock.

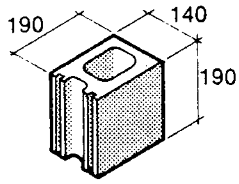
* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

15 Series

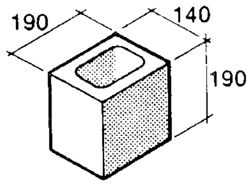
These diagrams cover standard block types in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



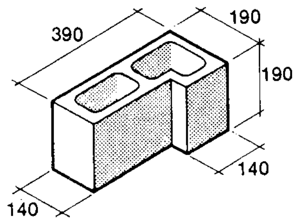
15.01
Standard whole



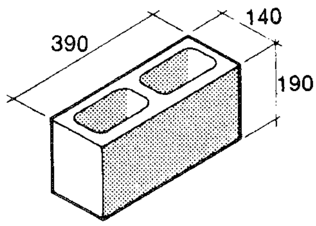
15.02*
Half (frog)



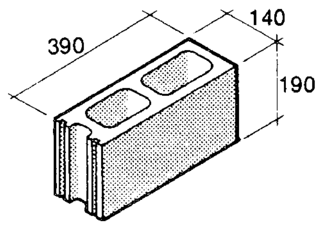
15.02*
Half
(also known as 20.20)



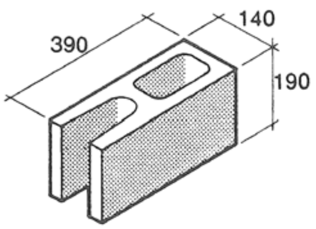
15.03
Corner



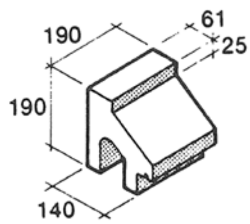
15.04
Plain ends



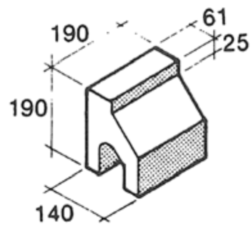
15.04*
Plain end (frog)



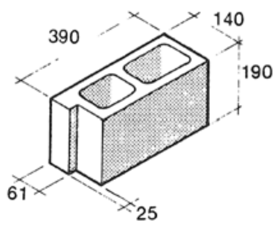
15.05
Open end



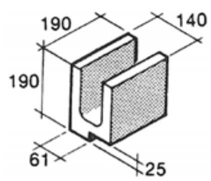
15.08
Sill (projecting)



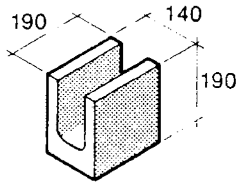
15.08
Sill (flush)



15.09
Rebate whole



15.11
Rebated lintel



15.12
Lintel & half end-closer

Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

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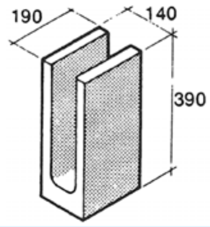
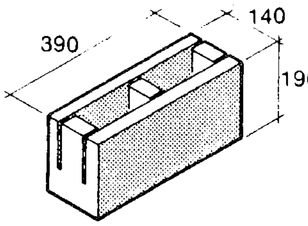


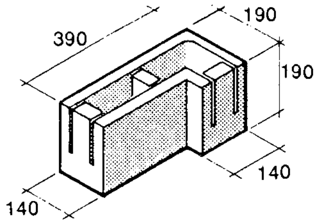
15.13

Deep lintel & full end-closer



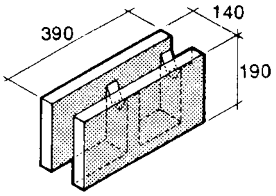
15.14

Knock-in bond beam
(also available with one open end)



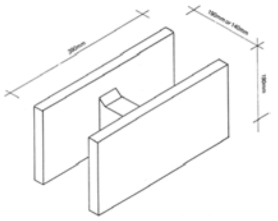
15.15

Corner bond beam, right hand
(left hand also available)



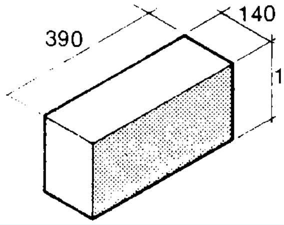
15.16*#

Open end bond beam, depressed web



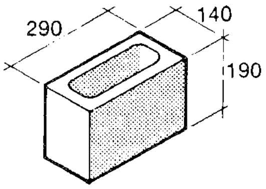
15.16H*#

Depressed web solid fill H shape



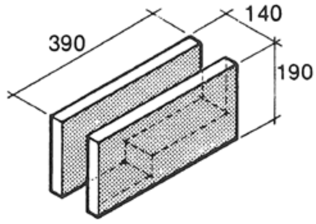
15.17

Solid whole



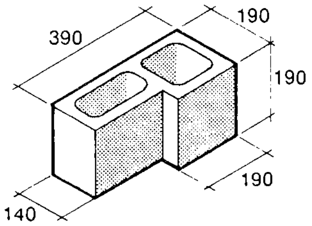
15.19*

Three quarter
(also known as 30.20)



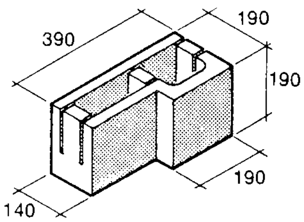
15.24

Channel open end bond beam



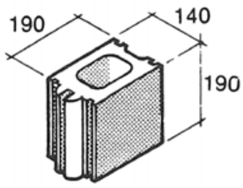
15.30*

Standard 200 mm pier



15.30*

Standard 200 mm pier, right hand
(left hand also available)



15.32

Control joint

Shaded blocks are types not usually available ex-stock.

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\# The 15.16H Block is manufactured by Mitchell Concrete Limited, Bowers Brothers Concrete Limited, and Firth Industries will manufacture to order.

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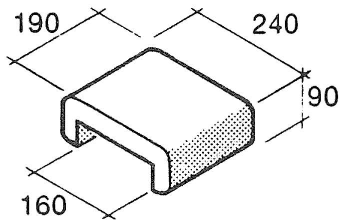
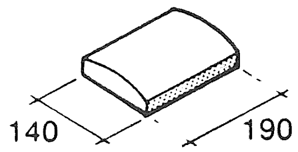
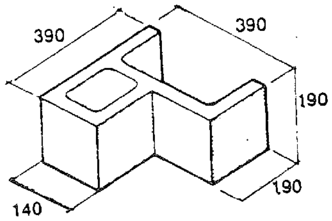


15.64
Capping block (overlapping)



15.64
Capping block (flush)



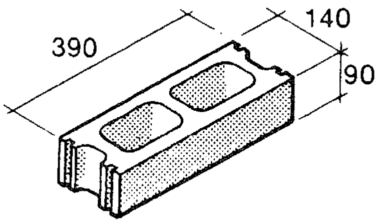
15.35*
Pilaster H type

Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

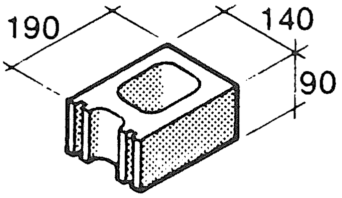
H 15 Series

These diagrams cover standard block types currently in production in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



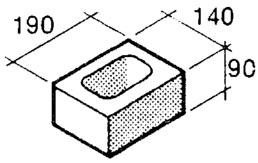
H 15.01*

Standard whole



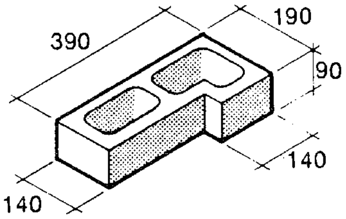
H 15.02*

Half (frog)



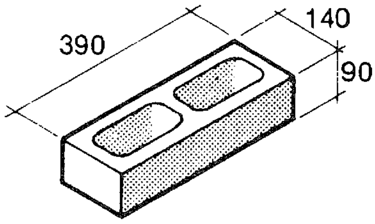
H 15.02*

Half



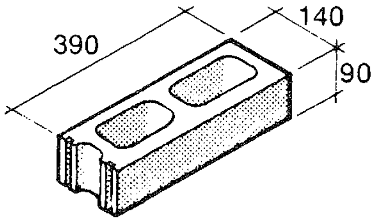
H 15.03*

Corner



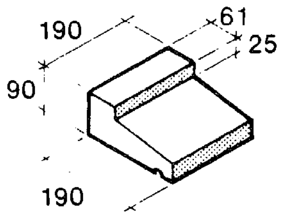
H 15.04*

Plain ends



H 15.04*

Plain end (frog)



H 15.08*

Sill (projecting)

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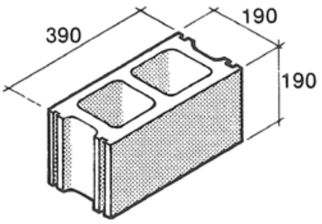
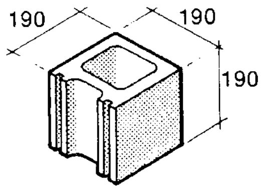
20 Series

These diagrams cover standard block types in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



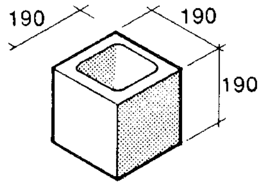
20.01

Standard whole



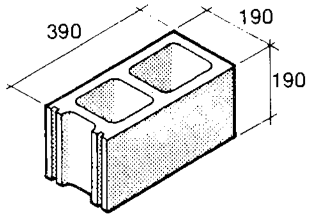
20.02

Half (frog)



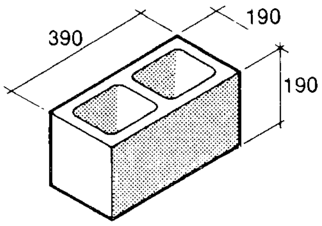
20.02

Half



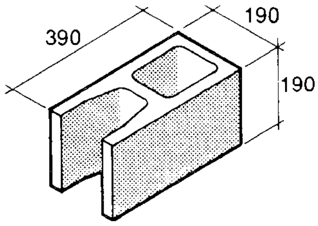
20.03

Corner



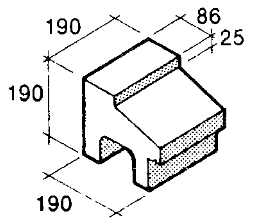
20.04

Plain ends



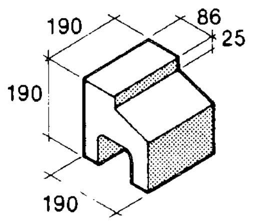
20.05

Open end



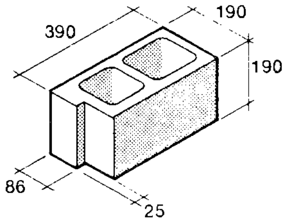
20.08

Sill (projecting)



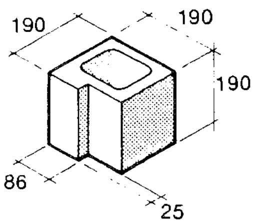
20.08

Sill (flush)



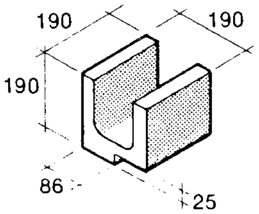
20.09*

Rebate whole



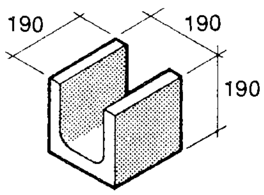
20.10

Rebate half



20.11*

Rebated lintel



20.12

Lintel & half end-closer

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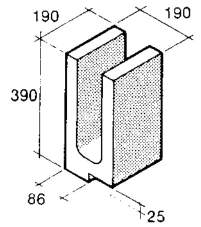
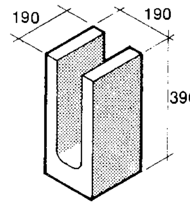


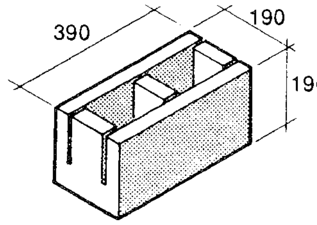
20.13*

Deep lintel & full end-closer
(rebated)



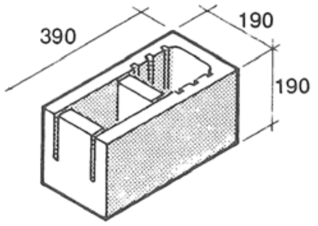
20.13

Deep lintel & full end-closer



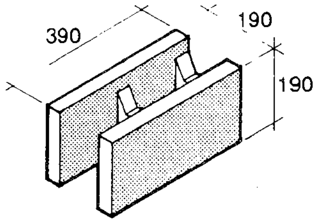
20.14

Knock-in bond beam



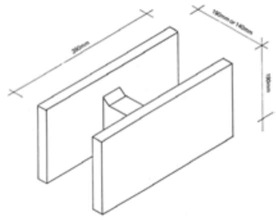
20.15

Corner bond beam



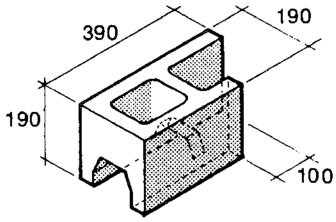
20.16*

Open end bond beam, Depressed web



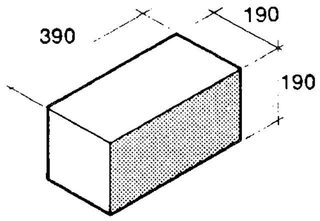
20.16H*#

Depressed web solid fill H shape



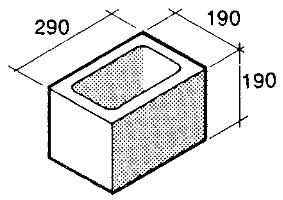
20.16

Right hand clean out
(left hand also available)



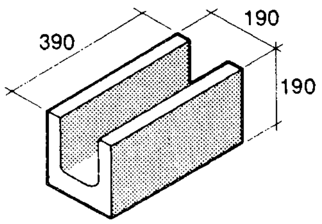
20.17*

Solid whole



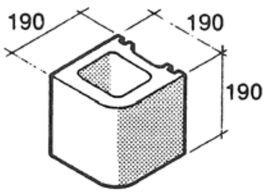
20.19

Three quarter
(also known as 30.02)



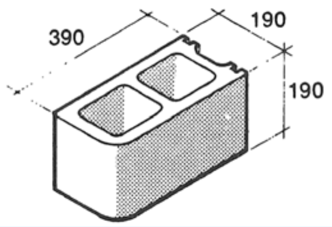
20.23*

Channel bond beam



20.26

Half single bull nose



20.27

Whole single bull nose

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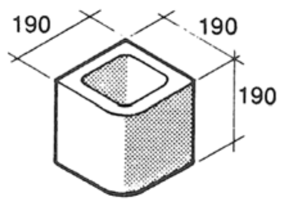
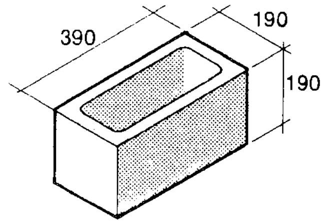
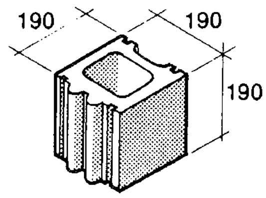


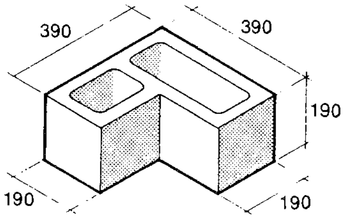
20.28
Half double nose



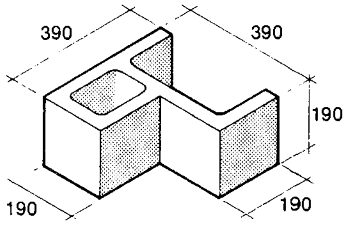
20.30
Standard 200 mm pier



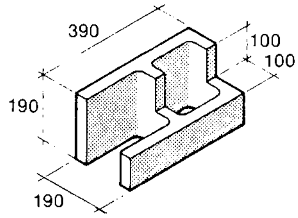
20.32*
Control joint



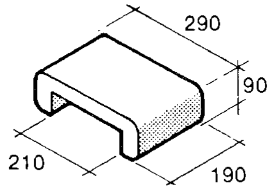
20.33
'L' pier, square end



20.35
Pilaster H type



20.45*
Header



20.64*
Capping block

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H 20 Series

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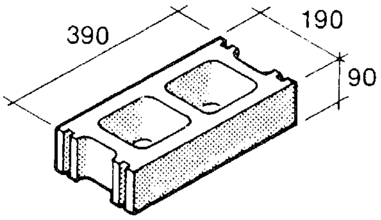
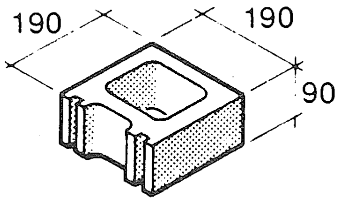
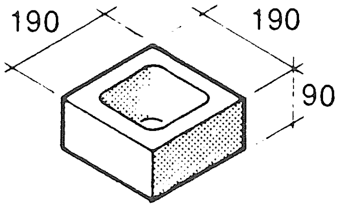


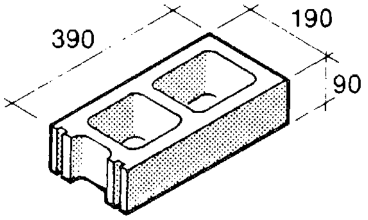
H 20.01*
Standard whole



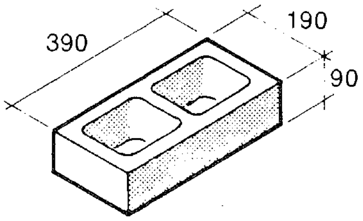
H 20.02*
Half (frog)



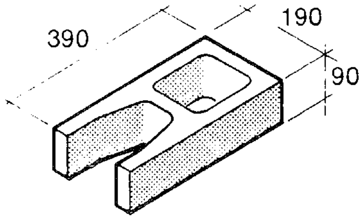
H 20.02*
Half



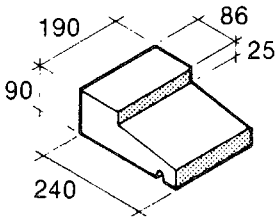
H 20.03*
Corner



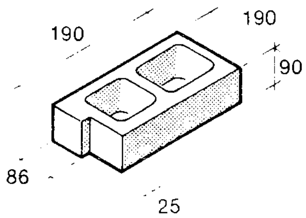
H 20.04*
Plain ends



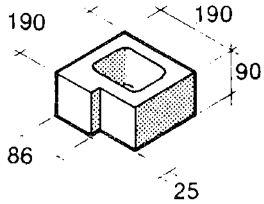
H 20.05*
Open end



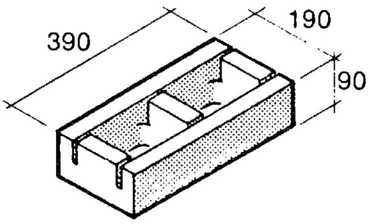
H 20.08*
Sill (projecting)



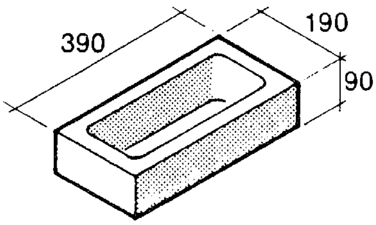
H 20.09*
Rebate whole



H20.10*
Rebate half



H 20.14*
Knock-in bond beam



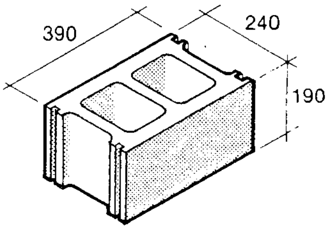
H 20.30*
Standard 200 mm pier

Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

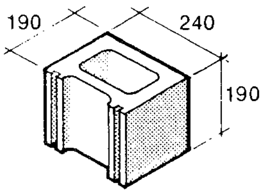
25 Series

These diagrams cover standard block types in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



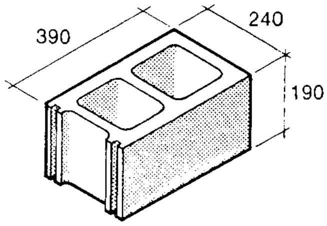
25.01*

Standard whole



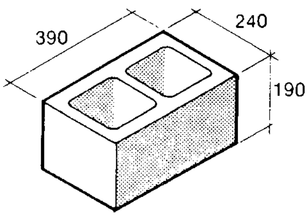
25.02*

Half (frog)



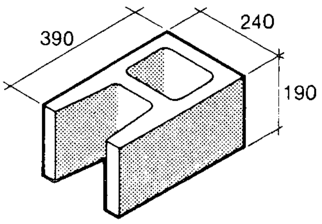
25.04*

Plain end (frog)



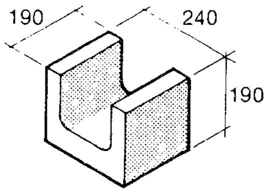
25.04*

Plain ends



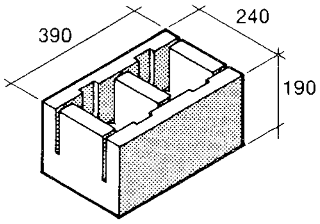
25.05*

Open end



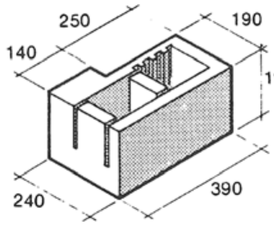
25.12*

Lintel & half end-closer



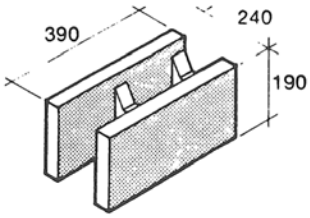
25.14*

Knock-in bond beam
(also available with one open end)



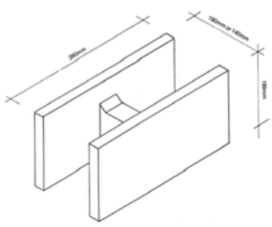
25.15*

Left hand corner bond beam
(right hand also available)



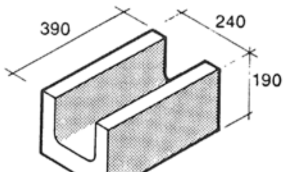
25.16

Open end bond beam, depressed web



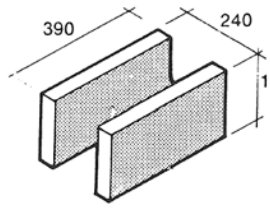
25.16H*#

Depressed web solid fill H shape



25.23

Channel bond beam



25.24

Channel open end bond beam

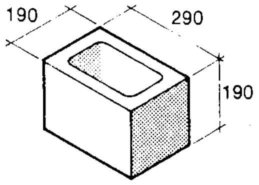
Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

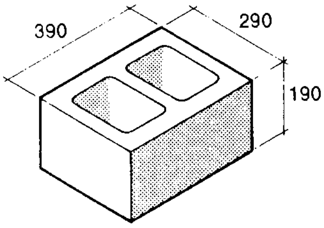
\# The 25.16H Block is manufactured by Mitchell Concrete Limited, Bowers Brothers Concrete Limited, and Firth Industries will manufacture to order.

30 Series

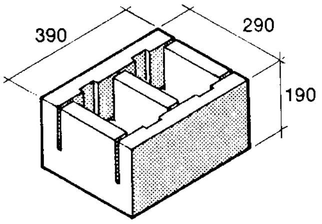
These diagrams cover standard block types currently in production in New Zealand but not necessarily available from all plants. Specifiers are therefore recommended to contact respective manufacturers to check the types of blocks available in the district of the job in hand.



30.02*
Half
(also known as 20.19)



30.04*
Plain ends



30.14*
Knock-in bond beam

Shaded blocks are types not usually available ex-stock.

* Blocks are not available from all plants and direct reference should be sought from manufacturers, see *NZCMA Member Companies* on page 3 of this section.

Split Blocks

A wide range of uses including facings, veneers, screens, planter boxes, feature walls and fences can be made of solid concrete masonry split blocks.

Most manufacturers produce a wide range of split units in quarter, half, three-quarter and full course heights. Some plants produce units with bolstered or pitched faces, and blocks from a wide range of aggregate textures and colours are also available.

Specifiers should therefore check with respective manufacturers to ensure that the chosen units are available in the district of the job in hand.

Split units are usually referred to by name rather than by code number. However, they are serialised in terms of course height. This is different to the coding of standard blocks by width. Unless otherwise specified, all split blocks are 90 mm wide.

Figure 1: Veneer Split Block

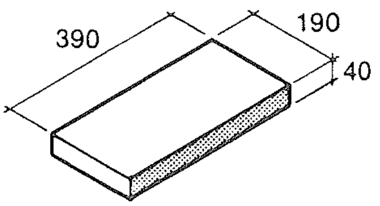
Figure 2: Structural Split Block

Figure 3: Split Block Wall
(Image Source: http://www.blockwall.org/)

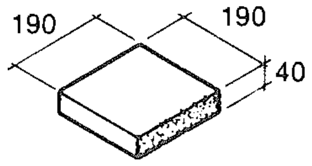
50 Series

(40 mm Actual Height)

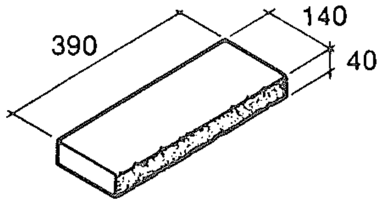
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.



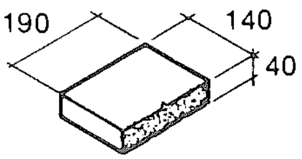
Unsplit paving slab
(formerly known as 5.17)



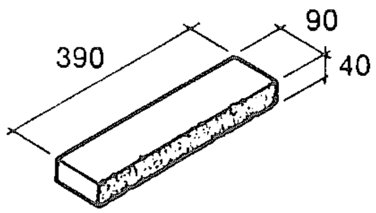
Split half
190 mm wide



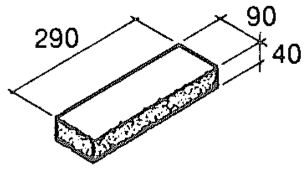
Split full
140 mm wide



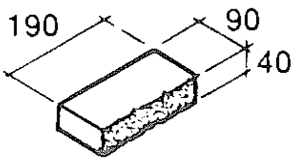
Split half
140 mm wide



Split full
90 mm wide



Split three-quarter
90 mm wide

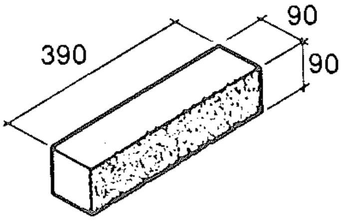


Split half
90 mm wide

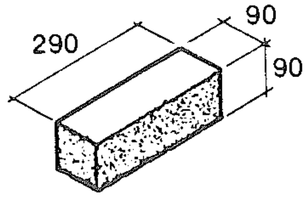
100 Series

(90 mm Actual Height)

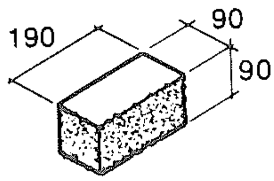
Not all the blocks shown in these diagrams are manufactured by all plants so specifiers should check with respective manufacturers to ascertain the full range of units are available in the district of the job in hand.



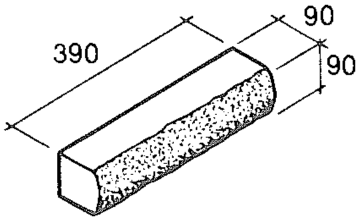
Split full



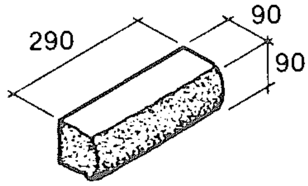
Split three-quarter



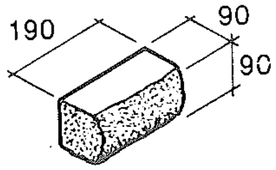
Split half



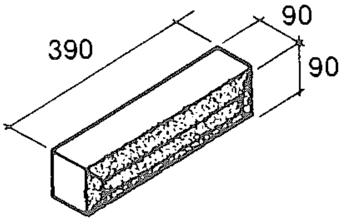
Pitched full



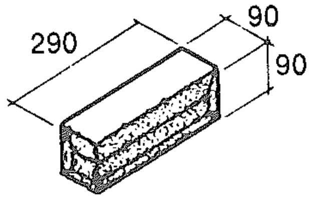
Pitched three-quarter



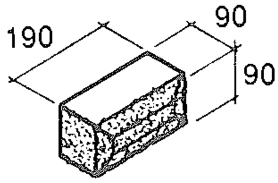
Pitched half



Bolstered full



Bolstered three-quarter

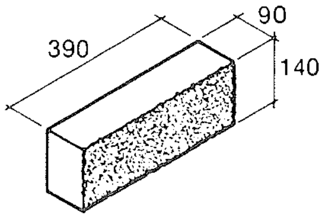


Bolstered half

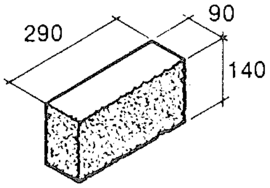
150 Series

(140 mm Actual Height)

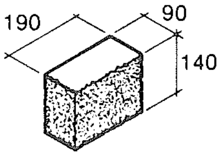
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.



Split full



Split three-quarter

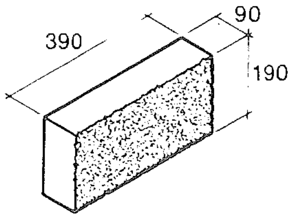


Split half

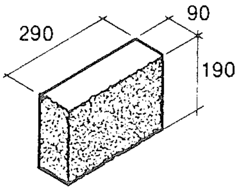
200 Series

(190 mm Actual Height)

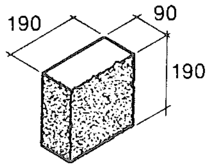
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.



Split full



Split three-quarter



Split half

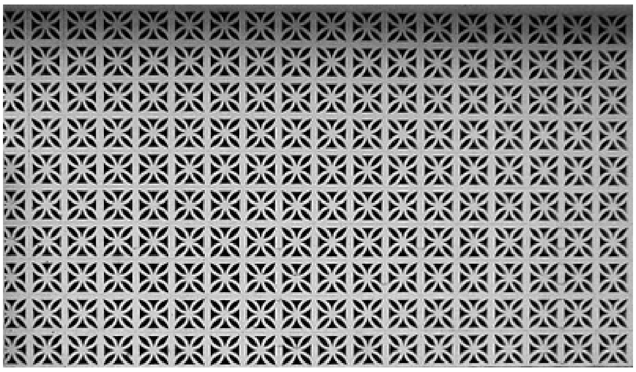
Special Finished Screen Blocks

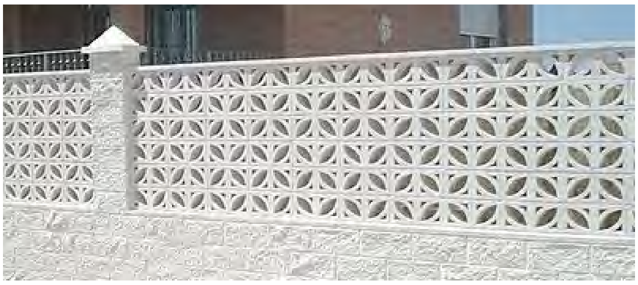
Concrete blocks are also available in a wide range of screen patterns, special finishes and textures, some of which are shown in the following diagrams.

Not all blocks shown in the diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of special or textured units available in the district of the job in hand.

In some cases manufacturers produce special or textured units other than those shown, and some plants produce range of supplementary textured units such as bond beam units, rebated units, fractional units and so on, all with matching faces.

Again, specifiers should check the availability of such units with respective manufacturers.

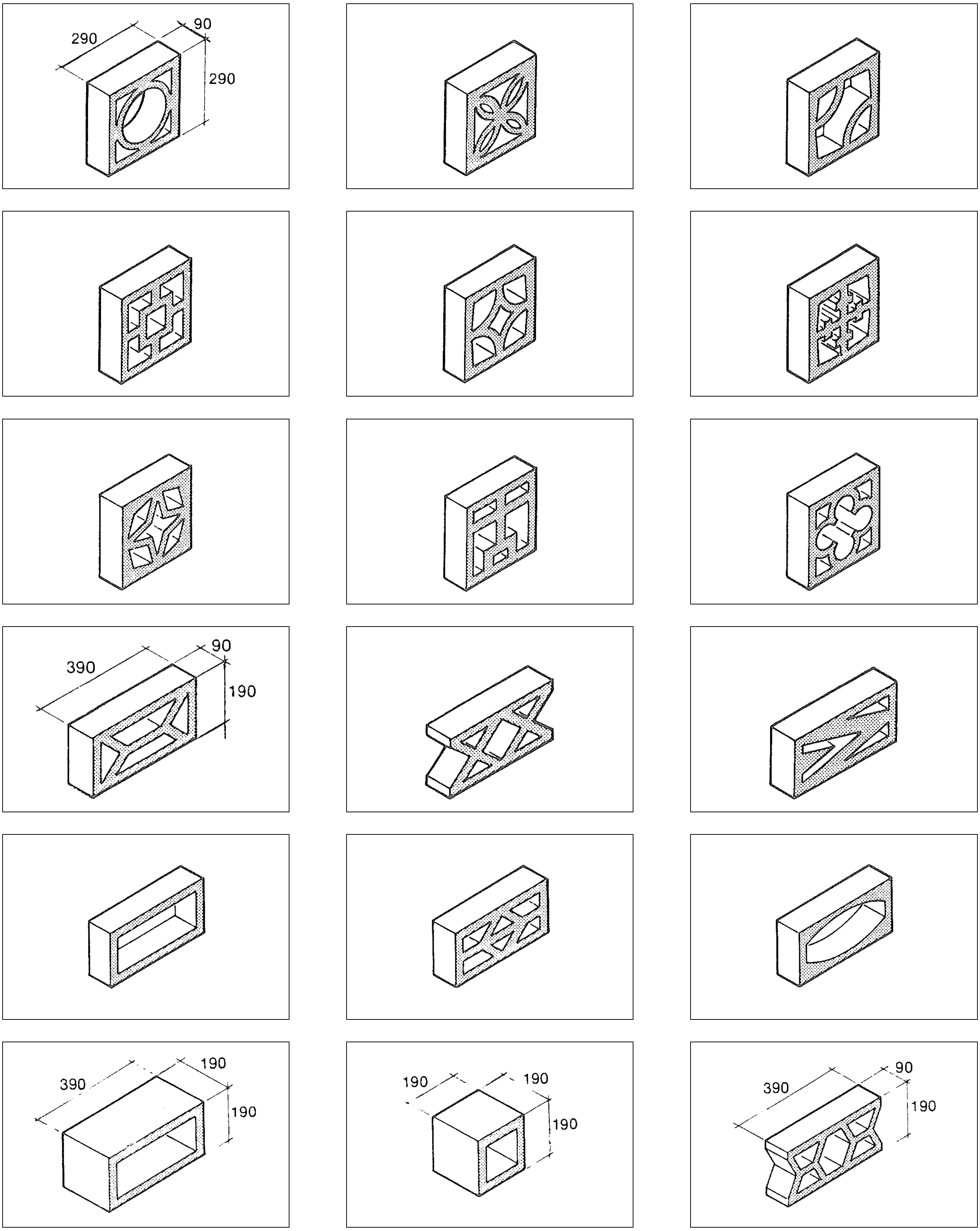




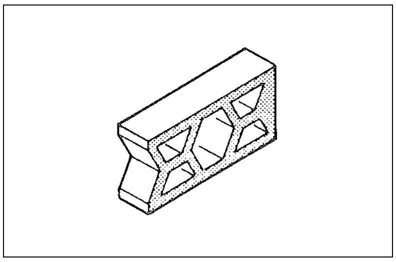
Images Source: http://www.moderndesigninterior.com/2011/01/mid-century-decorative-concrete-screen.html

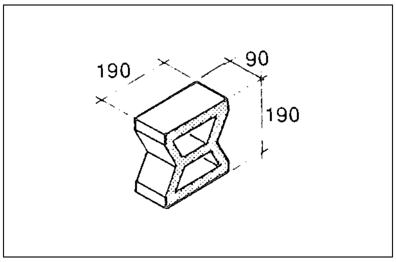
Typical Screen Blocks

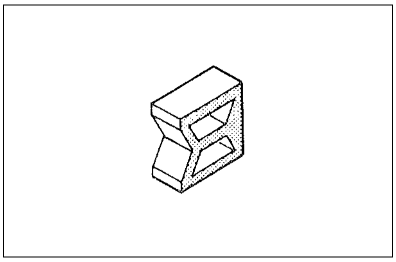
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.

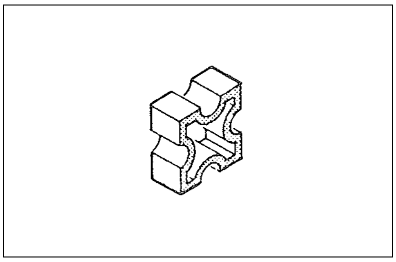


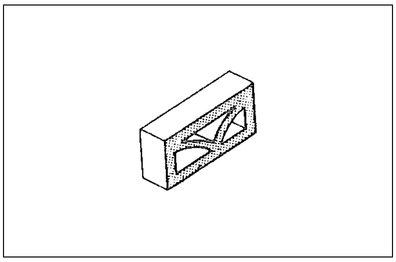
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.

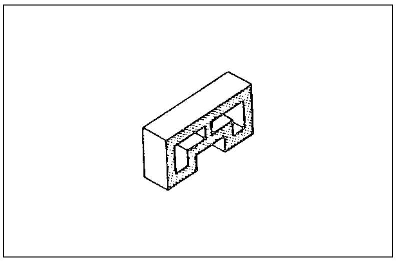






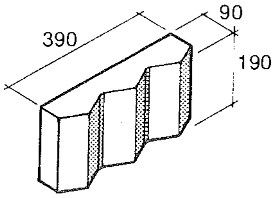




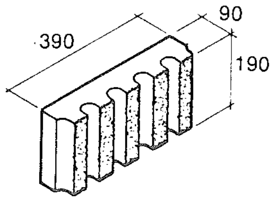


Textured blocks

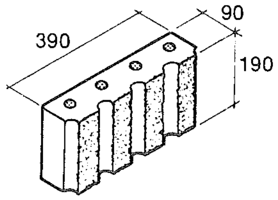
Not all the blocks shown in these diagrams are manufactured by all plants, so specifiers should check with respective manufacturers to ascertain the full range of units available in the district of the job in hand.



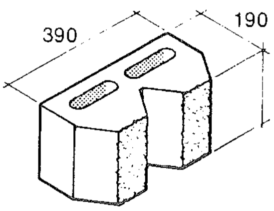
Tri-face



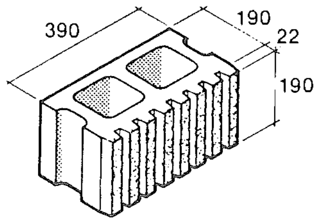
Deep grooved



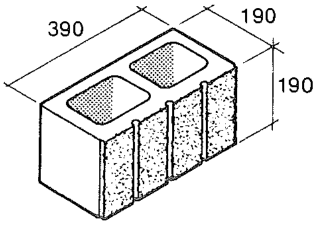
Rough grooved veneer



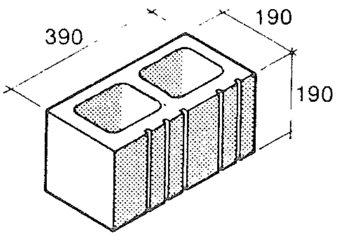
Split face hexagonal



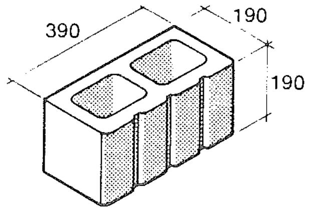
Split fluted, or fine grooved



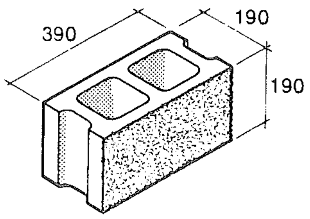
Rough grooved



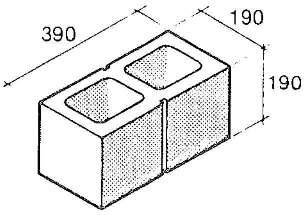
Random scored



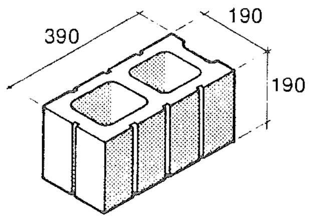
Rounded grooved



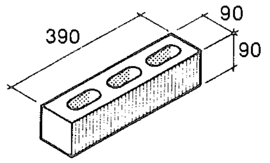
Split face



Plain scored



Scored standard



Striated

1.3 Modular Wall Masonry

Masonry Units

For reasons of ease of manufacture, storage, handling and construction, concrete masonry is produced in modular units.

To be able to satisfactorily design or construct a concrete masonry building or structure, it is necessary to understand the basics of masonry units:

- Modular dimensions
- Set-out
- Units available
- Construction details

Modular Dimensions

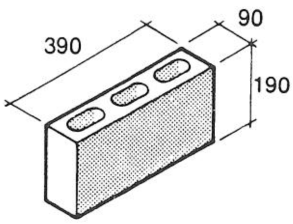
The fundamental modular unit is 400 mm long by 200 mm high (nominal dimensions). This unit is called "the standard whole". There is also a complimentary 200 mm long by 200 mm high unit called "the half".

These units are available in four different widths – nominally:

100 mm (10 series)
150 mm (15 series)
200 mm (20 series) – most commonly used unit
250 mm (25 series)

The actual size of the units is 10 mm less than the nominal size to allow for a10 mm mortar joint.

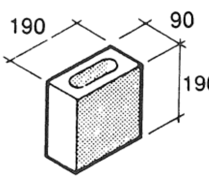
10 Series



10.01

Standard whole

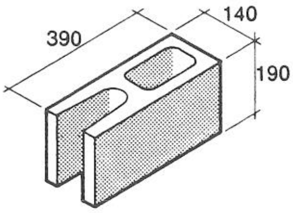
10 Series



10.02

Half

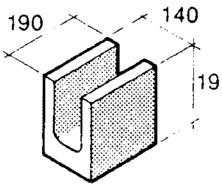
15 Series



15.05

Open end

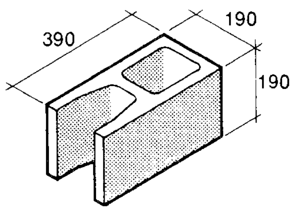
15 Series



15.12

Lintel & half end-closer

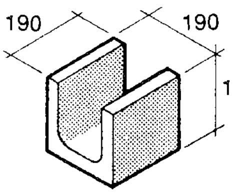
20 Series



20.05

Open end

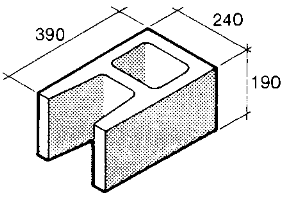
20 Series



20.12

Lintel & half end-closer

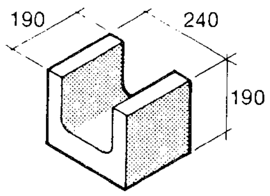
25 Series



25.05

Open end

25 Series



25.12

Lintel & half end-closer

Set-out

The nominal modular length set-out of the units is in multiples of 200 m.

This is based on the nominal block size comprising 190 mm actual block size plus a 10 mm mortar joint.

The actual set-out dimension is 10 mm more, or 10 mm less, than the nominal set-out dimension. This is due to the addition or deduction of one mortar joint in relation to the number of masonry units as shown below.

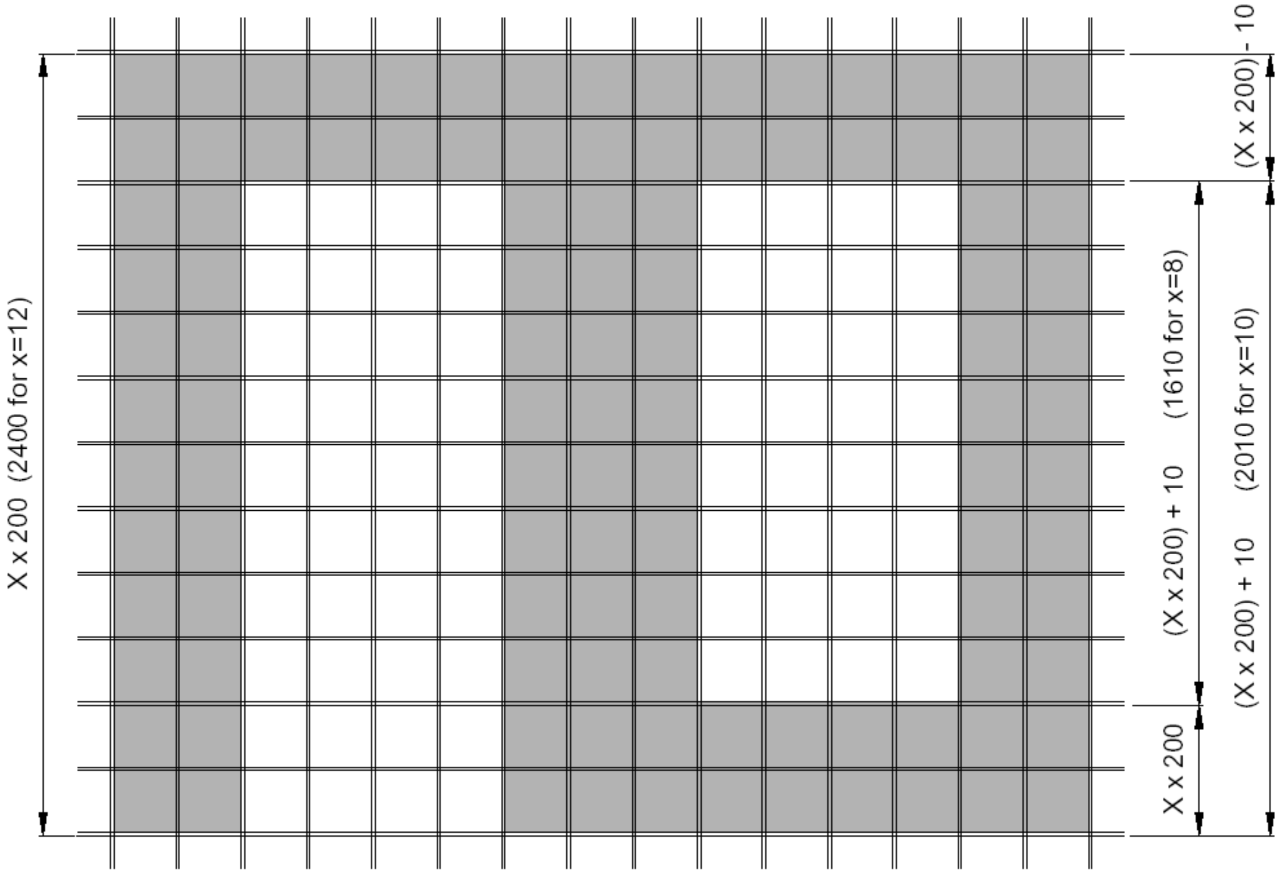


Figure 1: 200 Module Set-out: Elevation

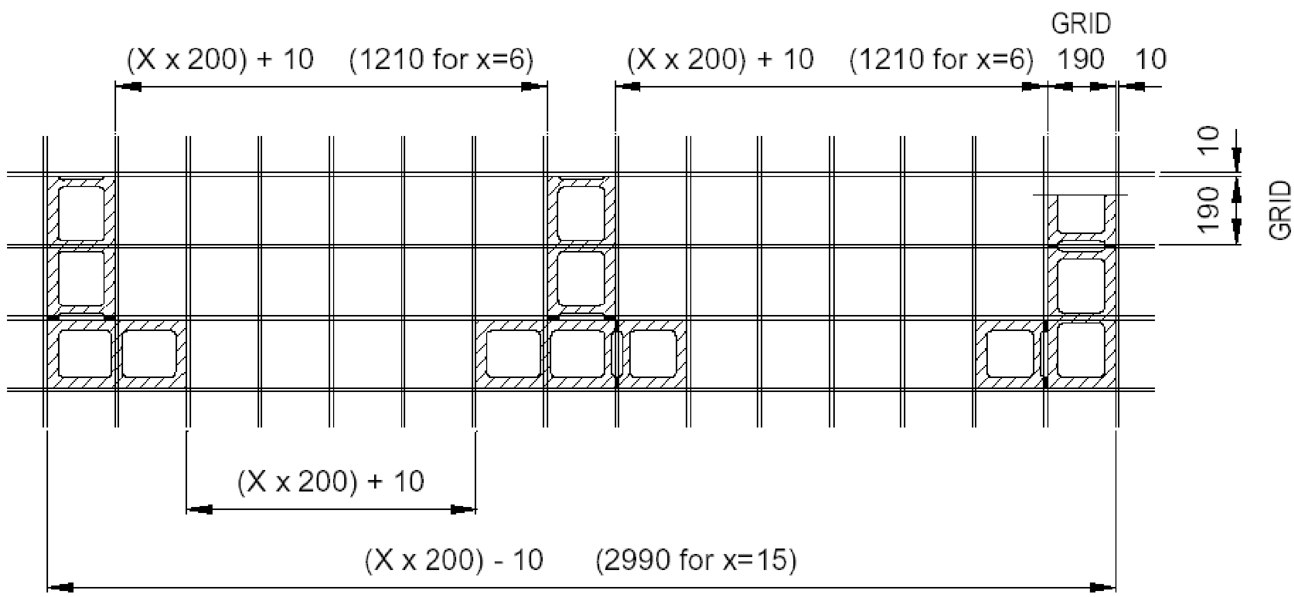


Figure 2: 20 Series Set-out: Plan

In order not to interfere with the 200 x 200 mm module, special attention is required at corners. The 20 series corner block has one plain end. The 10, 15 and 25 series have corner blocks manufactured to fit the 200 mm module.

The set-out of internal walls with the 20 series maintains the 200 module (Figure 2). Because of their thickness, the set out of internal walls with the 15 or 25 series requires either the addition or deduction of 50 mm (Figures 3 and 4).

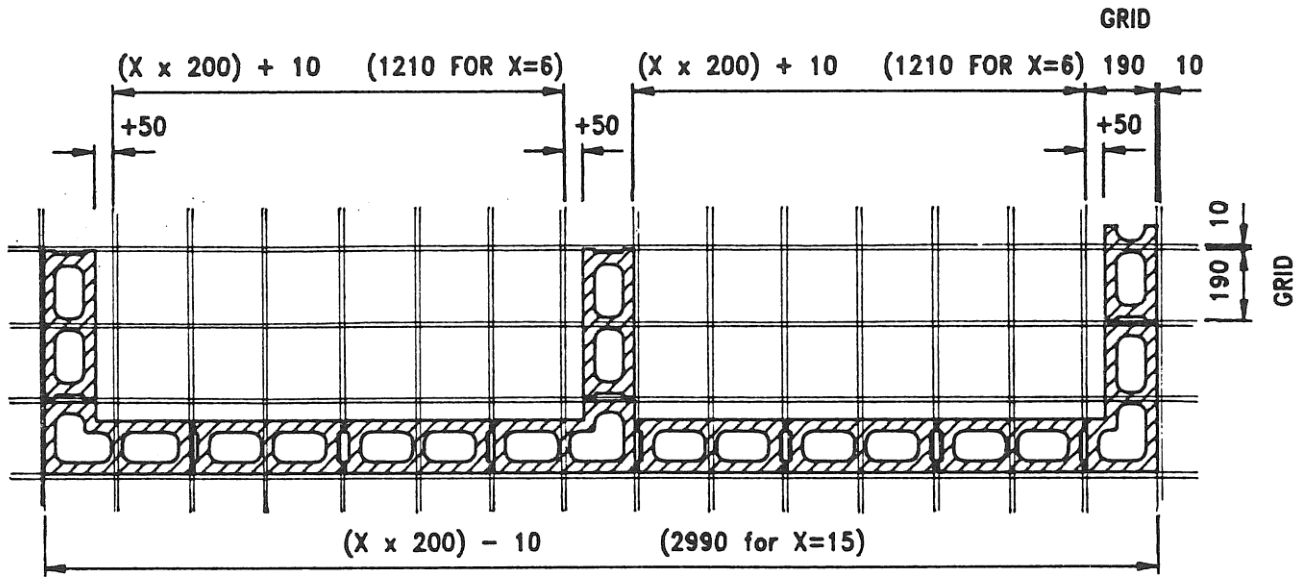


Figure 3: 15 Series Set-out: Plan

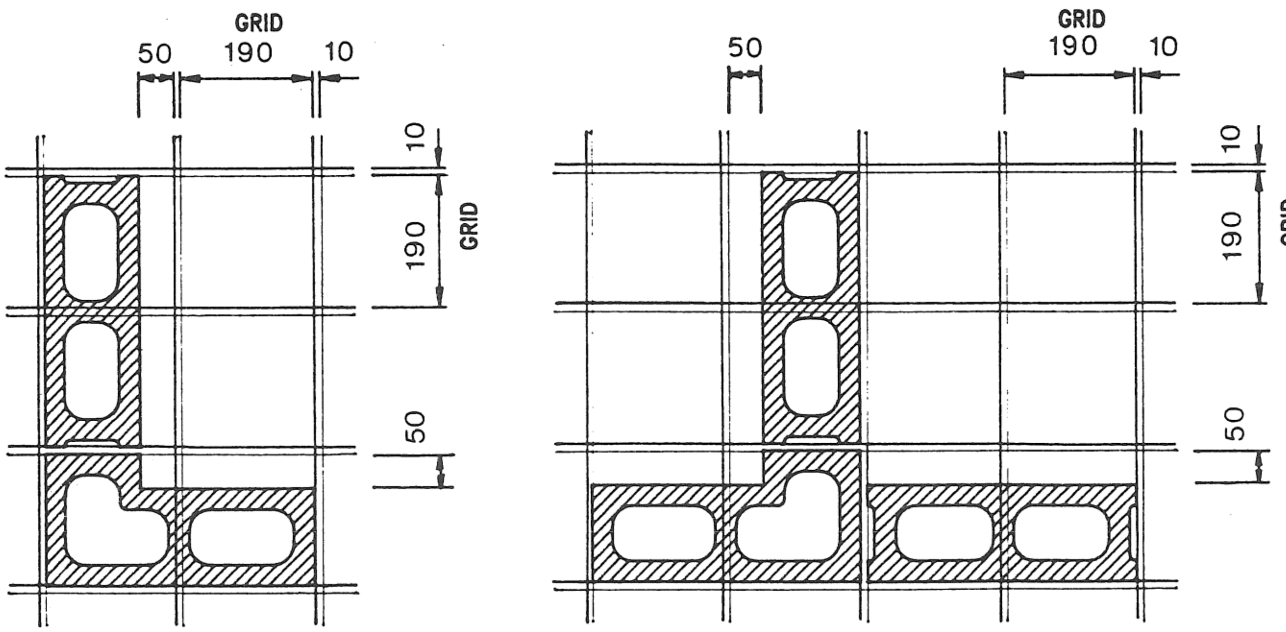


Figure 3(a): Enlarged Details of the Corner and Interior Wall of the 15 Series

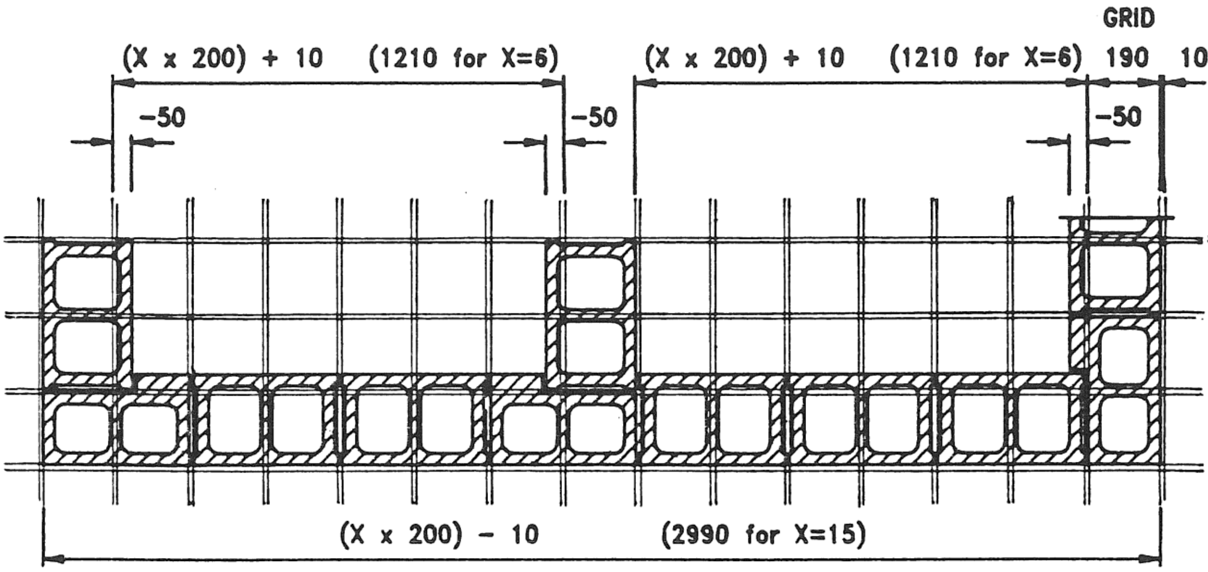


Figure 4: 25 Series Set-out: Plan

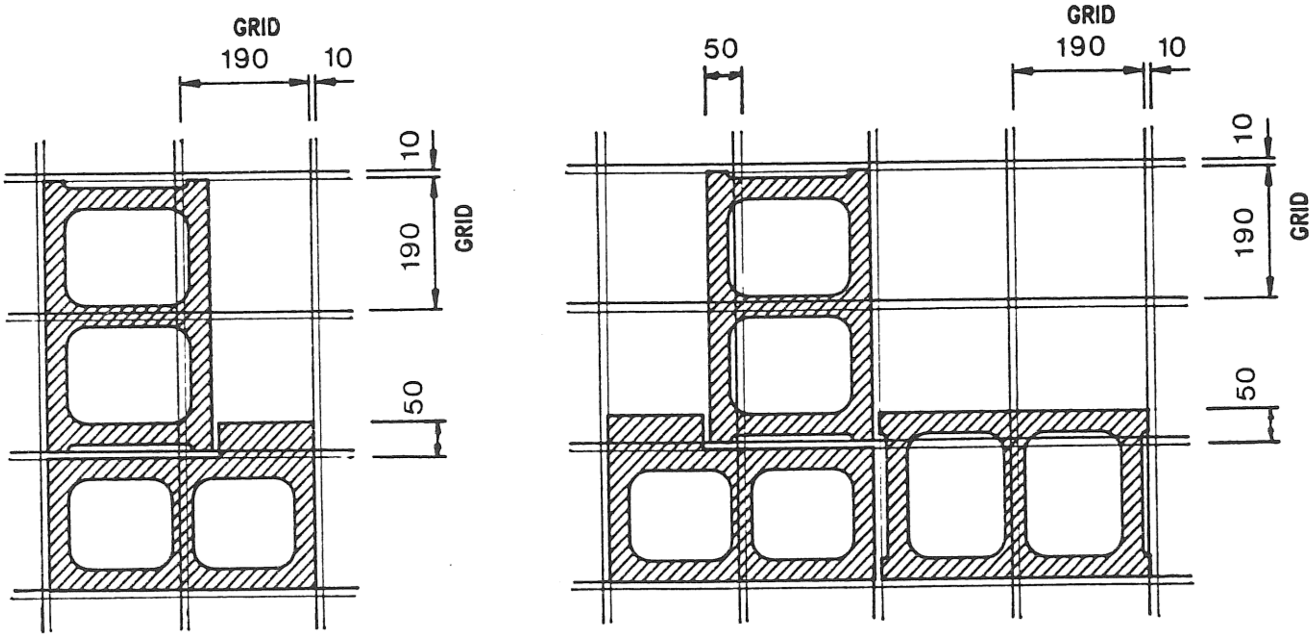


Figure 4(a): Enlarged Details of the Corner and Interior Wall of the 25 Series

Units Available

An extensive range of masonry units, including units for specific purposes, e.g. sills, is illustrated elsewhere in the "General" section of this manual. However, due to lack of demand for some units, not all are readily available.

Check the availability of particular units with local suppliers.

Construction Details

The successful detailing of modular masonry requires careful thought to be given to other building components (doors, windows, meter boxes, etc.) their finishes and fixing, and how to relate them to the modular system.

Details relate to a number of requirements, possibly including:

Type of Wall:	load bearing in-fill panel veneer cantilever
Structural Engineering:	Reinforcing concrete filling bond beams & lintels overturning resistance movement control joints
Construction Technique:	slab edge - set down/flush upstands ceiling/soffit framing external/internal finishes services and fitting insulation acoustics compatibility of materials other modular components
Local Body By-Laws:	fire resistance building heights
Weathering:	exposure
Appearance:	height of openings all uncut blocks selection of materials scale finishing trim hardware electrical switches electrical outlets fixtures
Maintenance:	cost (initial and in-use)

Satisfactory construction details depend on clearly determining the relevant requirements for the building, and then co-ordinating all the details so that these requirements are met.

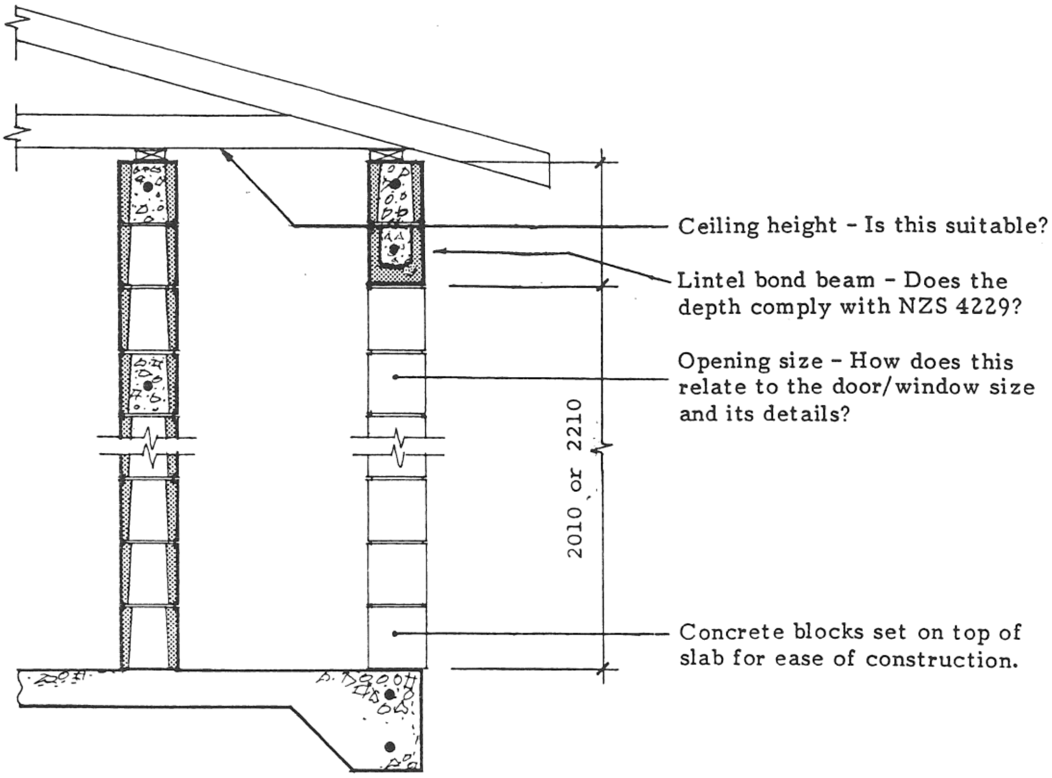
Unco-ordinated details are likely to detract visually from the building and could result in time delays and higher costs.

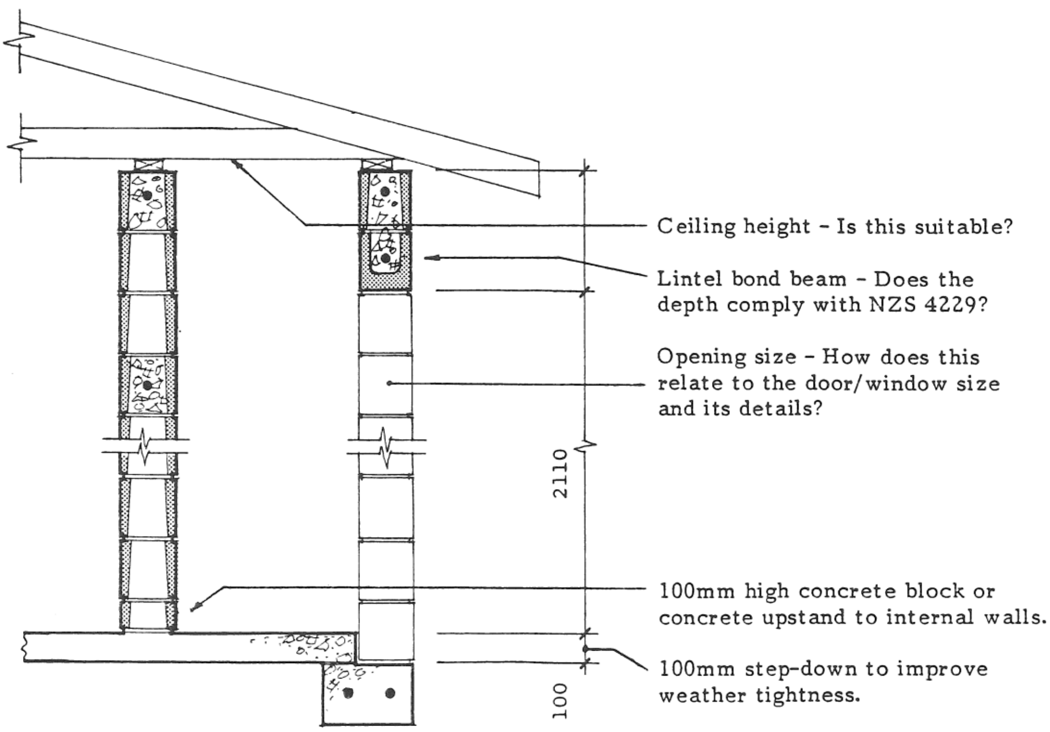
Assistance in determining some of these requirements can be found elsewhere in the manual and some matters need to be carefully checked – particularly structural requirements and exterior coatings.

Examples of general details are shown elsewhere in this manual, predominantly the sections entitled “Construction Details” and “Veneer Walls”.

On the pages that follow, are a selection of details which illustrate the required co-ordination. These details show a range of possible solutions to some of the requirements listed above, particularly those that occur at door and window openings.

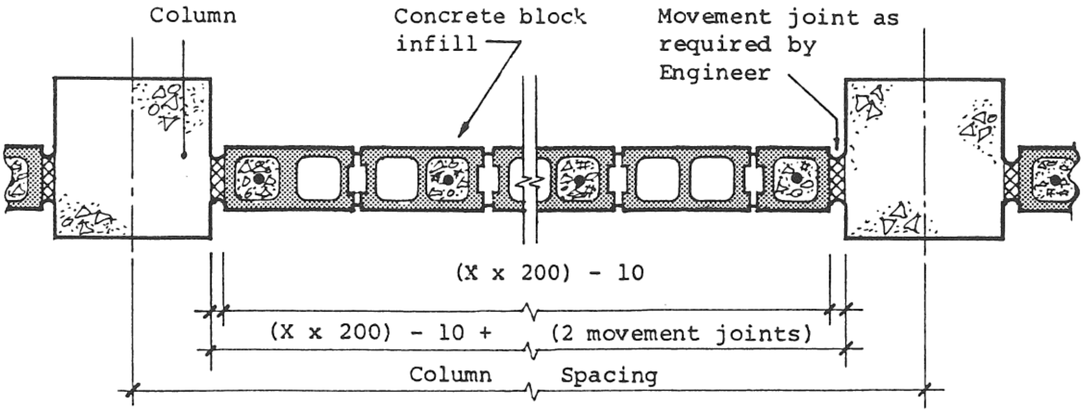
To comply with H1 of the New Zealand Building Code, thermal insulation may be required, depending on the required R-value.





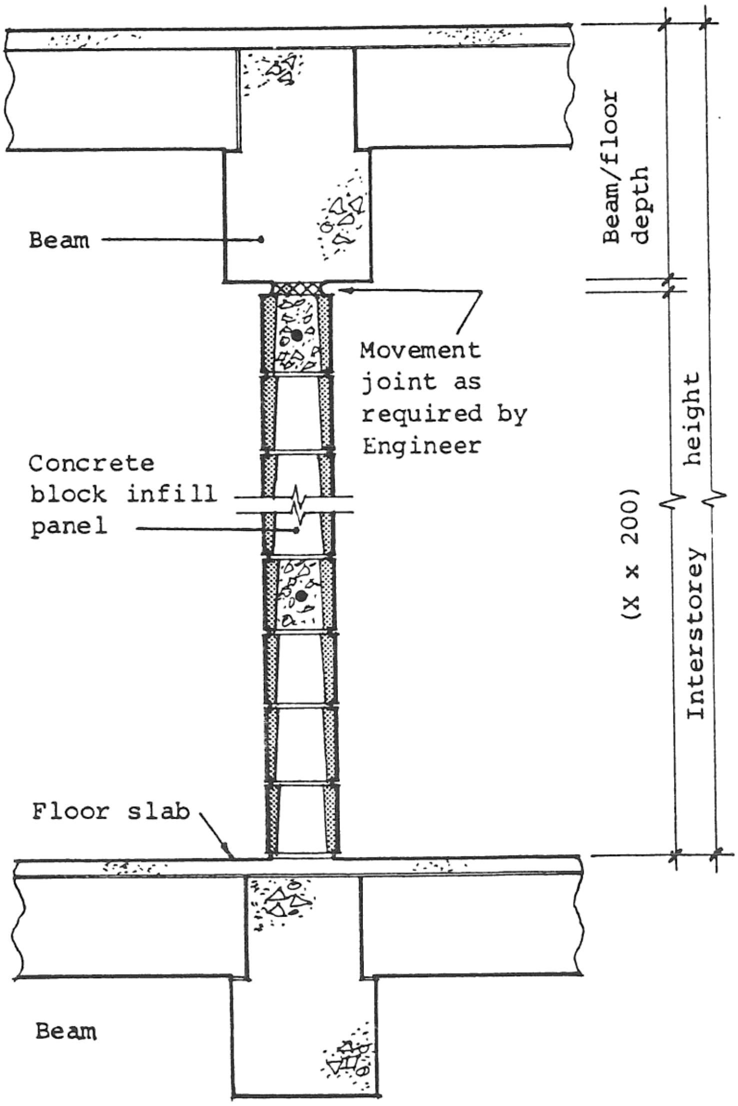
Vertical Modular Set-out

See "Construction Details" section for further illustration of co-ordination of details.

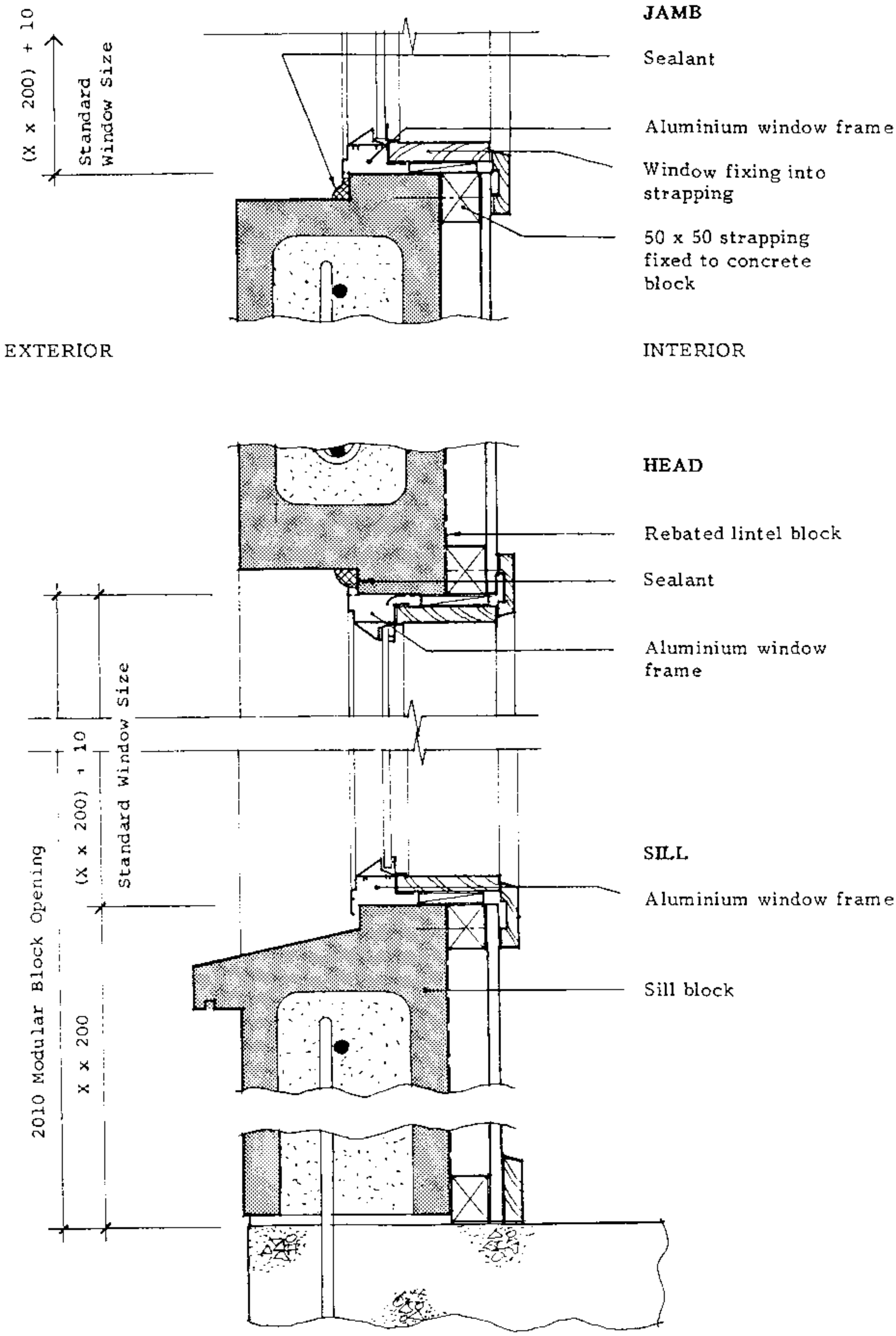


Infill Set-out: Plan

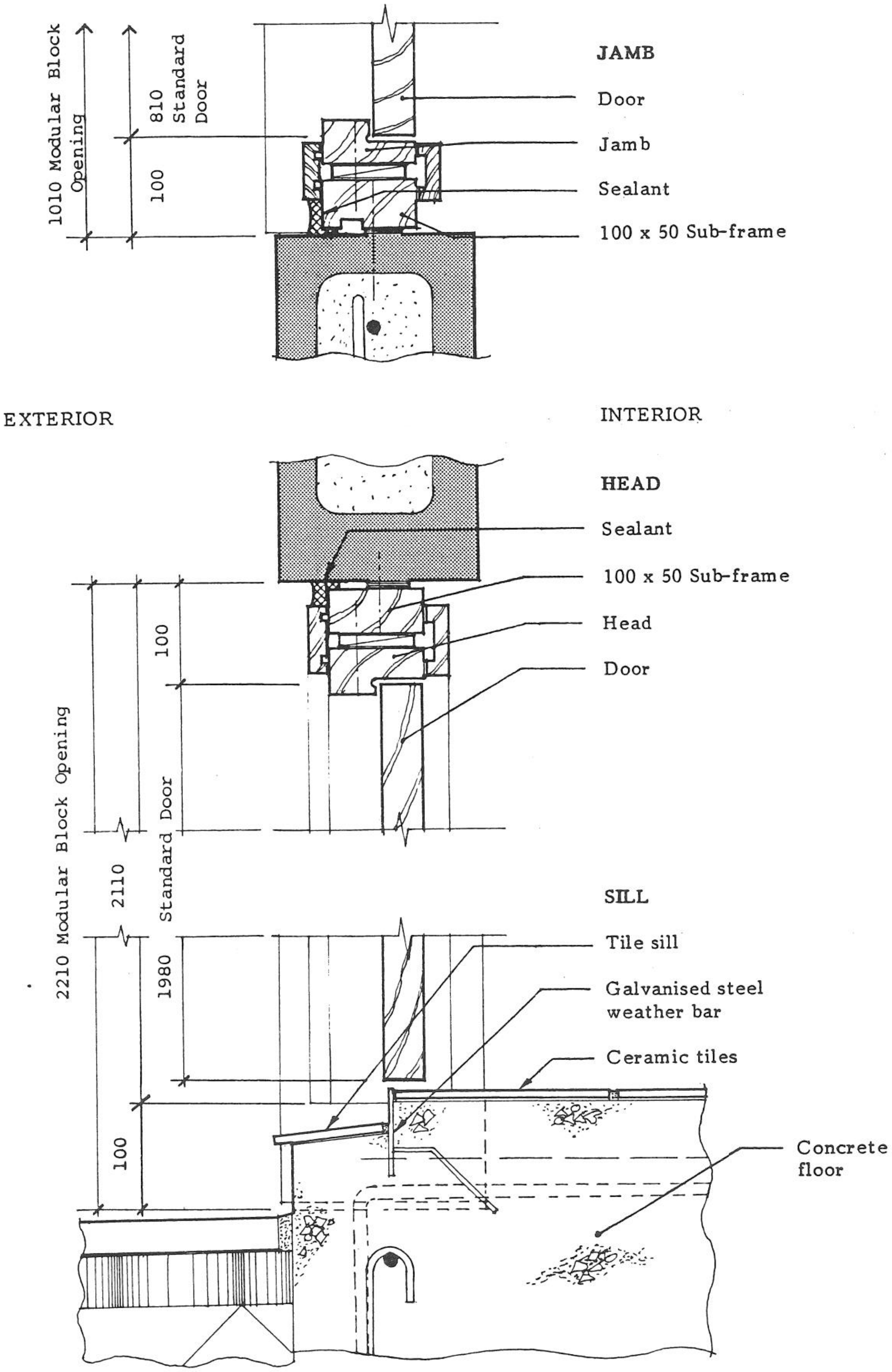
N.B. Column size to be determined in conjunction with size of infill panel, movement joints and column spacing.



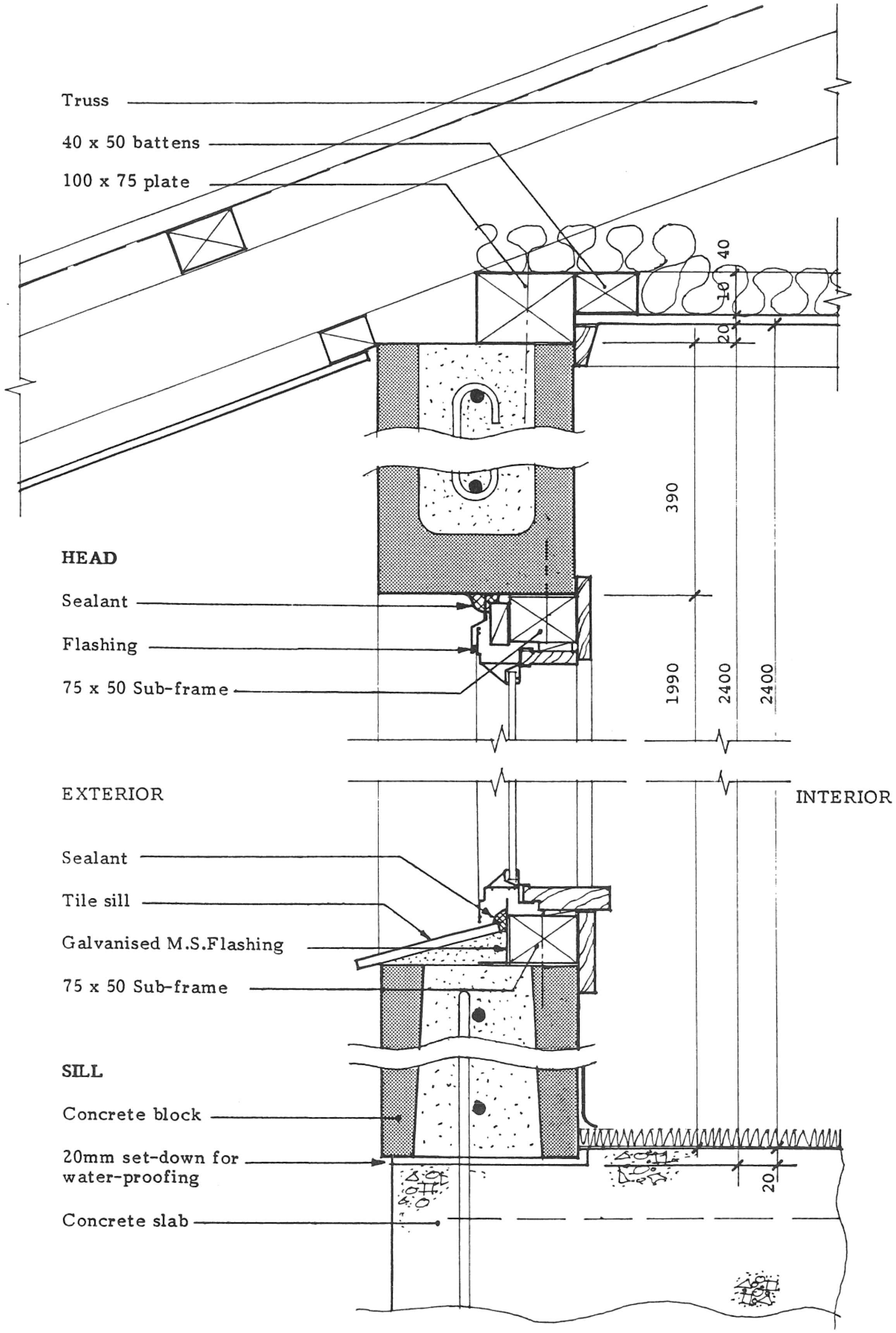
Infill Set-out: Section



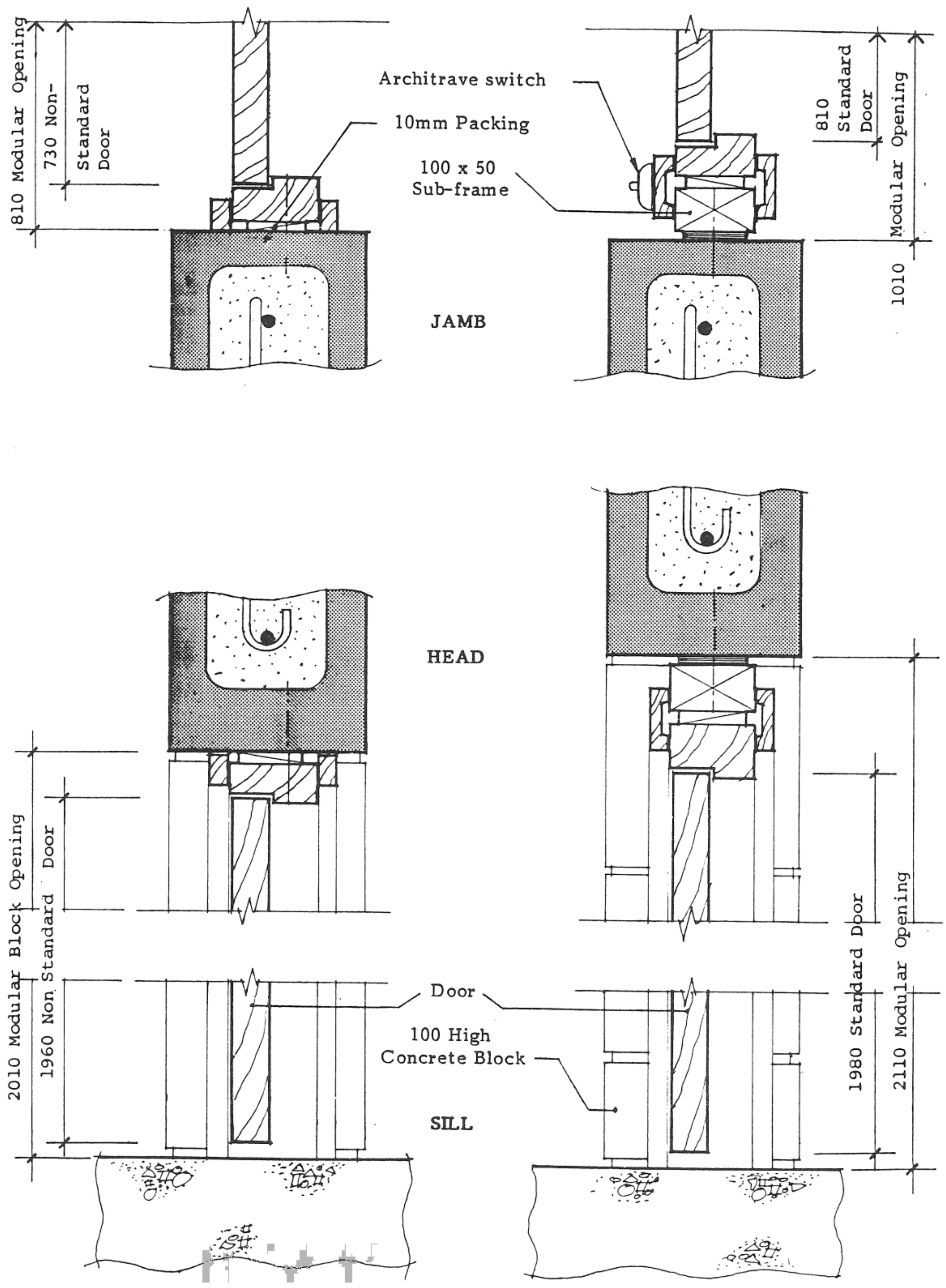
Aluminium Window with Rebated Concrete Block, Head, Jamb and Sill



Timber Door and Timber Sub-frame
Concrete Block Wall Set-down into Concrete Slab



Aluminium Windows in Timber Sub-frame
Concrete Block Wall Set-down into Concrete Slab



Internal Door
Concrete Blocks Set on Slab

Internal Door
100 mm Starting Block

1.4 Mortar and Mortar Joints

Summary of Requirements

Mortar						
Minimum Strength:	12.5 MPa at 28 days for structural masonry.					
Minimum Bond Strength:	200 kPa at 7 days for non-structural veneer.					
Durability:	***M4 Exposed/Coastal***	Cement 1 part	:	Lime 0-0.25 part	:	Sand 3 parts
	M3 Exterior/Inland	Cement 1 part	:	Lime 0.50 part	:	Sand 4 parts
	M2 Interior	Cement 1 part	:	Lime 1 part	:	Sand 6 parts
	Admixture complying with AS 1478 can replace lime.					
Sand:	To comply with *NZS 3103 Specification for sands for mortars and plasters*, Class A. The sand should not contain more than 0.04% of chloride by dry weight of sand.					
Cement:	To comply with *NZS 3122 Specification for Portland and blended cements (General and special purpose).*					
Water:	To comply with *NZS 3121 Specification for water and aggregate for concrete.*					
Pigment:	Dosage not to exceed 3% dry weight of cement.					
Pre-bagged products of Dry Mortar meeting these requirements are available from Cemix Limited and Dricon.						

Joints	
Nominal Thickness:	10 mm.
Tolerance:	± 3 mm.
Bottom Joint:	May vary in thickness from 7 mm to maximum of 20 mm to accommodate foundation/floor tolerances.
Tooling:	Maximum depth 6 mm.

Introduction

Masonry units are usually bonded together by laying and bedding the units in Portland cement mortar. The thickness of the mortar joint is part of the modular system of blockwork. There are some patent mortarless systems available overseas.

The nominal bedding joint thickness for concrete masonry is 10 mm.

The pliable nature of mortar also resolves problems such as could occur where adjacent units are of different types and possibly of differing moulds. *AS/NZS 4455 Part 1 Masonry units, pavers, flags and segmental retaining wall units - Masonry units* allows a manufacturing tolerance of ± 3 mm in unit height dimensions so it is feasible that a unit of one type could be a little higher or lower than a unit of another type. This variance can be accommodated by mortar joints. Mortarless systems require units to be manufactured to a significantly higher degree of dimensional accuracy.

New Zealand Building Code

The New Zealand Building Code sets out requirements or both structural masonry and veneer construction. The materials and workmanship aspects of these two strands of masonry are covered by the New Zealand Standard *NZS 4210 Masonry Construction: Materials and Workmanship*.

Section 2.2 of this Standard sets out the various requirements for materials and workmanship for mortar and mortar joints.

Requirements for weathertightness are directly found within the *New Zealand Building Code E2 AS1 and AS3*. This issue is discussed further in Section 2.2 of this Manual.

Materials

Cement used in masonry mortars shall be Portland cement complying with *NZS 3122 Specification for Portland and blended cements (General and special purpose)*. Lime shall comply with BS 890 (Building limes), but may be replaced wholly or partially by admixtures provided the strength requirement will be maintained and the bond between units will not be impaired. It is important that a full understanding of admixtures and their effects should be gained before they are used. Tests should be carried out to check that the nominated admixture does not reduce the quality of the mortar and that the required compressive and bond strengths can be obtained.

More than one type of admixture should never be used together unless such usage has been proved to be compatible. Sands for mortars are specified in *NZS 3103 Specification for sands for mortars and plasters* and are defined therein as Class A which requires that 100% passes a.4.75 mm test sieve. Particle size distribution, particle shape and texture are controlled by a sand flow test as set out in *NZS 3111 Methods of test for water and aggregate for concrete*. Provision is made in *NZS 3103* for service records of sands to be kept and to be the basis of automatic approval of those sands recorded as being of the required standard.

In practice, each mortar sand has its own characteristics which influence the precise proportions used in mortar mixes. A well-graded sand may be used in larger proportions than otherwise. Experience is the best guide to a good mortar sand, but if there is any doubt it is suggested that advice be sought from a block manufacturer or licensed mason.

The presence of chloride salts from the use of unwashed beach sand can cause corrosion of ties and contribute to salt deposits appearing on the wall surface.

BRANZ recommendations are that 0.04% of dry weight of sand is the maximum level to be tolerated.

Water for mixing mortar should be clean, potable and comply with *NZS 3121 Specification for water and aggregate for concrete.*

Mortars may be coloured by adding liquid or powder pigment during mixing. A suitable pigment must maintain colour fastness under sunlight, be chemically stable in the alkalinity produced by the cement and have no detrimental effect on the setting time, permeability, workability or strength requirements of the mortar. Pigment should be added at a rate not exceeding 3% by weight of cement unless it can be shown that the increased concentrations have no detrimental effect on the bond strengths of the mortar, the maximum permitted dosage being 6%.

Mortar Mixes

The sand/cement ratio of satisfactory mortar is influenced by the characteristics of the sand and by the service requirements of the mortar. It is accordingly difficult to pre-specify the exact sand/cement ratios for masonry mortars. Acceptable mortar mixes, therefore, usually result from considerable experience on the part of the concrete blocklayer; and supported and proved by standard tests and performance in service.

The following mortar mixes are given as a guide to sand/cement ratios, by volume:

Durability	Portland Cement	Hydrated Lime*	Mortar Sand
M4 Very High	1	0-0.25	3.0
M3 High	1	0.5	4.5
M2 Medium	1	1	6.0

* Where lime is replaced by a patent admixture it is important that such admixture be used in strict accordance with the supplier's instructions and that it would allow the mortar to attain its strength and bond requirements.

It should be noted that mortar ingredients are usually measured by volume rather than by weight. In order that consistency may be achieved it is recommended that accurate gauging boxes or buckets be used in preference to shovels or spades.

The importance of proper proportioning of all mortar materials cannot be over-emphasised. Over-dosing with any material could cause problems such as colour variation, cracking, absorption and reduced bond strength.

Compressive strength of the mortar in structural masonry should be at least 12.5 MPa at 28 days and the masonry to mortar bond should be 200 KPa at 7 days.

Generally the compressive strength of mortar should be less than the strength of the units it is bonding together. The general strength requirement for concrete masonry units is 14 MPa.

The bond strength is of limited interest for structural masonry where reinforcing steel and grout provide the primary bonding. However in unreinforced veneer adequate bond strengths are necessary since it is this property that transfers loads of wind and earthquake to the wall ties, i.e. the bond strength is more important than compressive strength.

Typical values of bond strengths on concrete bricks are included in the Veneer Section (5.3).

The water retention test which relates to the loss of water from the wet mortar to the unit may be requested although a practical on-site test can be performed as follows:

(a) Mortar two bricks together to correct joint width, strike off mortar and wait two minutes.

(b) Lift the couplet by the top brick to a convenient height, usually waist height, turning the couplet over so that the lower brick becomes the top brick.

(c) The couplet is then lowered holding the new top brick.

(d) The couplet should not part during this test.

If after adjustment to the mortar mix or dampening of bricks, the test still cannot be performed, then the invoking of a full test to ASTM C91 may be required.

NZS 4210 Masonry construction: Materials and workmanship describes sample procedures for the compressive strength tests in Appendix 2A and for the bond test in Appendix 2B (referencing AS 3700). Reference should be made to that document for the details.

It is very important that having followed the procedures laid down for sampling and making specimens of mortar, that correct curing of these specimens is undertaken otherwise misleading results will be produced.

The standard curing regime for mortar strength specimens is shown in the *CCANZ Information Bulletin 51 Taking Test Cylinders on Site*.

Mixing

Mortar materials should be thoroughly mixed to an even consistency in a mechanical mixer. Paddle type mixers are preferable, although tilting-drum mixers may be used provided the blades have been adapted to produce the churning effect of a paddle mixer. Mixing time should be at least five minutes.

For small qualities of mortar, e.g. three or four buckets not exceeding 0.03 m, hand-mixing may be used provided full-mixing and blending is achieved.

Generally, mortar not used within 1½ hours should be discarded unless special provisions have been made.

Mortar may be retempered by the addition of water and thorough remixing providing that such mortar is used within the 1½ hour prescribed period from the initial mixing of the mortar, i.e. retempering does not extend the life of the mortar beyond 1½ hours.

Joint Types

Figure 1 (page 4) shows some of the tooling details commonly practiced. Some are not recommended for external application because of their poorer weatherproofing properties, but this will be of lesser significance where cavity protects the inner wall.

Because of the positive barrier to ingress of moisture any of the joint details illustrated in Figure 1 may be applied to external cavity or veneer walls without risk to inside finishes.

Of the details shown, types A, B and C are suitable for internal or external use. Raked and extruded joints should not be used externally except in cavity or veneer construction. The flush joint is recommended only for walls which received a later applied finish or coating.

The joints A, B and C, should only be tooled to a maximum depth of 6 mm after initial stiffening has occurred. The delaying of the tooling operation is vital if a tight weatherproof joint is to be produced in horizontal and, particularly, vertical joints.

Figure 2 (page 4) illustrates in an exaggerated way what is happening in the joint and how tooling gives an improved weathertightness. The whole matter of the tooling of external joints is of paramount importance and strict attention to delaying the operation after initial set of the mortar must be given.

Further details of tooling are discussed in Section 1.5 Blocklaying and under Section 2.2 Weather Resistance.

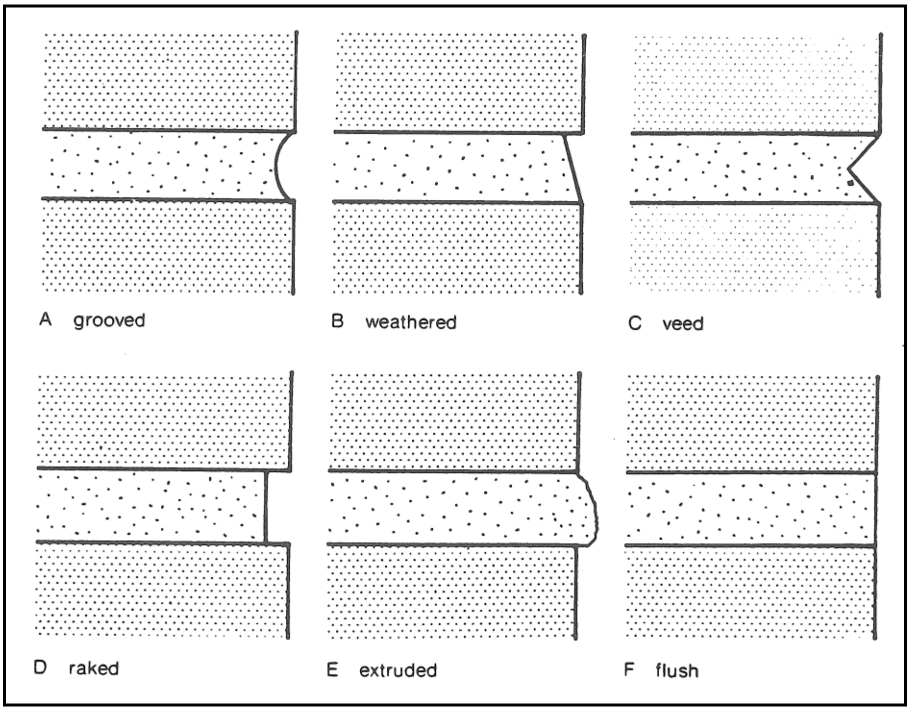


Figure 1: Joint Types

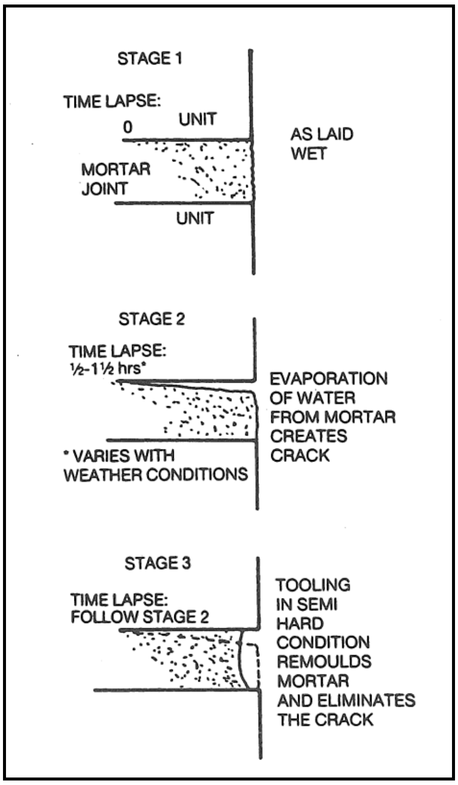


Figure 2: Tooling of Joints

1.5 Blocklaying

General

The purpose of this section is to give the specifier a background to the procedures used to construct a concrete masonry wall.

Design drawings and structural details should clearly establish the types of blocks to be used, their bonding patterns, reinforcing and grouting details and the relationship between the concrete masonry and other elements of the structure in hand. Examples of this are shown in Section 3 *Construction Details* of this manual.

Block units should be ordered well ahead of required delivery times to allow for manufacture and strength gain.

Construction/Specification details including tolerances are contained in the Construction Section. These details follow the requirements of *NZS 4210 Masonry Construction: Materials and Workmanship.*

Units

It is recommended practice that concrete masonry units be kept and laid dry in order to minimise shrinkage.

Dirt on a mortar bedding face will reduce bond and, if on an outer face, could mar the appearance of the unit.

Concrete masonry units should be kept clean and reasonably dry before laying by stacking and storing them on planks, pallets or similar supports to keep them clear of the ground. In particular climatic circumstances concrete blocks may be lightly dampened immediately before mortaring. Special units should be used at corners, windows, doors, bond beams, lintels, pilasters, etc., with little or no cutting. If cutting is necessary, such should be carried out with a concrete-cutting table saw or similar equipment to produce neat and true fashioning of the required unit.

Extreme Weather Conditions

As mentioned above it is preferable that concrete blocks be dry when laid. However, variations to recommended procedures for normal conditions may be adapted in extreme weather conditions.

When air temperatures are above 27°C or when there is a drying wind at lower temperatures the following practice is applicable.

1. Masonry units may be lightly dampened before laying, such as by brushing on water using a soft paint brush.
2. Mortar should be kept moist and should not be spread on the wall so far ahead of the units being placed as to cause a loss of plasticity.
3. Mortar should be prevented from drying so rapidly that it cannot cure properly; this may be done by applying a very light fog spray several times during the first 24 hours after laying or by other protective measures over the same period.
4. Grout should also be protected from too rapid drying.

When air temperatures are below 5°C the following practice is applicable.

1. Water used for mixing mortar should be heated.
2. Masonry should be protected for not less than 24 hours after laying by covers, blankets, heated enclosures, or the like to ensure that the mortar can gain strength without freezing or harmful effects from cold winds.
3. Frozen materials and materials containing ice should not be used.

Laying the First Course

The foundation face on which the blockwork is to be laid shall be clean and free of laitance, loose aggregate and any other material that would reduce the bonding if the mortar to the foundation. The bedding face of the foundation should be checked for horizontal and vertical alignment. Any variance that would cause the base mortar joint to be less than 7 mm or more than 20 mm thick should be corrected before the blocklaying is commenced. If a moisture barrier/coating is required it should be applied at this stage.

It is recommended that profile lines be placed in two directions to ensure precise location of the corners. After locating the corners it would be prudent to check the layout by using a rod marked at 400 mm modules or by stringing out the blocks for the first course without mortar (Figures 1 and 2). A chalked snap-line is sometimes used to mark the footing, thus helping to align the blocks accurately.

Except for partially grouted hollow unit masonry walls a mortar bed is then spread under the lines of the face shells and furrowed with a trowel to ensure plenty of mortar along the bottom edges of the face shells of the blocks for the first course (Figure 3).

For partially grouted walls mortar should be under the face shells and those cross webs that separate grouted cells from hollow cells (Figure 4).

The corner block should be laid first (Figure 5) and carefully positioned (Figure 6).

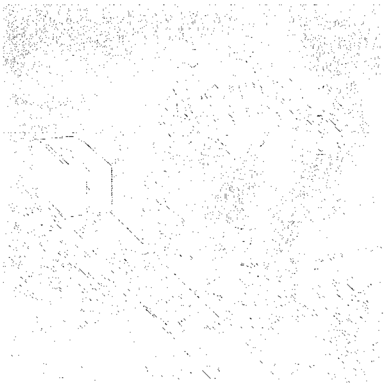


Figure 1
Layout with marked rod

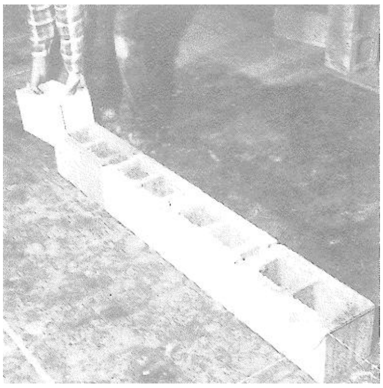


Figure 2
Check layout with dry blocks

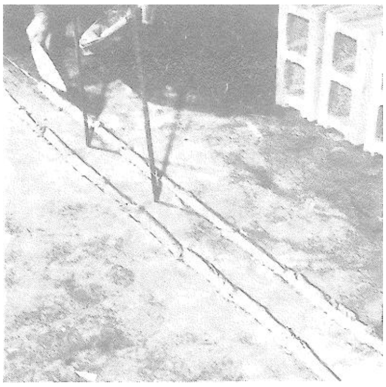


Figure 3
Mortar bed for first course

Figure 4
Web mortar for partial grouting

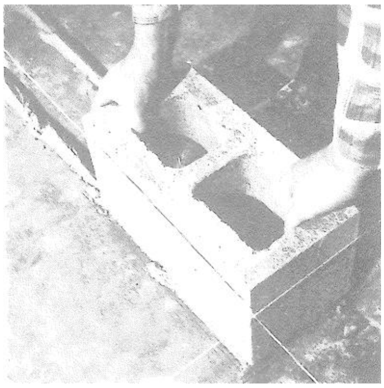


Figure 5
Lay corner block first

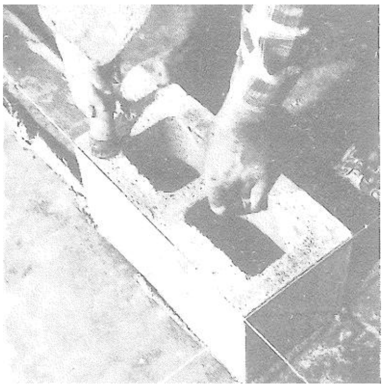


Figure 6
Position carefully

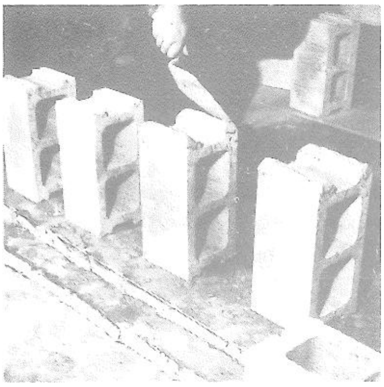


Figure 7
Mortar vertical face shells

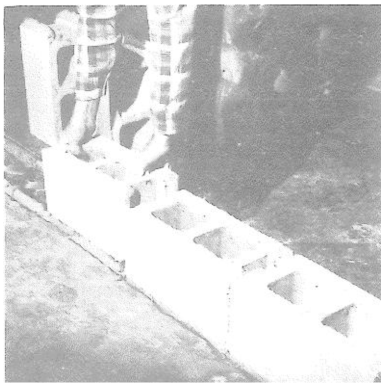


Figure 8
Place block against previous one

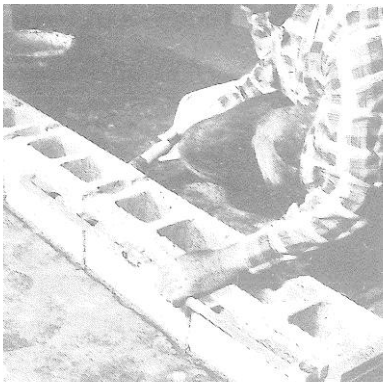


Figure 9
Check alignment

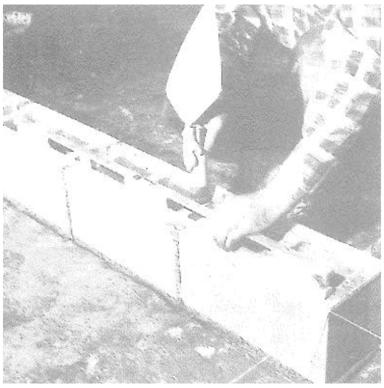


Figure 10
Check height and level

Figure 11
Check vertical alignment

Figure 12
Face shell mortar bedding

All blocks should be laid with the thicker end of the face shell up, as this provides a larger mortar bedding area. For vertical joints, only the ends of the face shells are buttered. It is essential that sufficient mortar is used in the vertical joints in order to achieve uniform thickness of mortar for the full height of the block. By placing several blocks on end, the blocklayer can apply mortar to the vertical face shells of three or four blocks on one operation (Figure 7). Each block is then brought over its final position and pushed downward into the mortar bed and against the previously laid block, thereby producing well-filled vertical mortar joints (Figure 8).

After three of four blocks have been laid, a mason's level is used as a straight edge to assure correct alignment of the blocks (Figure 9). Blocks are then carefully checked with the level and brought to proper height (Figure 10) and made plumb (Figure 11) by tapping with the trowel handle. The first course of concrete masonry should be laid with great care, making sure it is properly aligned, leveled and plumbed, as this will assist the blocklayer in laying succeeding courses in building a straight, true wall.

Laying Up the Corners

After the first course is laid, mortar is usually applied only to the horizontal face shells of the laid block except in partial fill when mortar on cross webs enclosing a cell to be filled is also required. This is called face-shell mortar bedding (Figure 12). Mortar for the vertical joints can be applied to the vertical face shells of the blocks yet to be placed.

The corners of the wall are built first, sometimes two or three courses higher than the centre of the wall. As each course is laid at the corner, it is checked with a level for alignment (Figure 13) and for being level and plumb (Figures 14 and 15).Each block is carefully checked with a level or straightedge to make certain that the faces of the blocks are all in the same plane. This precaution is necessary to ensure true, straight walls.

The use of a storey or course-pole, which is simply a board with markings 200 mm apart, provides an accurate method of finding the top of the masonry for each course (Figure 16). Mortar joints for concrete masonry should be10 mm thick, plus or minus 3 mm. Each course, in building the corners, is stepped back a half block and the blocklayer checks the horizontal spacing of the block by placing his level diagonally across the corners of the blocks (Figure 17).

Laying Blocks Between Corners

When filling in the wall between the corners, a mason's line is stretched from corner to corner for each course including the first and the top outside edge of each block is laid to this line (Figure 18). The manner of handling or gripping the block is important. Practice will determine the most practical way for each individual (Figure 19). Tipping the block slightly towards him, the blocklayer can see the upper edge of the course below, thus enabling him to place the lower edge of the block directly over the course below.

By rolling the block slightly to a vertical position and placing it against the adjacent block, it can be laid to the mason's line with minimum adjustment. All adjustments to final position must be made while the mortar is soft and plastic. Any adjustments made after the mortar has stiffened will break the mortar bond. By tapping lightly with the trowel handle, each block is levelled and aligned to the mason's line (Figure 20). The use of the mason's level between corners is limited to checking the face of each block to keep it lined up with the face of the wall.

To assure good bond, mortar should not be spread too far ahead of actual laying of the block or it will stiffen and lose its plasticity. As each block is laid, excess mortar extruding from the joints is cut off with the trowel (Figure 21) and is usually thrown into and reworked with the fresh mortar. If the work is progressing rapidly, some blocklayers apply the extruded mortar out from the joints to the vertical face shells of the block just laid (Figure 22). Should there be any delay long enough for the mortar to stiffen on the block, the mortar should be removed and reworked.

Dead mortar that has been picked up from the scaffold or from the floor should not be used. In some instances, a full mortar bed maybe specified on all concrete block construction. This requires mortar on the cross webs as well as on the face shells (Figure 23).

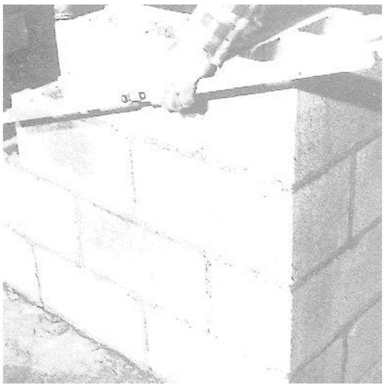


Figure 13
Check alignment

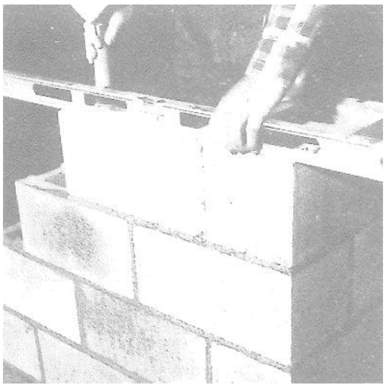


Figure 14
Check level

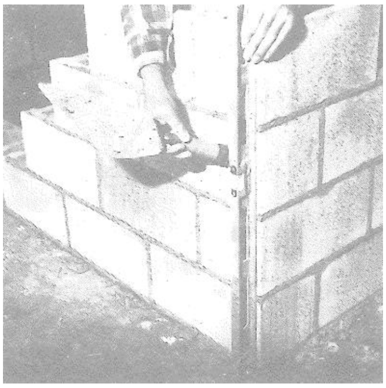


Figure 15
Check plumb

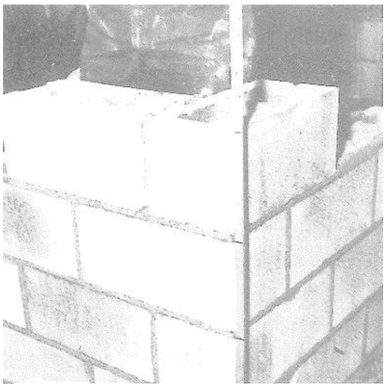


Figure 16
Check course heights

Figure 17
Check spacing

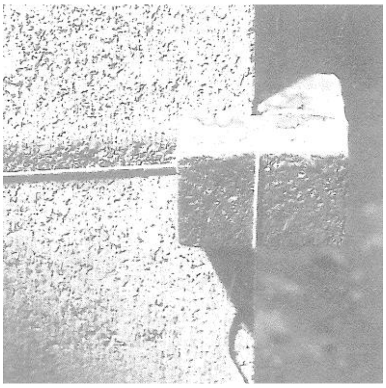


Figure 18
Mason's line at each course

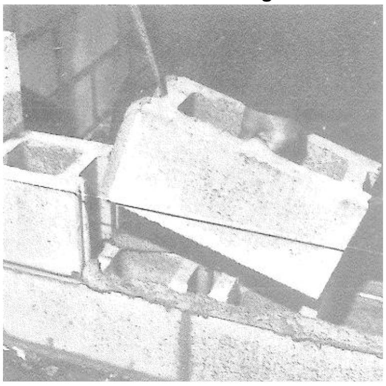


Figure 19
Lay to line

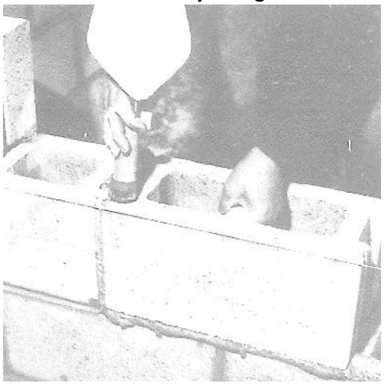


Figure 20
Tap into position

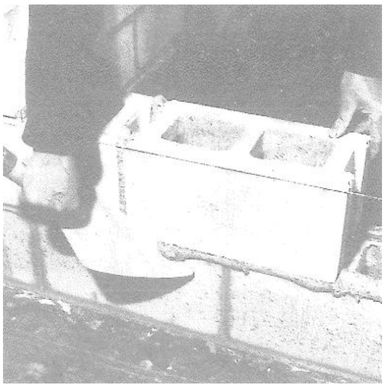


Figure 21
Remove excess mortar

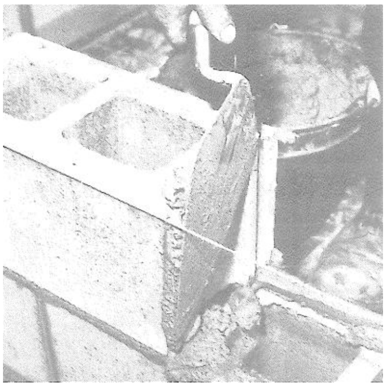


Figure 22
Vertical joint mortaring

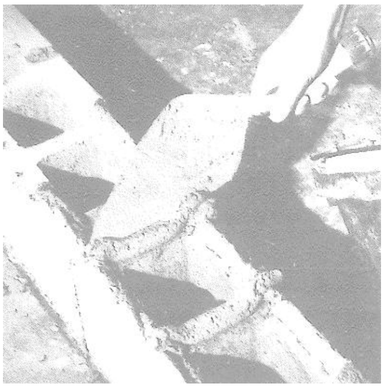


Figure 23
Full mortar bedding

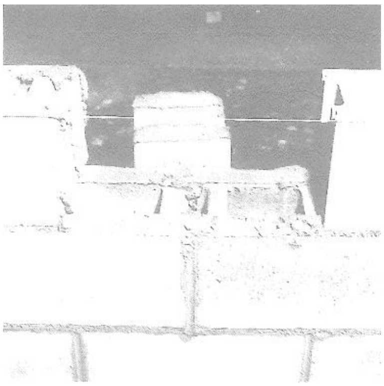


Figure 24
Mortar for closing block

Closure Block

When installing the closure block, the vertical mortar beds of one end of the closure block and the vertical mortar beds of the enclosure block of the other end are buttered with mortar (Figure 24). The closure block should be carefully lowered into place (Figure 25). If any of the mortar falls out, leaving an open joint, the joint should be packed with mortar using a trowel and jointing tool (Figure 26).

Tooling

Weathertight joints and neat appearance of concrete block walls are dependent on proper tooling. After a section of the wall has been laid and the mortar has become hard enough to resist the pressure of a thumb, the mortar joints should be tooled. The tooling operation compacts the mortar and forces it tightly against the masonry on each side of the joint. Proper tooling also produces joints of uniform appearance with sharp, clean lines. Unless otherwise specified, all joints should be tooled either concave or V-shaped.

The jointer for tooling horizontal joints should be upturned on both ends to prevent gouging the mortar. A suitable handle should be located approximately in the centre for ease of handling. Concave joints are preferable and are formed by a tool made from a 15 mm round bar (Figure 27). For V-shaped joints, a tool made from a 12 mm square bar is generally used. Tooling of the horizontal joints should be done first, followed by striking the vertical joints (Figure 28). Raked joints are formed by a rollered gouge (Figure 29). After the joints have been tooled any mortar burrs should be trimmed off flush with the face of the wall with a trowel (Figure 30) or removed by rubbing with a piece of concrete block of the same colour as those in the wall (Figure 31). Do not move or straighten the blocks in any manner once the mortar has stiffened or even partly stiffened. Final positioning of blocks must be done while the mortar is soft and plastic. Any attempt to move or shift the blocks after the mortar has stiffened will break the mortar bond (Figure 32) and allow the penetration of water. After the mortar has firmed a second tooling of joints will enhance the physical properties and appearance of the joints.

Cavity Walls

A cavity wall consists of two walls or wythes separated by a continuous air space and securely tied together with strong non-corroding metal ties embedded in the mortar joints. Ties are usually laced about 400 mm vertically and 600 mm horizontally. Weep holes are required at the bottom of cavity walls spaced at approximately 800 mm centres. The base of the cavity should be such as to drain away moisture which might collect in the cavity and build up against the inner wall. To keep the cavity clean, a 25 mm board of almost the same width as the cavity is laid across a level of wall ties to catch mortar droppings (Figure 33). The board can then be raised, cleaned and laid in the wall at the next level (Figure 34).

Expanded polystyrene is often used in sheet form within the cavity to improve insulation values.

In some cases it might not be practical to use a cleaning board, when it would be necessary to temporarily leave out every third or fourth block of the first course to facilitate cleaning of the cavity. These blocks would be mortared in after such cleaning.

Patching and Cleaning Block Walls

Any patching of the mortar joints or filling of holes left by nails or line pins should be done with fresh mortar.

Particular care should be taken to prevent smearing mortar into the surface of the block. Once hardened, embedded mortar smears can never be removed and they detract from the neat appearance of the finished wall. Paint cannot hide mortar smears. As concrete block walls should not be cleaned with an acid wash to remove mortar smears or mortar droppings, care must be taken to keep the wall surface clean during construction.

Any mortar droppings that stick to the block wall should be allowed to dry before removal with a trowel (Figure 35). The mortar may smear into the surface of the block if it is removed while soft. When dry and hard, most of the remaining mortar can be removed by rubbing with a small piece of block (Figure 36). Brushing the rubbed spots removes practically all of the mortar (Figure 37).

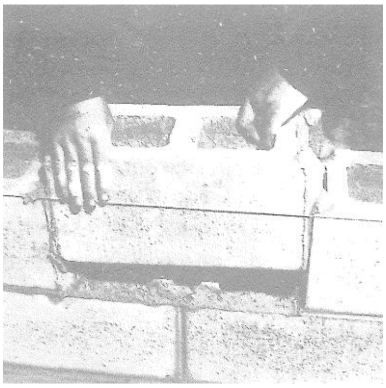


Figure 25
Laying closure block

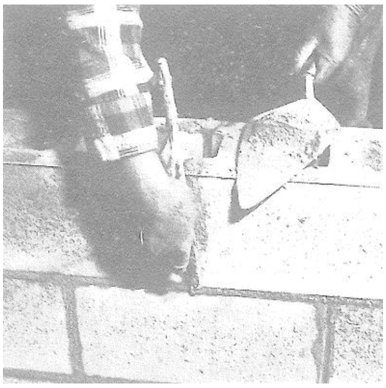


Figure 26
Packing vertical joint

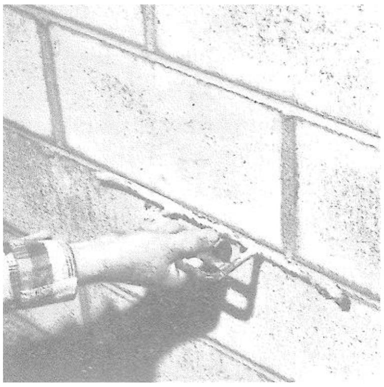


Figure 27
Jointing tool

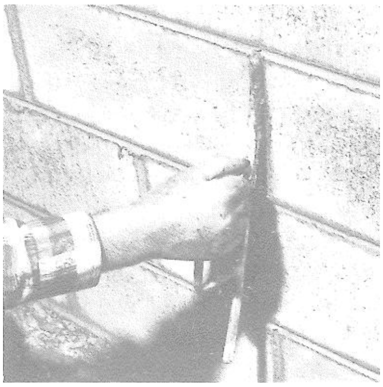


Figure 28
Tooled joints

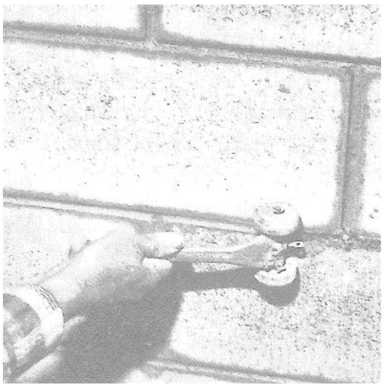


Figure 29
Raked joints

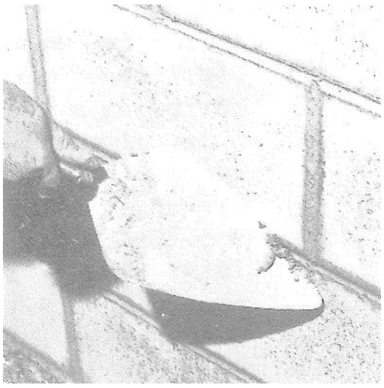


Figure 30
Trim off burrs

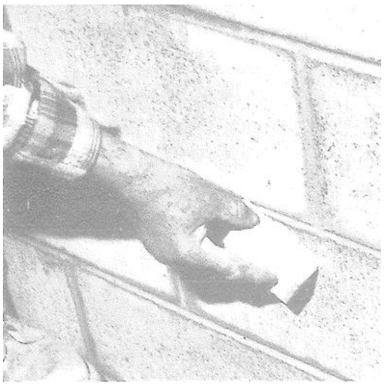


Figure 31
Rub with concrete block

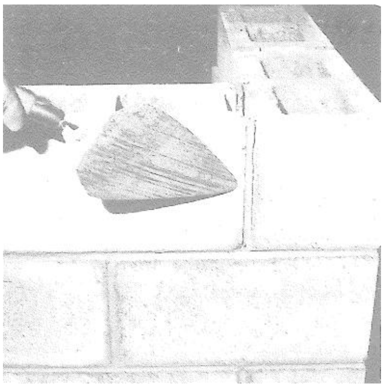


Figure 32
Moved blocks will crack joints

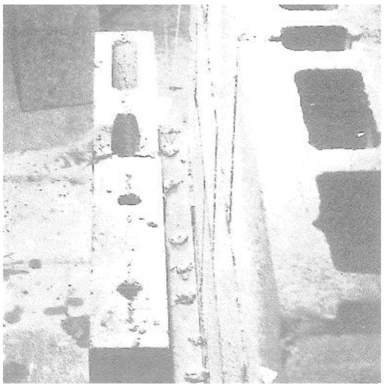


Figure 33
Cavity board

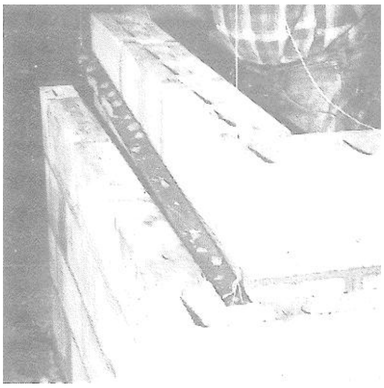


Figure 34
Board raised for cleaning

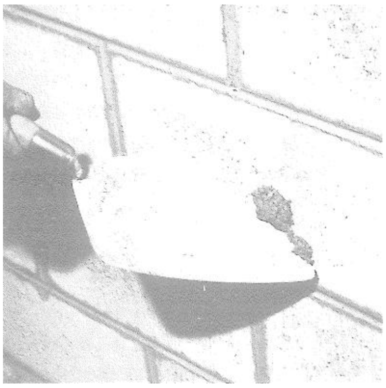


Figure 35
Removal of mortar droppings

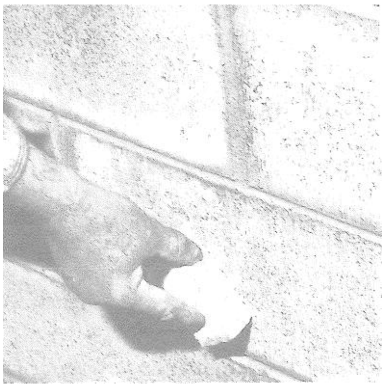


Figure 36
Rub over with block

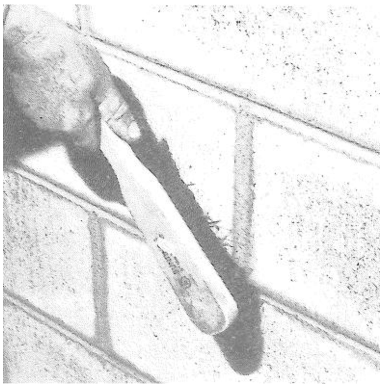


Figure 37
Brushing

Cutting Blocks

Concrete masonry units usually are available in proportionate sizes as well as full-length units. However, to fit special job conditions it is necessary to cut a block. For fast, neat cutting, masonry saws are often used.

Protection

In extreme weather or site conditions that would cause excessive dampening or drying of recently laid units, it is suggested that boards, building paper or tarpaulins are used to protect the top of the block walls at the end of the day's work.

Joint Types

These are described in Section 1.5.

Note: Details of reinforcement fixing and grouting, essential additional features of masonry blocklaying, are featured in the Construction Section.

1.6 Masonry Trades and Registration of Masons

New Zealand Masonry Trades Association (NZMTA)

The New Zealand Masonry Trades Association was formed in 2016 to provide a single amalgamated body of what had been 14 regional trade associations. Any company that regularly engages and undertakes masonry work may register as a member.

The purpose of the New Zealand Masonry Trades Association is to keep members up to date with legislation or trade information that affects them and to work with members to provide resources to assist in making compliance easier.

Contact details for NZMTA are:

New Zealand Masonry Trades Association
PO Box 50137
PORIRUA 5022
Email: info@nzmta.co.nz

Brick & Blocklayers' Federation New Zealand Inc. (BBFNZ)

The Federation was originally formed in 1966 and has undergone a series of changes leading to the current organisation.

The structure provides an umbrella for the New Zealand Masonry Trades Association, brick and block manufacturers and distributors, tool and safety gear providers and building companies that specify brick and block products.

The purpose of the Brick and Blocklayers Federation is to protect and promote the industry.

BBFNZ is governed by a seven person board which includes five elected representatives from the New Zealand Masonry Trades Association and two elected representatives from the manufacturing and distributing memberships. The BBFNZ also appoint a Technical Officer to assist in setting technical or workmanship standards.

Contact details for BBFNZ are:

Brick and Blocklayers Federation NZ
PO Box 50137
PORIRUA 5022
Email: info@bbfnz.co.nz
Website: www.bbfnz.co.nz

Registration of Masons

The New Zealand Masonry Trades Registration Board has been providing a registration of skills compliant with meeting requirements of the principal Standards *NZS 4210 Masonry Construction: Materials and Workmanship* and *NZS HB 4236 Masonry Veneer Wall Cladding*

However, this scheme was closed in July 2011 because the Department of Building and Housing have set up an alternative scheme under the title *Licensed Building Practitioner*.

Ever since the setting up of the Registration scheme it has been impossible to secure approvals via New Zealand Standards or the Building Code which would have seen a requirement for trade registration skills to have been a requirement of construction.

After 35 years, the DBH have finally recognized the need to have qualified tradespeople executing the work.

This recognition of trade skills and the compulsory need to become a Licensed Building Practitioner spells the end of the Registration scheme developed in 1974 by the Trade Federation, IPENZ, ACENZ, BOINZ, NZCMA, COWI and others.

See the new Licensing arrangements below.

Bricklaying and Blocklaying Licensed Building Practitioners

There are two areas of practice for the Bricklaying and Blocklaying license:

1. Brick/masonry veneer.
2. Structural masonry.

For Category 1, 2 and 3 buildings it is essential to hold a licence for the appropriate work.

The issuing of a licence is dependent upon a number of principal areas where competency must be shown:

(a) Demonstrate knowledge of the regulatory environment of the building industry.

(b) Demonstrate knowledge of current bricklaying and blocklaying practice.

(c) Plan masonry work.

(d) Carry out masonry work.

The more detailed competency requirements can be found at http://www.lbp.govt.nz/lbp/the-board/the-building-practitioners-board/the-lbp-rules.

Information on who has a current Licensed Practitioner license can be obtained by visiting https://lbp.ewr.govt.nz/publicregister/search.aspx.

1.7 Concrete Masonry Wall Units

Introduction

The specification for concrete masonry wall units was initially covered by NZS 595, superceded by NZS 3102:1983 and finally overtaken by a joint Australian/New Zealand Standard *AS/NZS 4455 Masonry units, pavers, flags and segmental retaining wall units*. This latest document covers all masonry products, i.e. not just concrete as was the case of the earlier New Zealand Standard.

AS/NZS 4455 is published in three parts:

1. AS/NZS 4455 Part 1 Masonry units, pavers, flags and segmental retaining wall units - Masonry units.
2. AS/NZS 4455 Part 2 Masonry units, pavers, flags and segmental retaining wall units - Pavers and flags.
3. AS/NZS 4455 Part 3 Masonry units, pavers, flags and segmental retaining wall units - Segmental retaining wall units.

There is a testing suite of Standards *AS/NZS 4456 Masonry units, segmental pavers and flags - Methods of test* which details test procedures covering the performance parameters set out in *NZS 4456*.

The tests listed are:

- **4456.1** Sampling for testing
- **4456.2** Assessment of mean and standard deviation
- **4456.3** Determining dimensions
- **4456.4** Determining compressive strength of masonry units
- **4456.5** Determining the breaking load of segmental pavers and flags
- **4456.6** Determining potential to effloresce
- **4456.7** Determining core percentage and material thickness
- **4456.8** Determining moisture content, dry density and ambient density
- **4456.9** Determining abrasion resistance
- **4456.10** Determining resistance to salt attack
- **4456.11** Determining coefficients of expansion
- **4456.12** Determining coefficients of contraction
- **4456.13** Determining pitting due to lime particles
- **4456.14** Determining water absorption properties
- **4456.15** Determining lateral modulus of rupture
- **4456.16** Determining permeability to water
- **4456.17** Determining initial rate of absorption (suction)
- **4456.18** Determining tensile strength of masonry units and segmental pavers
- **4456.19** Determination of bow

Summary

The following are the principal requirements of *AS/NZS 4455 Part 1 Masonry Units* relating to concrete wall units. For full details, consultation of the full document is required.

- **Dimensional deviations**

  Test method DW4 *AS/NZS 4456.3:*

  Standard deviation of not more than 2 mm and a difference between the mean and work size of not more than 3 mm.

- **Unconfined Compression Strength**

  Test Method *AS/NZS 4456.4:*

  The minimum strength required for concrete masonry is specified by *NZS 4210* as:

  1. 12.5 MPa for structural masonry.
  2. 10 MPa for non-structural external units.

- **Integrity**

  There are provisions to ensure the thickness and shape of units are suitable for transportation and handling.

- **Durability**

  The minimum requirement for durability for concrete masonry specified by *NZS 4210* is 10 MPa.

  Provisions for freeze thaw or salt attack resistance may be required for special locations.

- **Demonstration of Compliance for Strength**

  The standard does set compliance requirements:

  1. Single Lot:

     Mean value greater than specified characteristic value + 1.20.

     S (standard deviation) is based on 30 specimen's results or taken as 0.15 times mean value.

  2. Lots taken in continuous manufacture:

     Central level shall be greater than specified characteristic value $\pm 1.65\,S - \frac{2S}{\sqrt{n}}$.

  Each sample shall be at least five samples.

- **Requirements of New Zealand Concrete Masonry Association**

  The Associations requires its members to produce masonry to meet the requirements of nationally set Standards, i.e. *NZS 4210* and *AS/NZS 4455* using the testing methods defined in *AS/NZS 4456*.

  Specification outlines for *AS/NZS 4455* Parts 2 and 3 are contained in Section 7 of the Manual.

2.1 Fire Resistance

Introduction

Concrete masonry is generally considered to have good fire resistance since it is non-combustible (i.e. it does not burn), absorbs heat only slowly, and does not give off toxic fumes or smoke. As it is a poor conductor of heat and has a high heat capacity, concrete is used to protect other construction materials such as steel and timber from fire. In addition, it is this slow heat absorption which enables concrete to act as an effective fire shield to protect adjacent spaces and contents, as well as itself from internal fire damage.

These inherent properties, combined with the appropriate design of structural elements, ensure concrete performs well in fire.

However, there are some issues surrounding concrete's performance in fire which require careful consideration. This bulletin outlines these issues, while providing general guidance to designers and specifiers on the design of concrete structures against fire.

Principles of Fire Protection

The first and most important objective in fire protection is to safeguard the lives of any people who are in the structure which is on fire, and enable them to exit the building quickly and safely. Secondly, the structure must be designed to allow enough time for fire fighters to safely carry out any search and rescue operations, along with firefighting operations. Thirdly, there are requirements to protect other property under the New Zealand Building Code (NZBC), these include preventing the fire from spreading as well as preventing hazardous materials at the fire site entering waterways.

Concrete masonry is commonly used to provide stable firecells in large industrial or multi-storey buildings as a means to contain a fire and prevent it spreading to the whole building. This is also called fire separation or compartmentation. Concrete walls reduce the spread of fire horizontally and concrete floors vertically. Concrete provides the opportunity to install safe separating structures in a reliable and economical way.

Fire performance is the ability of a particular structural element (as opposed to any particular building material) to fulfil its designed function for a period of time in the event of a fire. The three functions of Stability (R), Integrity (E) and Insulation (I) are universally recognised to define fire protection. Time periods (fire ratings) are attributed to each of these functions to designate the level of fire performance.

The overall Fire Resisting Rating of an element is termed FRR, thus a FRR of 90/90/90 requires a 90 minute rating for each of stability, integrity, and insulation.

Stability

Stability is the load bearing capacity provided by the primary elements within a firecell and includes elements which are part of the structural frame as well as those providing support to other fire rated elements.

The Stability fire rating (R) is based upon the time an element can withstand a standard fire test and retain its loadbearing capacity while allowing for a level of superimposed load.

Integrity

Integrity is the flame arresting separation typically provided by secondary elements, e.g. internal walls, to protect people and goods from flames, harmful smoke and hot gases. Primary elements, along with secondary elements, are also rated for integrity.

The time during which an element's fire separation capability is maintained is determined by the tightness of joints to limit smoke and gas penetration.

Most casualties suffocate in a fire because of the smoke rather than burn in the flames. Concrete does not develop smoke during fire.

Insulation

Insulation is the heat shielding capability provided by either primary or secondary elements. It is applied to fire separations where the transmission of heat may endanger occupants on the non-exposed side or cause fire to spread to other fire compartments. The fire rating is the time defined by a maximum permitted rise of temperature on the non-exposed side.

Active versus Passive Fire Protection

Concrete's and concrete masonry's inherent fire resistance provides a robust passive protection system that usually requires no additional fireproof linings or coatings.

Fire protection for lightweight construction often relies on active protection systems such as sprinklers. However, the robustness of sprinkler systems following an earthquake has been questioned in light of the vulnerability of reservoir water supply and the significant risk of a post-earthquake fire. Many reservoirs are fitted with auto-shut valves for earthquake events.

The integrity of fire linings following an earthquake has also been brought into question by BRANZ research, particularly where dislodged plaster filling at joints, resulting from seismic shaking, compromised the integrity by 40%. In addition, it has also been found that poor workmanship in retrofitting of structures after installation of services through lightweight fire rated elements has compromised subsequent levels of fire protection. Furthermore, fire linings/paint could be damaged by impact e.g. hit by the edge of carried through items or furniture.

It is fashionable to pour scorn on the “old fashioned” approach for fire containment by using a masonry wall between occupancies. Reliance must be placed in "up market active systems" if one is to remain in fashion.

The Great Fire of London, 1666, was one event that led to the use of masonry walls between occupancies. In 1940 the fire-bombing of cities such as London put the “masonry system” to the ultimate test, i.e., containment of fire since for the most part little water was available.

New Zealand in common with other countries has embarked on a series of fire protection code changes. By and large, previous codes have been claimed to be ultra conservative and too prescriptive with requirements demanding the use of non-combustible materials such as concrete or concrete masonry.

Associated with changes in fire resistance ratings are changing requirements for using sprinkler systems. While few would question the efficiency of fire control when these active systems work, examples can be quoted of their failure to activate. Indeed, the question of activation after, for example, an earthquake when water pressure may be lost could be critical.

It is these circumstances that PASSIVE control by compartmentisation is paramount using non-combustible materials which have a proven in-service track record. While the passive system does not put the fire out it certainly can prevent spread. It is argued that compartmentisation passive rules should not be relaxed when active systems such as sprinklers are included. Sprinklers may well reduce contents fire loss for the owner and are therefore desirable from an insurance point of view. If they do not work then it is a statistical risk for insurers.

Concrete Performance in Fire

At temperatures of 150°C upwards there is some loss of water from the silicate hydrates in concrete, while temperatures above 300°C result in the loss of bound water, and in turn strength.

While concrete may undergo strength loss at temperatures 300°C and above, the main losses are not apparent until above 500°C. Even though flame temperatures are up to double 500°C, the temperature of the internal concrete remains relatively low as a result of concrete’s slow heat absorption. Therefore, only intense fires of long duration may cause any weakening of concrete structures.

Cement type can have some influence on strength loss. Cements with fly ash and ground granulated blast furnace slag have lower quantities of free calcium hydroxide which can give reduced hydration loss on heating, and consequently lower strength loss.

The fire rating of a concrete is also influenced by the aggregate type. This results partly from the coefficient of thermal expansion between the aggregate and the cement paste being different, particularly at higher temperatures.

The thermal conductivity of concrete depends on the nature of the aggregate, porosity and moisture content. As water is driven from the concrete in a fire, the conductivity of the ‘dry’ concrete is more relevant.

Lightweight aggregate concretes in particular, have very good fire performance in ‘dry’ building fires because they have a thermal expansion closer to cement paste. They also have good aggregate bond and high aggregate temperature stability. Limestone has an additional advantage in that it breaks down at temperatures over 660°C giving off carbon dioxide which provides a blanketing effect against heat penetration.

The effect of aggregate type on fire resistance is demonstrated in the table below. Based on minimum effective slab and wall thicknesses the table shows a range of insulation fire ratings (I) for three different aggregate types.

Fire Resistance Rating (minutes)	Effective Thickness (mm) for Different Aggregate Types		
	Type A Aggregate	Type B Aggregate	Type C Aggregate
30	50	45	40
60	75	70	55
90	95	90	70
120	110	105	80
180	140	135	105
240	165	160	120

Note: Aggregate types:

A - quartz, greywacke, basalt and all others not listed

B - dacite, phonolite, andesite, rhyolite, limestone

C - pumice and selected lightweight aggregates

Source: NZS 4230:2004

The interpretation for masonry walls is as follows:

Fire Resistance Insulated Rating Minutes	Solid or Solid Filled Masonry				
	Wall Thickness (mm)				
	70	90	140	190	240
30	OK	OK	OK	OK	OK
45	OK	OK	OK	OK	OK
60		OK	OK	OK	OK
90		*	OK	OK	OK
120			OK	OK	OK
180			OK	OK	OK
240				OK	OK

* Using a B and/or C Aggregate 90 mm would meet the 90 minute criteria.

Fire Resistance Insulated Rating Minutes	Partial Filled Masonry			
	Wall Thickness*1			
	90	140	190	240
30	OK	OK	OK	OK
45	OK	OK	OK	OK
60	OK	OK	OK	OK
90	OK*2	OK*2	OK*2	OK*2
120		OK*2	OK*2	OK*2
180			OK*2	OK*2
240				

*1 Based on two face shell thicknesses of 35 mm.

*2 Based on masonry units being light weight.

Equivalent thickness is often quoted:

90 mm	Equivalent thickness	65.8 mm
140 mm	Equivalent thickness	85.0 mm
190 mm	Equivalent thickness	101.8 mm

The concrete cover to reinforcement to ensure protection from fire is shown below.

Fire Resistance Rating Minutes	Cover (mm)
30	20
45	20
60	20
90	35
120	40
180	45
240	50

Most masonry wall reinforcement is in the centre of the wall and hence well exceeds the minimum values quoted. However, care is needed when dealing with columns, pilasters and beams.

Spalling of the surface concrete is a phenomenon which may occur in certain circumstances where the surface concrete breaks away at high temperatures. The problem is not a significant feature for concrete masonry.

Reinforcing steel loses strength at elevated temperatures – there is a 15% loss from 350°C up to 50% at around 600°C, and an 80% loss at 750°C. However, concrete or grout's low thermal conductivity protects reinforcing steel from significant temperature gain provided it has sufficient cover. Thus, the specification of minimum cover to reinforcement has to meet both durability and fire performance requirements.

Basis of Fire Design

Standard test methods are used to determine the fire performance of materials or structural elements. These tests may either be at a small scale with a component of a building in an oven or furnace, or at full scale in a mock-up of a fully assembled building subjected to a fire regime.

Standard fire time temperature curves have evolved to represent typical fires experienced in practice. The curves for fires representing three scenarios for building fires, hydrocarbon fires and tunnel fires are shown in **Figure 1.** These curves are different for the different scenarios. For instance, the temperature of a building fire rises much more slowly and peaks at a lower temperature than a hydrocarbon fire from burning vehicles as there is less combustible material present. Tunnel fires have a significantly higher peak temperature owing to the confinement of the fire. The hydrocarbon load in a tunnel can be considerable, which includes not only vehicles but also the bituminous road surface. As a result, the use of concrete roads in new tunnels is now recommended. **Figure 2** represents a standard furnace time temperature curve for a building fire taken from *AS 1530.4-05 2005 Methods for Fire Tests on Building Materials, Components and Structures (Part 4: Fire-Resistance Tests of Elements of Building Construction).*

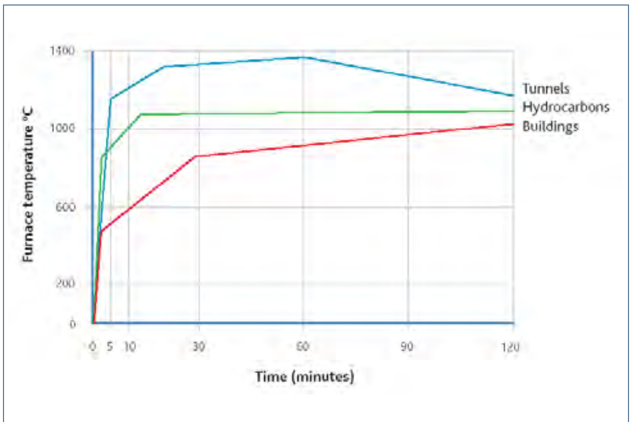


Figure 1: *Standard fire curves for three scenarios – tunnels, hydrocarbons and buildings*

Source: Concrete and fire safety. UK Concrete Centre (2008)

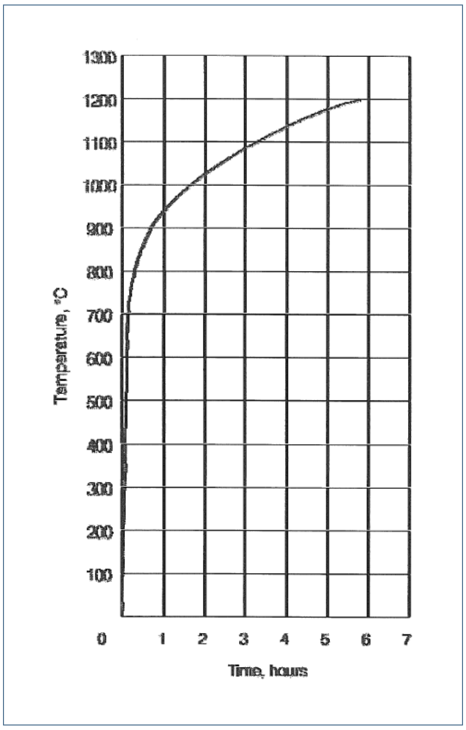


Figure 2: *Standard furnace temperature-time curve*

Source: AS 1530.4-05 2005

The *NZS 3101:2006 Concrete Structures Standard* and the *NZS 4230:2004 Design of Reinforced Concrete Masonry Structures* cite *AS 1530.4-05 2005* as the compliance code for carrying out fire tests on building components or assemblies. This code gives a standard time-temperature curve. This will differ from the time temperature relationship in an actual fire as controlled by a number of factors – fuel, fuel geometry, ventilation and restraint provided on members from adjacent areas of the building which are unaffected by the fire.

The increasing sophistication of computer modelling techniques has enabled data from standard fire tests on building components to be interpolated into the predicted fire behaviour of building assemblies and whole buildings. This has had the effect of reducing the need for comprehensive whole assembly fire tests of buildings.

NZBC Fire Safety Requirements

NZBC Compliance Document Clause C Protection from Fire is structured into seven acceptable solutions, C/AS1 to C/AS7 plus two verification methods. The acceptable solutions (AS) represent seven different risk groups for buildings with a maximum number of storeys of 20. All AS are 'deemed to comply' solutions, other specific design solutions can be submitted but require verification.

Each AS consists of seven parts, these are:

- Part 1: General - Scope, occupant load, etc.
- Part 2: Firecells, fire safety systems and fire resistance ratings.
- Part 3: Means of escape - Safeguard people from illness or injury whilst escaping and facilitate fire rescue operations.
- Part 4: Control of internal fire and smoke spread.
- Part 5: Control of external fire spread.
- Part 6: Fire-fighting.
- Part 7: Prevention of fire occurring.

There are two categories of fire ratings: Life rating and property rating:

1. Life rating applies to fire rating requirements in Part 3: Means of escape and Part 4: Control of internal fire and smoke spread.
2. Property rating applies to fire rating requirements in Part 5: Control of external fire spread.

The Fire resistance rating of each AS or risk group may vary and is described in Part 2.3.

The most common rating is 60 minutes. With automatic fire sprinklers and smoke detectors fitted, the rating can be halved in some cases. However, as mentioned earlier, sprinklers may not be fed when water is most needed in an earthquake event as auto-shut valves stop the water flow from the reservoirs.

Further fire safety systems are shown with table 2.1 in Part 2 of each AS.

Most fire rated boundary walls utilise concrete for its superior toughness, good fire rating and being structurally sound, as well as having good Spread of Flame Index (SFI) and Smoke Developed Index (SDI) properties according to AS/NZS 1530.3, which are satisfactory for the worst case scenario.

High-rise apartments are generally concrete structures and the fire escapes (deemed safe places) are concrete enclosed, usually bare, to comply with the need to have no combustibles within the space.

In the design of a multi-level building, each level is a separate firecell. Hence penetrations through the floor must be protected against the passage of fire by the installation of a suitable fire stopping mechanism. The most common need for floor penetrations is to distribute services up the building. Services on each level do not require any further fire protection unless there is a special case to warrant it. Another alternative is to provide a fire rated shaft running up the building with all services in it. This requires the services taken from the shaft onto each level to be protected with a suitable fire stopping mechanism.

Structural Fire Design

Chapter 5 of NZS 4230:2004 sets out the design requirements for fire resistance of structural concrete masonry.

In the scope which sets out this chapter, it is noted that the basis of the design relates to AS 3600 which has been adapted to reflect the properties of New Zealand masonry. The reason for basing on AS 3600 relates to the fact that all structural masonry is reinforced and hence fits more closely to reinforced concrete than unreinforced masonry documents.

The background for AS 3600 relates to the Australian fire document AS 1530, Part 4. The chapter deals with the following design:

- Clause 5.4: Fire resistance of walls

  In particular axial load is interrelated to the slenderness ratio of the wall and its end fixity arrangements.

- Cause 5.5: Fire resistance for beams
- Clause 5.6: Fire resistance for columns

  In particular, columns can be exposed to fire on all sides or be partially exposed as sides become protected by a wall for example.

Fire resistance rating can also be calculated by reference to Clause 5.7 and using fire test information generated by AS 1530, Part 4 or BS 474, Parts 20-22.

Structural Fire Engineering

The specialist discipline of structural fire engineering involves the knowledge of fire load, fire behaviour, heat transfer and the structural response of a proposed building structure.

The application of structural fire engineering allows the use of a performance based approach using advanced calculation methods which lead to more economical, robust and innovative concrete buildings.

Analytical computer based modelling of whole buildings utilise the interaction between building elements which can result in structures being safer than calculated in design based on individual structural elements. For example, when a concrete slab expands under high temperatures to push outwards against its supports, a mechanical arching effect takes place in the slab. The compression generated in the bottom of the slab can greatly increase the load capacity.

External Walls

Section 4.8 of NZS 3101: 2006 gives particular requirements to prevent collapsing external walls outwards in a fire. The loadings code requires free standing external walls to be designed to resist a face load of 0.5 kPa in the after fire condition. When a fire occurs inside a building, the interior face of the wall heats up and expands while the exterior face remains relatively cool. Coincidentally the eccentricity of axial load on the wall causes additional deflections due to the P-delta.

The wall has the potential to collapse when the actions on the wall due to thermal bowing and P-delta effect can lead to the wall's capacity being exceeded effect – see **Figure 3**. Such a collapse into adjoining property risks placing fire crews or neighbours in danger. A base-cantilever-resisting mechanism is usually required to prevent collapse.

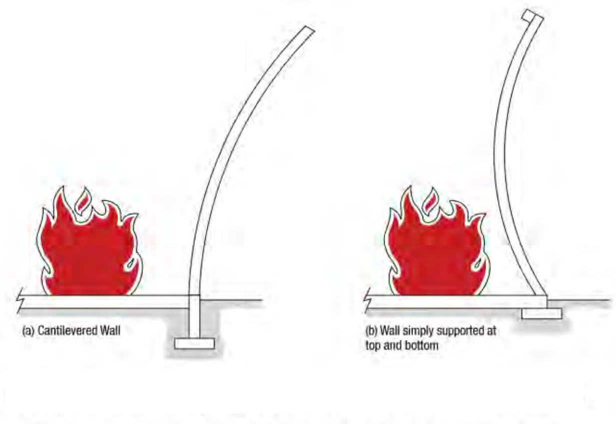


Figure 3: *Deformation profile caused by heating one side of the wall*

A wall connected to a very weak or flexible roof structure will need to be designed with a cantilever base connection. The design of the unprotected mild steel connections needs to be based on 30% of the yield strength of the exposed steel in ambient conditions.

Components made from other types of steel shall use mechanical properties of the steel at 680°C. Details for FRR ratings and fixing of proprietary inserts are given in the standard. Adhesive (glued) anchors are have been found to behave poorly at elevated temperatures and need to be protected from fire.

Recent research has been carried out on slender panels by BRANZ and the University of Auckland owing to a concern of the high slenderness ratios of on-site and off-site precast panels being used in practice. A range of slenderness ratios from 30 to 75 were investigated. NZS 3101:2006 places a slenderness ratio limit of 75. Other maximum slenderness ratios have been proposed. By comparison NS 4230 sets a slenderness ratio of 50.

Insurance and Fire Damage

A recent, independent European investigation on the cost of fire damage in relation to the building material from which houses are constructed used statistics from the Insurance Association in Sweden (Forsakrings Forbundet). The study was on large fires in multi-storey buildings in which the value of the structure insured exceeded €150k. The sample set was 125 fires which occurred between 1995 and 2004. The results showed that:

- The average insurance pay-out per fire and per apartment in concrete/masonry houses is around one fifth that of fires involving other materials (approximately €10,000 compared with €50,000)
- A major fire is less than one tenth as likely to develop in a concrete/masonry house than one built in other materials
- Of the concrete houses that burned only nine per cent needed to be demolished whereas 50 per cent of houses built from other materials had to be demolished

The time taken to repair a building after a fire is important in terms of downtime for commercial businesses. Concrete and concrete masonry buildings are generally easier and quicker to repair.

In buildings subject to arson attack such as schools, the loss of contents and repair time is also critical. These losses can be significantly less in concrete and concrete masonry buildings.

In the UK there have been a disproportionate number of fires in timber structures under construction. The fire load of a timber building being constructed is significant, and cannot be contained effectively until compartmentation is completed. In a concrete and concrete masonry structure the fire load during construction is significantly less.

Conclusions

The excellent performance of concrete and concrete masonry structures in fire is widely accepted. The role of concrete in providing passive fire protection gives a significant advantage over steel and timber structures and provides a more robust solution to fire protection. The behaviour of concrete in fire is well understood, and is substantiated by a wealth of fire testing research data.

Concrete design standards have historically been based on prescriptive data generated from fire tests. Eurocode 2 outlines an alternative approach based on computer simulation and performance based fire-safety engineering. This allows a greater degree of flexibility in terms of sizing concrete elements for fire safety and will lead to the more efficient design of concrete and concrete masonry structures.

Sources and Further Reading

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2.2 Weather Resistance and Surface Coatings

Introduction

It is important to be aware that from August 2011 Clause E2 (External Moisture) of the New Zealand Building Code included an Acceptable Solution (E2/AS3) for weathertight concrete and concrete masonry construction that references the Cement & Concrete Association of New Zealand's – *CP 01, Code of Practice for Weathertight Concrete and Concrete Masonry Construction* in the following referred to as 'CCANZ CP 01'

This Section (2.2) of the *Concrete Masonry Manual* refers to the many relevant details on weathertight concrete and concrete masonry construction contained in 'CCANZ CP 01'.

In choosing any construction system for the external walls of a structure, the designer and builder must exercise great care not only in the selection of materials but also in the supervision of workmanship. Even the most perfect material will not perform to its potential if the design detailing and workmanship are inadequate.

The segmental nature of concrete masonry requires sound masonry units, proper mortar, good detailing and competent workmanship. This is particularly so in single skin walls which are more vulnerable to moisture transmission than are cavity walls in which a drained air space provides an excellent barrier against the passage of water. The following matters have been observed to play a significant role:

1. Design:

   - Inadequate eaves overhang
   - Poor detailing of reveals and sills
   - Lack of adequate flashing details
   - Lack of control joints
   - Failure to specify tooled joints.

2. Materials and Workmanship:

   - Insufficient mortar in joints
   - Failure to recompact joints (tooling) after initial hardening
   - Bond and shrinkage failures between block and mortar
   - Moisture penetration of blocks
   - Failure to drain unfilled cells
   - Failure to reconsolidate grout in filled cells.

Each of the matters listed can readily be resolved by attention to design/specifications and by the use of qualified tradesmen. The tradesmen should be members of the local Blocklayers' Association as well as being a Licensed Building Practitioner in Brick and Blocklaying. Matters of dispute on workmanship can be investigated by an independent and experienced blocklayer should a need ever arise.

Members of Masonry Tradesmen's Associations are well aware of the need to display - by example - competent workmanship for works under construction.

It is worth reiterating that design features such as ample roof overhangs contribute significantly to avoiding incidents of leakage. In addition, while surface coatings are a method of providing a significant degree of weather resistance, they do not necessarily make up for all the design and construction shortcomings.

Specifiers and builders should also be aware of the influence of environment on performance. One type of masonry unit, for example, may be completely satisfactory in its normal applications but exhibit serious moisture problems when employed in a location of greater rainfall and perhaps more wind exposure. This distinction will be emphasised by factors such as lack of roof overhangs in wetter climates.

A major requirement is satisfactory performance, and that may call for a suitable surface coating. If the selection of the masonry unit is based on economics, the comparison of costs should include the price delivered on site, and the cost of any subsequent treatment to ensure satisfactory performance.

If the selection is based on appearance, it must be realised that the apparent advantage of a particular block may be of no consequence should performance demand the application of a coating.

In every case of residential construction, the wall and its external treatment have to comply with the NZBC, either clause E2/AS3 or with an alternative solution.

Mortar

Weathertight joints are dependent on the joint space being completely filled with good quality mortar, correctly tooled to shape and seal its outer surface and to compress it against the face of the masonry unit. Some time after laying the masonry, the mortar begins to harden. As it does so it will tend to shrink slightly and pull away from the lips of the joint. The firm and proper use of the jointing tool restores intimate contact of the mortar and the masonry units, and also seals any cracks which may have been left when the wall was laid.

The mortar is ready for tooling when it can still be impressed by firm thumb pressure, but without adhering to the thumb.

Proper tooling contributes significantly to the weatherproofing of the wall, but it does require that the joints are already well filled with mortar, particularly the perpends or vertical joints. Mortar may have fallen from these joints as the units were laid, or mortar cracks may have formed during the alignment of the blocks. Additional mortar should be forced into any sparse joints so as to ensure complete filling. This should be done as soon as possible after laying and aligning the masonry units, and definitely before the mortar within the joint has lost its plasticity. Reference should be made to illustration No. 28 in the Blocklaying section of the manual.

Grooved or veed tooled joints, as illustrated in the chapter dealing with Mortar and Mortar Joints, are recommended as the most effective for weatherproofing the wall. Weathered joints are also suitable but the relationship of vertical and horizontal joints is not so easily resolved.

Further details of mortar mixes, joint types, tools and tooling are given in those sections of the manual dealing with Blocklaying, and Mortar and Mortar Joints.

Control Joints

The weather-resistance of concrete masonry is enhanced by controlling the positions of cracks arising from shrinkage or structural movement, and the proper caulking of those cracks, or control joints as they are more correctly called.

The exact position and spacing of control joints will vary from job to job but should not be larger than 6 m in horizontal direction when using non-specific design standards. Information on methods of forming and sealing control joints is given in the general chapter of the Construction Details section of this manual and also in the section on Veneer Walls.

Further information is given in 'CCANZ CP 01', Code of practice for Weathertight Concrete and Concrete Masonry Construction.

Grouting

Structural use of concrete masonry requires a degree of reinforcement and grout filling to an extent that is resulting in an increasing number of fully grouted walls being built. It is very important not to assume that solid filling automatically must produce a weathertight wall. In particular, reconsolidation of grout some time after filling is a vital ingredient to ensure a weathertight solid filled wall. The details of grouting methods are described in the construction section, including the preferred use of gas release type of expanding admixture which automatically compensates for initial volume losses in the grout.

Flashings

The development of chemical sealants and mastics has allowed a reduction in the use of flashings, but it is still prudent design detailing for metal or plastic flashings to be used at parapets, gutters, roof intersections and larger projections.

Materials used for flashings must be moisture-proof, resistant to atmospheric corrosion, resistant to alkali that might be present in the blocks or mortar, resistant to casual puncture or abrasion, easily formed to the required shape and capable of maintaining that shape.

Where a coating is to be applied to the wall, compatibility of flashing material and coating should be confirmed.

Where flashings occur on a building facade, their lower edges should be bent outwards ***at least 20 mm*** and then downwards so as to cause water to drip clear of the facade.

Offsets

It is recommended that concrete masonry sill courses be of the projecting type, in order that water running off same will also drip clear of the facade. The breaking down of rainwater flow in its passage down or across a facade will assist in control of weather staining. The positioning of sill courses and offsets should be considered for that reason at the design stage.

Offsets of approximately 15 mm where concrete masonry meets, say, concrete beams or columns will also assist rainwater flow control and reduce weather staining. Such offsets also provide a rebate against which sealant could be applied by pressure gun, if required.

Table 1: Types of Concrete Masonry Units

	Type 1	Type 2	Type 3	Type 4
Principal Aggregate	Limestone	Scoria	Greywacke	Pumice
Dry Density Kg/m^3	1,794	1,711	2,195	1,701
Dry Density Kg/m^3	208	225	128	166
Permeability Factor (as per AS/NZS 4455.1:2008)	363	24	4	19

Weepholes

Weepholes should be formed in the vertical mortar joints of the outer leaf or skin of a cavity wall immediately above the supporting beam or slab. Refer also to the Veneer walls and Construction Details sections of this manual. Weep holes should also be formed in the bed joints connecting to non-filled masonry cells in partial fill construction.

Masonry Units with Surface Coatings

Although it is not always necessary to provide a coating to concrete masonry, in most cases walls are given some treatment either for decoration or for weatherproofing. Decoration may be desired to provide colour or texture. In some cases, it may be intended to simplify maintenance or perhaps offer protection against the disfigurement of graffiti. Occasionally, specialised coatings will be applied for hygiene reasons, or to offer protection against chemical attack. Paint manufacturers will recommend products suited to these purposes.

The Australian/New Zealand Standard AS/NZS 2311:2009, Guide to Painting of Buildings, Section 3.9.2, contains recommendations for preparing masonry surfaces. In particular attention is drawn to the alkaline nature of concrete, moisture content, surface conditions and efflorescence.

Weathertightness is a more fundamental matter. The New Zealand Concrete Masonry Association has recently commissioned research work by the New Zealand Concrete Research Association and the Building Research Association of New Zealand on the effectiveness of various coatings in restricting moisture movement through masonry walls. This is a complex subject, with several avenues available for further research. The results so far, however, have shown some interesting patterns which form the basis of these recommendations.

Four different types of masonry units were selected to cover a range of aggregates, manufacturing processes, etc. Resistance to moisture movement is only one of several characteristics of the concrete masonry unit. It is therefore not surprising to find that this property varies considerably as different manufacturers programme their productions according to their raw materials, their plant capabilities, and the individual characteristics they wish to emphasise in the product. The four types selected for test are shown in **Table 1**.

The types represent a range of characteristics of masonry products currently in production from four plants. They do not match each individual plant's production. In each area, the manufacturer will be able to advise which of these four most closely represents his product from the point of view of resistance to moisture movement. The most critical characteristic is that of permeability.

Surface Preparation for Coatings

The surface to be coated must be clean and free from any dust or loose particles.

Oil spots and the like must be cleaned off by wiping with a solvent or by scrubbing with a detergent or trisodium phosphate solution. The wall must then be rinsed with water and allowed to dry.

Another condition that may be detrimental to the successful coating of concrete masonry is efflorescence. Efflorescence is not common in concrete masonry but can arise through site conditions or other influences.

Efflorescence usually shows as a whitish bloom of salts on the surface of concrete within which movement of water has carried the salts to the surface. If water continues to enter such concrete, even after coating, efflorescence could continue and cause damage to the coating. Efflorescence on concrete surfaces may be removed by acid brushing but prevention of efflorescence or its recurrence requires the elimination of movement of water into and through the concrete. Such water can come from many sources, some of which are wet soil behind a wall, water vapour condensing in a wall or roof, leakage from rain or surrounding water.

Problems of this sort must be resolved before coatings are applied.

Before applying solvent-based coatings, the surface should be dry for the top 1.5mm. Dry concrete masonry is usually light grey in colour. If in doubt, a moisture meter should be used.

In addition to paying attention to the masonry surfaces, care must also be exercised at adjacent or surrounding elements of the structure such as windows, flashings, services and sealants. Some of the aforementioned preparation methods could be damaging to such other elements if carried out without proper consideration and care.

Synthetic coatings and chemical sealants are in increasing use. In most instances, the coating selected for the masonry will also be applied to any sealants which have been employed, for example, at control joints or around joinery. Information on compatibility of coating and sealant should be requested from the manufacturer of each product. This precaution should be exercised with synthetic flashings or other items where there may be a reaction between the various materials.

Types of Coatings

The range of coating types and the sources of their supply seem to be ever increasing. Within this healthy development there have become known several manufacturers and suppliers with whom detailed selections and specifications for particular applications should be discussed.

Paint formulations can vary considerably, such that - for example - even though two paints may contain the same binding resin, the quality of the paints will be very different if one of those paints is formulated with less resin (consequently greater quantity of filler). A good quality paint, supplied from a reputable manufacturer is recommended.

The following comments are given to assist in providing a base for such discussions. The nature of the subject is such that not all coatings are mentioned, but those in current use are:

1.0 Paints/Coatings

1.1 Acrylic Pigmented Standard or Elastomeric High Build Paint ≥ 180 μm

Concrete walls shall be sufficiently dry to give a relative humidity reading of less than 70% at the time of coating application. The substrate shall be free of contaminates prior to the application of the coating system.

Coatings shall be applied out of direct sunlight and at temperatures between 5°C and 30°C, with the expectation that the temperature will be in that range for the following 12 hours.

Coatings shall not be applied in damp conditions.

Pigmented elastomeric high build acrylic coatings for exterior use shall have a dry film thickness of at least 180 μm. No less than two coats shall be applied.

1.2 Clear Sealer Coating

The coating system shall be supplied by a single supplier who takes responsibility for the system as a whole, encompassing the weathertight coating. The system shall be applied by the coating manufacturer’s approved applicator.

Clear coating systems are to be recoated every five years at a minimum or in accordance with the manufacturer’s specifications.

The clear coating system shall be designed to prevent water ingress into the pores of the concrete or masonry. The system shall allow the passage of water vapour from the interior to the exterior.

Clear coating systems shall be tested for permeability in accordance with AS/NZS 4456.16. The test shall be conducted on a standard masonry block with a density of between 1,350-1,500 kg/m^3 over a test period of two hours and show a permeability of 1 mm/hr or less.

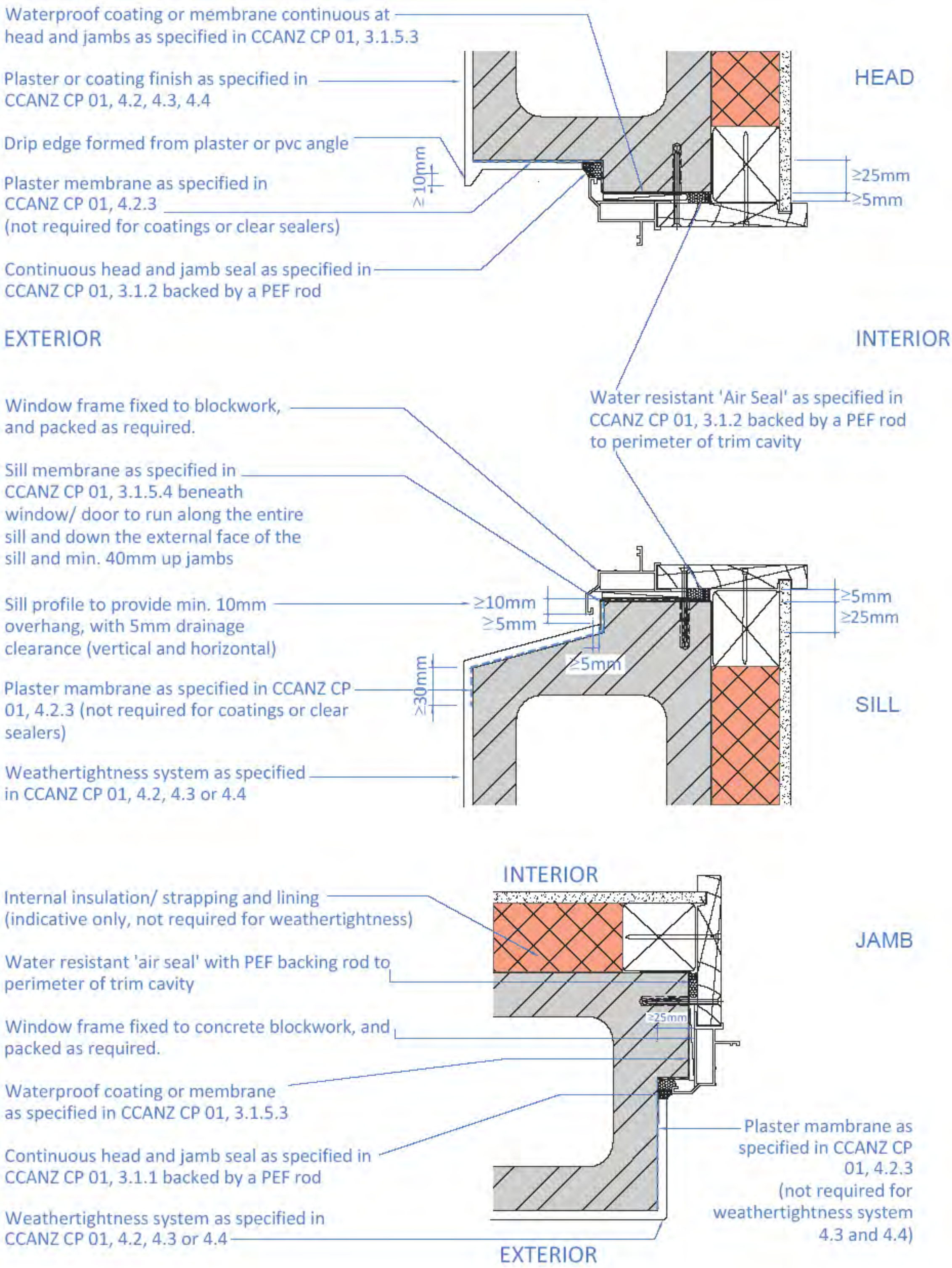
When building with a low permeability block, the tested permeability shall be 3 mm/hr or less. Low permeability blocks are blocks with a permeability of less than 10 mm/hr when tested in accordance with AS/NZS 4456.16.

1.3 Clear Coat Impregnations

Clear coat impregnations shall comply with EN 1504 part 2.

Detail 1 (page 5) shows typical window joints in coated and plastered concrete masonry walls.

Further construction details are given in ‘CCANZ CP 01’ with details 3b, 8, 19, 23, 27, 34a, 34b, 54, 58a, 62 and 66.



Comment 1: Structural layout is indicative only and subject to individual project design.
Comment 2: Thermal insulation is not required for weathertightness.

Detail 1 (not to scale)

Window - Head, Sill and Jamb

Weathertightness system: external coating or plaster

2.0 Plaster Systems

The substrate shall be free of contaminates prior to the application of the base coat.

Plaster shall be applied out of direct sunlight at temperatures between 5°C and 30°C, with the expectation that the temperature will be within that range for the following 24 hours.

2.1 Polymer Based Plaster System

Polymer based plaster systems comprise of a base coat with:

(i) Plaster of at least 3 mm thickness to form a flat plane surface,

(ii) Reinforcing with an alkali-resistant fibreglass mesh as specified in section 2.5.3.2,

(iii) Cover to the mesh of at least 1.0 mm of plaster, and a

(iv) Minimum bond strength of 0.1 MPa to the concrete or concrete masonry substrate, and a

(v) Polymer modified cement-based plaster finish coat with a standard acrylic coating of no less than 80 µm dry film thickness.

2.2 Polymer Modified Cement-based Plaster System

Polymer modified cement-based plaster systems comprise of a base coat with:

(i) Plaster of at least 3 mm thickness to form a flat plane surface,

(ii) Reinforcing with an alkali-resistant fibreglass mesh as specified in section 2.5.3.2,

(iii) Cover to the mesh of at least 1.0 mm of plaster and a

(iv) Minimum bond strength of 0.1 MPa to the concrete or concrete masonry substrate, and a

(v) Polymer modified cement-based plaster finish coat with a standard acrylic coating of no less than 80 µm dry film thickness.

2.3 Solid Plaster System

Solid plaster systems comprise of a base coat with:

(i) Bond or scratch coat 3 to 4 mm thick, reinforced with an alkali-resistant fibreglass mesh as specified in section 2.5.3.2, and

(ii) Flanking coat 9 to 15 mm thick in accordance with NZS 4251, and a

(iii) Solid plaster finish coat, 2-3 mm thick, applied in accordance with NZS 4251.

2.4 Three Coat Cement Solid Plaster

Three coat cement-based solid plaster in accordance with NZS 4251 Section 3: *Plaster system for concrete masonry walls.*

Detail 1 (page 5) shows typical window joints in plastered concrete masonry walls.

Further construction details are given in 'CCANZ CP 01' with details 3a, 8, 19, 23, 27, 34a, 34b, 54, 58a, 62 and 66.

3.0 Exterior Insulation Finish System (EIFS)

3.1 Limitations

EIFS shall be:

(a) Designed and tested as a total system, to meet NZBC E2,

(b) Supplied by a single supplier who takes responsibility for the system as a whole encompassing the durability, weathertight detailing and overall weathertightness, and

(c) Not fixed:

  (i) so as to form a horizontal surface, or

  (ii) in such a way as to allow water to pond.

3.2 General

COMMENT: *It is recommended that installation and finishing of EIFS is carried out by trained applicators who are approved by the New Zealand supplier of the system.*

3.3 Materials

EIFS shall comprise:

(a) A polystyrene rigid insulation board,

(b) A polymer-modified cement-based base plaster or a polymer-based base plaster, reinforced with fibreglass mesh, and

(c) A polymer-modified cement finishing plaster system or polymer-based finishing plaster system in one or more coats.

3.3.1 Polystyrene Board

Polystyrene boards shall be either:

(a) Expanded polystyrene (EPS) complying with AS 1366: Part 3, Class H or Class S, or

(b) Extruded polystyrene (XPS) that complies with AS 1366: Part 4.

COMMENT: *The minimum board thickness will be determined by structural and thermal requirements. For some EIFS, polystyrene boards are available with the base coat plaster factory-applied.*

The polystyrene boards shall be mechanically fixed at no greater than 600 mm centres and adhered to the wall using a cement-based mineral adhesive coat, tested for bond strength between polystyrene and concrete or masonry substrate, in accordance with ASTM E2134-01(2006).

The concrete or masonry wall shall be free of contaminates prior to application of the adhesive.

3.3.2 Fibreglass Reinforcing Mesh

The entire exterior surface of the polystyrene sheet (including corners) shall be continuously reinforced with an alkali-resistant fibreglass mesh, which shall:

(a) Weigh no less than 150 grams per m^2,

(b) Have an aperture size from a minimum 3 mm x 3 mm to a maximum of 6 mm x 6 mm square,

(c) Comply with the requirements of EIMA 101.91 test No. 6.3 and ASTM E2098,

(d) Be tested for alkali resistance by 28 days immersion in 5% sodium hydroxide with no visual degradation at the end of the test, and

(e) Overlap at mesh to mesh joints for at least 75 mm.

3.3.3 Base Coat Plaster

The base coat plaster shall:

(a) Be at least 3 mm thick and form a flat plane surface and be either:

  (i) polymer-modified cement-based plaster, or

  (ii) polymer based plaster,

(b) Be reinforced with an alkali-resistant fibreglass mesh as specified in section 4.1.3.2 of ‘CCANZ CP 01’,

(c) Cover the mesh by at least 1.0 mm,

(d) Be applied out of direct sunlight at temperatures between 5°C and 30°C, and with the expectation that the temperature will be within that range for the following 24 hours, and

(e) Have a bond strength with the polystyrene board tested in accordance with ASTM E2134-01(2006).

3.3.4 Finish Coat

The finish coat shall comprise either:

(a) A polymer-modified cement-based plaster or a polymer-based plaster, finished in both cases with a paint coating, or

(b) Either a pre-coloured polymer-modified cement-based plaster, or a pre-coloured polymer-based plaster with the top coat applied as a decorative plaster that is sealed or glazed.

COMMENT: *Dark colours cause* finishes *to reach higher temperatures, which results in more thermal expansion and a greater risk of cracking. Coating manufacturers can supply reflectance values.*

3.3.5 Openings and Penetrations

(a) All window/door openings shall have waterproof membranes as specified in section 3.1.5.2 of 'CCANZ CP 01',

(b) All wall recesses shall have waterproof membranes as specified in section 3.1.5.2 of 'CCANZ CP 01',

(c) All window/ door openings, wall recesses and penetrations shall have sealant, or air seals as detailed in sections 3.1.1 to 3.1.2 of 'CCANZ CP 01', and

(d) Openings and penetrations in EIFS shall be completed as shown in Detail 2 (page 9) and Details 53, 57a and 57b of 'CCANZ CP 01'.

COMMENT: *This is the minimum standard, and additional elements required by the system supplier should not be excluded on the basis of this Code of Practice.*

3.3.6 Decorative Mouldings

Decorative mouldings formed from polystyrene shall be glued onto the base coat plaster and in addition meshed on at the top edge. The adhesive bond strength shall be tested in accordance with ASTM E2134-01(2006). Control joints shall be reflected through the mouldings.

COMMENT: *Decorative mouldings formed from other materials are available, but due to unknown weight and rigidity of the mouldings specific design of the fixing is required.*

3.4 Movement Joints

Control joints shall be provided to coincide with the control joints in the masonry or concrete substrate. The joint shall be 8 to 15 mm wide as shown in Detail 69a of 'CCANZ CP 01'.

The front of the joint shall use either a sealant as specified in section 3.1.1 of 'CCANZ CP 01'or an EIFS joint profile as per Detail 69a of 'CCANZ CP 01'. The sealant shall have a width to depth ratio of 2:1.

At junctions between concrete walls and timber or metal frame walls, a control joint and back flashing as shown in Details 61 to 64 shall be provided.

3.5 EIFS/Floor Slab Junction

The bottom of the EIFS shall run at least 100 mm into ground as shown in Details 1 and 2 of 'CCANZ CP 01' and incorporate a waterproofing capillary break in the insulation.

The capillary break shall be formed by a continuous cut through the insulation board. The bottom section of the insulation board shall be made watertight by applying a membrane as specified in 4.2.3 of 'CCANZ CP 01'and as shown in Details 1 and 2 of 'CCANZ CP 01'.

COMMENT: *If the EIFS terminates above ground, no capillary break is required but the bottom edge of the EIFS should be finished using a PVC cap incorporating a drip profile.*

3.6 Parapets and Balustrades

Parapets and balustrades shall comply with section 6.11 of 'CCANZ CP 01'. Balustrades shall use the same weathertightness details and specifications as for parapets.

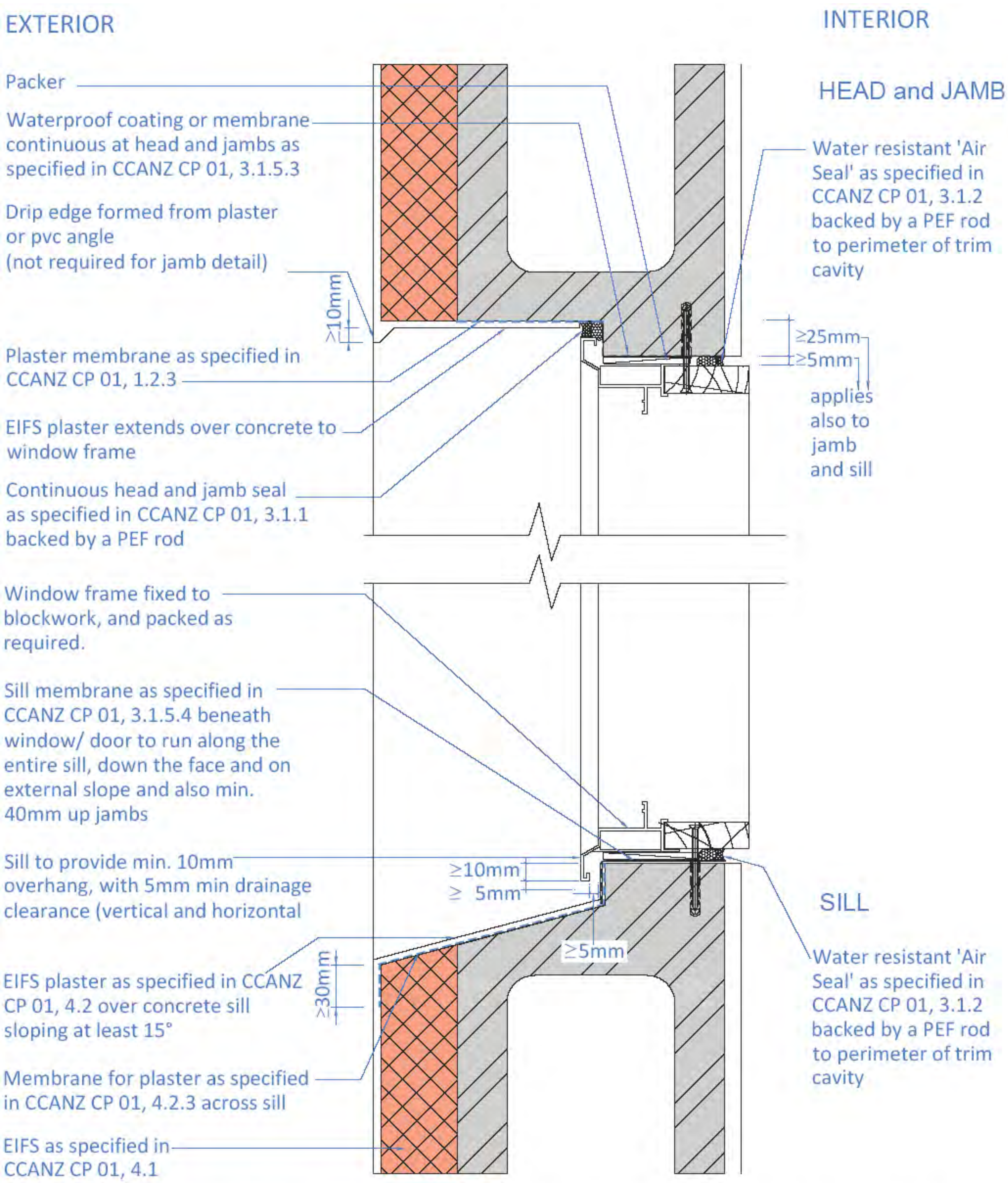
3.7 Fixings

Fixings of downpipes brackets, garden taps and other outside fittings shall be in accordance with NZBC E2/AS1 Paragraph 9.9.4.4.

Designs of fixing brackets for connecting items carrying substantial loads such as stringers for decks are outside the scope of this Concrete Masonry Manual and will require specific design.

The types of coatings available for the finishing of concrete masonry surfaces are so numerous and diverse that in all cases reference should be made to reputable manufacturers before finally deciding what system to use.

When appropriate, the paint coatings were applied in different ways to the masonry specimens using brush, roller or spray techniques.



Comment 1: Structural layout is indicative only and subject to individual project design.

Detail 2

(not to scale)

Window - Head, Sill and Jamb

Weathertightness system: EIFS

Summary

The properties of concrete masonry can vary significantly from one plant to another but with appropriate care there need be no problem with regard to weatherproofing. The manufacturer can provide information on his product and recommend the most appropriate surface coatings for any given situation.

Surface coatings should not be regarded as compensation for poor design detail or construction techniques. It is important that the established practices as outlined in other sections of this manual should be conscientiously applied.

Care in the preparation of the surface prior to application of a coating is important. The normal precautions recommended for any paint application should be followed but particular attention should be given to the possible presence of efflorescence materials. Possible chemical reactions between coating materials and sealants or flashings, etc. should be considered.

Special care should be exercised in the selection of coatings in those areas which are prone to heavier rainfalls at frequent intervals or high wind.

References

In order to evaluate the methods of weathertightness requirements reference should be made to 'CCANZ CP 01', a free download is provided online at: www.ccanz.org.nz/images/document/CCANZ-CP%2001%202011.pdf.

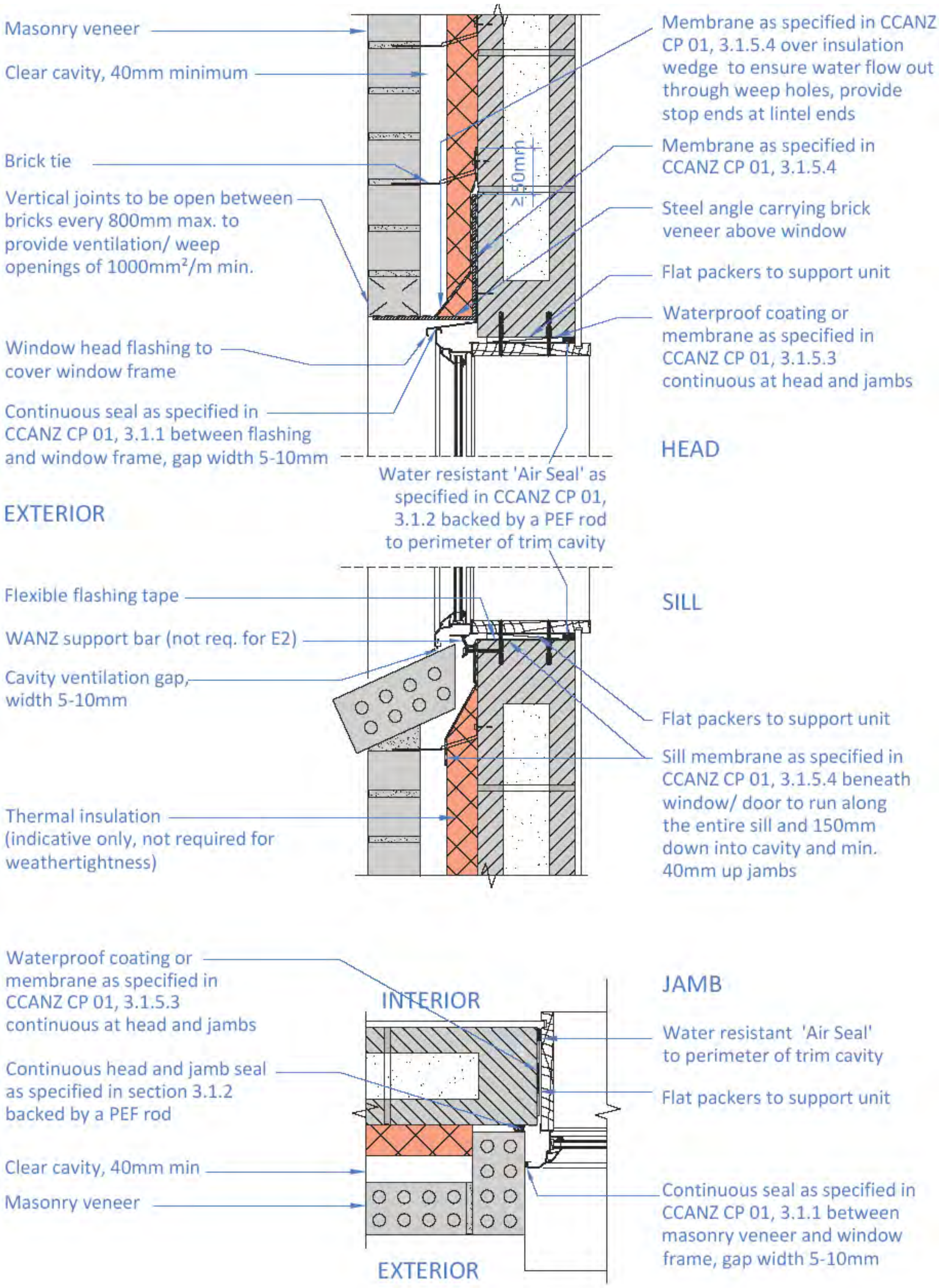
Veneer Section

The basic requirements for veneer construction are contained in Section 5 of the Masonry Manual.

This section has recently been revised to take into account the latest revisions to E2/AS1, E2/AS3 and NZS 4229:2013 Appendix E.

E2/AS1 deals with construction using timber frame as a structural wall support. E2/AS3 deals with construction using concrete or masonry as a structural wall support, which also appears as Appendix E of NZS 4229:2003.

The weatherproofing aspects are illustrated in Detail 3 on page 11. More details are shown in Section 5 of the Masonry Manual.



Comment 1: Structural layout is indicative only and subject to individual project design.
Comment 2: Thermal insulation is not required for weathertightness.

Detail 3

(not to scale)

Window - Head, Sill and Jamb

Weathertightness system: brick veneer with drained cavity

2.3 Thermal Performance/Passive Solar Design

Thermal Performance

Requirements for thermal performance of domestic dwellings are set out in NZS 4218:2009, *Thermal Insulation - Housing and Small Buildings*.

This standard is cited by MBIE as part of the acceptable solution within the New Zealand Building Code, Clause H1/AS1.

Concrete masonry construction R-values shall be determined by either the schedule, calculation or modelling method of the above standard.

Schedule Method

The schedule method shall only be used if the glazed area is ≤ 30% of the total wall area.

R-values are determined by Table 2 or Table 4. Table 2 requires R-values between 1.9 and 2.0 depending on the climate zone where the building is set. R-values required by Table 4 vary between 0.8 and 1.2 again in dependence on the buildings climate zone.

Table 2 can be used in any case. Table 4 can only be used if:

(a) the concrete masonry wall is openly exposed to the interior, and

(b) the density of the wall is ≥ 215 kg/m^2.

Comment: Fully filled, 150 mm concrete masonry walls are usually ≥ 215k g/m^2 but confirmation of the manufacturer is required for final assessment.

See NZS 4218, section 4.1 for further details.

Calculation Method

The calculation method shall only be used if the glazed area is ≤ 40% of the total wall area.
The advantage of this method over the schedule method is that a reduction of some building element's R-values can be compensated by increasing the R-values of other building elements.

For further details see NZS 4218, section 4.2.

Modelling Method

The modelling method can be used for any building but shall be used if the glazed area is larger than 40% of the total wall area.

The sum of the modelled and calculated heating and cooling load of the proposed building shall not exceed the reference building where R-values of Table 2 (or 4 if high mass) have been used.

The advantage again is that a reduction of some building element's R-values can be compensated by increasing the R-values of other building elements.

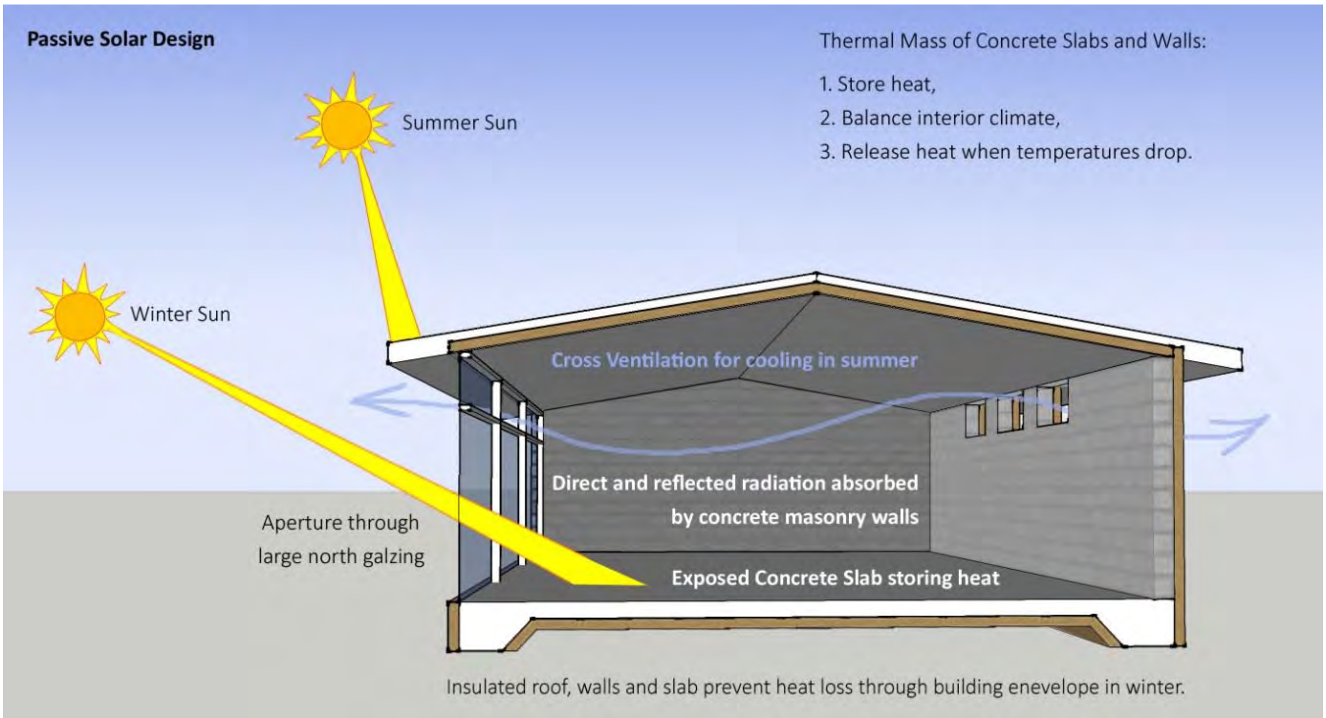
For further details see NZS 4218, section 4.3.

Passive Solar Design

The principles of passive solar design are:

1. Thermal mass
2. Large north glazing, no or minimal south glazing
3. Thermal insulation
4. Air sealage
5. External summer shading
6. Providing for cross ventilation

Image: *Cranko Architects using principles of passive solar design for a residence in Wellington.*



For further information see the 'Designing Comfortable Homes' Book, free download online at: www.CCANZ.org.nz/files/DCH_Book_WEB.pdf.

Principles of passive solar design as illustrated:

> *'Large glazing to the north to gain the sun's heating energy in winter plus roof overhangs to block the sun in the summer avoiding overheating are evident for passive solar design.*
>
> *Concrete masonry walls together with an exposed concrete slab capture direct and reflected radiant heat and balance the interior climate by cutting off daily peaks, highest and lowest temperatures.'*

Trombe Walls

The CCANZ *Concrete and Concrete Masonry Trombe Wall Information Bulletin* covers the design and application of Trombe walls which are passive solar building elements that, designed correctly, provide an effective way of conserving energy in buildings.

Trombe walls are sun-facing, solid concrete or concrete masonry walls that are separated from the outside by glass and an air space. They make full use of the low rising sun's energy during winter months by collecting this heat during the day and releasing it into the room behind over an extended period of time and during the night.

When the sun shines, energy comes through the glass and is stored in the wall's thermal mass. When the sun sets or is blocked and the temperature drops, the wall releases its heat into the room behind.

Trombe walls work in a similar way to a greenhouse, by trapping solar radiation. The solar heat's higher-energy ultraviolet radiation has a short wavelength and this passes through glass almost unhindered. When this radiation strikes a wall or slab, the energy is absorbed and then reemitted in the form of longer-wavelength infrared radiation. The infrared radiation does not pass through glass as easily, so the heat is trapped and builds up in the enclosed space.

This publication can be downloaded from http://www.nzcma.org.nz/document/279-32/IB96_Trombe_Walls.pdf

2.4 Acoustic Performance

Contents

Introduction

Acoustics is the science and technology which deals with sound in and around buildings. It studies the phenomena of sound reflection, absorption and insulation and applies this knowledge in noise control and the creation of rooms with special acoustic characteristics such as concert halls and theatres.

For the practitioner, sound insulation is probably the most usual criteria to be satisfied.

It is interesting to note that many lightweight systems set their goals on trying to upgrade their systems sound properties on the levels achieved by concrete masonry. Sound insulation characteristics of masonry walls have been taken for granted in the past. Relaxation in fire regulations has permitted the greater use of lightweight construction which has often been to the detriment of sound insulation values.

Each principal phenomena of sound, i.e. insulation, reflection and absorption, is studied in this Manual Section.

Summary – Sound Insulation Performance/Classification

At the moment the only restriction placed upon the designer with respect to sound attenuation is on building elements which are common between occupancies. Both the Sound Transmission Class of walls, floors and ceilings, and the Impact Insulation Class of floors shall be no less than 55, as specified in New Zealand Building Code G6.3.1 and G6.3.2.

Table 1 is included to give further recommended classifications of sound attenuation performances, with descriptions, STC values and comments and methods of compliance in masonry construction.

Table 1: Recommended classification of sound attenuation performance

Description	STC (Median)	Comment	Masonry Specification
Space divider	< 30 dB	Has little acoustic merit and is a space divider only.	
Acoustic separation	> 30 dB	Will provide acoustic separation between adjacent occupants but conversations may be heard if listened for. Adequate for many offices.	Any masonry construction
Acoustic privacy	> 40 dB	Will provide privacy. Conversations will only be heard if loud, or in very low background noise. Adequate for offices and those walls inside a house where a high rating may be called for, e.g. living and sleeping areas.	Solid filled 10 series Partially filled 15 series
Acoustic	> 50 dB	Domestic intertenancy, motel, hotel walls will require to be security of this standard.	Solid filled 15 series
Acoustic special	> 55 dB	NZ Building Code G6 Domestic intertenancy	Solid filled 20 series

Tests carried out at Auckland University in 1993 gave the following results for the STC for 20 series solid filled:

Wall Type	STC
20 Series masonry block wall, every third cell reinforced all cells grout filled	55
Same as case (b) but with both faces of the wall painted with acrylic paint	56

From these results it can be seen that the STC rating for walls can be achieved through the use of concrete masonry.

These results and further overseas regulations and practices are referred to in more depth later in the Acoustic Properties Section

General

Noise can be defined as all unwanted sounds. Whether sound is experienced as pleasant or as mere noise it is a very subjective thing. A few examples will make this clear. The owner of a sports car will be excited by the roar of his engine while bystanders are most likely disturbed and regard it as a form of environmental pollution. Playing one's favourite music loudly on the stereo can be a pleasant experience while the neighbour might regard that same volume of music as a nuisance. In general, one's own sound sounds good and that of somebody else's does not.

Our modern industrial society produces an ever increasing number of sound sources and at the same time we are growing towards denser housing forms. This results in a growing need for noise control. Since noise is now regarded as a form of environmental pollution it has become a public issue and subject to growing legislation.

Protection from noise (excessive sound) is not a luxury but a need. Some countries have bylaw legislation setting standards of acceptable sound insulation. These are outlined in the classification section to follow. Continuous exposure to high noise levels affects our psychological well-being. This is well understood in the world of industry where it was proved that the reduction of noise of more than 90 dB to acceptable lower levels led to higher productivity through better concentration resulting in more output, less accidents and more job satisfaction.

Concrete masonry blocks possess a combination of acoustical properties found in few other building materials. The open surface texture has good sound absorbing qualities while the mass of a block wall guarantees good sound insulation at the same time as the structural and fire resistant functions within the building are fulfilled.

The Nature of Sound

Sound can best be explained as pressure waves or vibrations, in the air or any other elastic material. Sound needs a medium in which to travel; sound is not transported in a vacuum. In air, sound travels with a speed of approximately 340 m/s (20°C).

Sound can be generated by any object that will vibrate rapidly. From this source sound travels equally in all directions. When a sound is generated in a room the sound waves are reflected after a few milliseconds by the walls, floor and ceiling. Shortly after, the room is filled with sound waves travelling in all directions.

Airborne Sound

Airborne sound is the term used to describe sound waves travelling through the air. It is sound caused by such things as speaking, music, machinery and traffic.

The most common airborne sounds in apartments are neighbours talking and traffic noise.

Airborne sound is measured in RwdB, which is the difference in sound levels either side of a barrier, such as a building partition. The higher the RwdB figure, the better the sound attenuation.

The airborne sound attenuating performance of a material or system is described by its sound transmission class (STC), which classifies its ability to resist airborne sound transfer at the frequencies 125 Hz to 4000 Hz. The NZBC requires the STC of walls, floors and ceilings to be no less than 55.

However, the human ear can perceive frequencies as low as 50 Hz, which is below the frequencies considered for a building material or system's STC. Sounds at these frequencies, such as bass tones from deep voices or stereo systems, can be some of the most penetrating and disturbing.

The conventional rule for reducing airborne sound transmission is the mass law. This says that the heavier the structure, the less sound it will transmit (i.e. the more attenuation for airborne sound and sound at low frequencies). R'w is the weighted apparent sound reduction index.

Impact Sound

Impact sound is the term used for sound waves that are generated on a partition and caused by such things as footsteps on the floor, hammering onto a wall, slammed doors and windows, or vibrating plant devices.

The most common impact sound in apartments is caused by people walking on the floor above a unit.

Impact sound is measured in LnwdB, which is the sound level transmitted below a floor from impact applied to the floor from above. The lower the LnwdB figure, the better the sound attenuation.

The impact sound attenuating performance of a material or system is described by its impact insulation class (IIC). IIC indicates the amount of impact noise isolation provided by a floor/ceiling assembly. The NZBC requires the IIC of floors to be no less than 55.

Impact sound can only be minimised by uncoupling the building elements. Therefore, to reduce the noise from people walking in the apartment above, it is necessary to separate the stepped-on surface from the structure.

Frequency

The human ear is a very sensitive instrument to register sound. The frequency of the sound waves is experienced as pitch and the amplitude of air pressure variation is experienced as the loudness of a sound.

The ear can detect sound with a frequency ranging between 20 Hz (low pitch) and 20,000 Hz (high pitch). Hz (Hertz) is the unit of frequency - previously known as cycles per second. The sensitivity of the ear is not the same over the whole frequency range. When human beings get older they slowly lose their ability to hear sounds with very high frequencies.

The highest sensitivity is found in the middle range. Most important is the range from 125 Hz to 4,000 Hz. This is essentially the range of frequencies important in building acoustics.

Doubling or halving the frequency is experienced as a difference of one octave. Most sounds encountered in daily life however are not pure sounds, i.e. they contain more than one frequency. They are a mixture of continuously changing composition and intensity.

Loudness

Sound waves can be described as compression waves travelling in a manner akin to the push-pull effect between wagons of a long train when starting and stopping.

The pressure difference between the compressed and decompressed parts of the wave has to pass a certain minimum pressure to become audible. This threshold varies according to frequencies.

Sound waves in fluids (e.g. air and water) are compression waves, and the passage of sound waves through air will cause variations in air pressure. In solids, sound waves may cause bending in shear or flexure.

Under favourable circumstances the human ear can detect airborne sounds with an rms (root means square) pressure variation of only 2×10^{6} Pa, at 100 Hz. The sound intensity, i.e. rate of energy transfer per unit area (watt/m^2) corresponding to this threshold pressure is 10^{-12} W/m^2.

The intensity of these faint sounds is indeed very small, but for a short period, however, the ear can withstand sounds with a rms pressure of 200 Pa or an intensity of 100 W/m^2 without damage. Such a sound lies on the threshold of pain.

In the practical situation the rms pressure is not often used because it gives a scale which does not relate very well to how differences in sound intensity levels are experienced.

This led to the introduction of the "belscale" which is a logarithmic scale based on sound energy intensity (this is related to the square of the rms pressure). In the belscale a change of 1 B (bel) corresponds with a tenfold change in energy level, 2 B corresponds with a hundredfold change, 3 B with a thousand-fold change and so on. For practical reasons the bel (B) was subdivided in ten decibels (dB), thus creating the well-known term decibel.

A decibel is the smallest change in sound intensity detectable by the human ear. The decibel scale is a relative scale and in order to be practical a zero point had to be chosen. The average threshold of hearing was adopted for this purpose.

Table 2 (page 4) shows the decibel scale with the related rms pressures and energy intensity levels.

The scale is illustrated with examples and the subjective reaction to such sound levels.

Table 2: Decibel Scale

rms pressure in Pa	**Energy intensity in W/m²**	**Decibels (dB)**		
200	100	140	Threshold of pain	Test stand jet engines on short distance
	10	130		Loud air-raid siren
20	1	120	Threshold of comfort	Machine gun on short distance
	10^{-1}	110		Wood working shop
2	10^{-2}	100		Loud car horn at 6 m
	10^{-3}	90	Very Loud	Printing plant
2×10^{-1}	10^{-4}	80		Orchestra fortissimo
	10^{-5}	70	Loud	Busy street
2×10^{-2}	10^{-6}	60		Average radio
	10^{-7}	50	Noisy	Average conversation
2×10^{-3}	10^{-8}	40	Normal	Quiet office
	10^{-9}	30		Soft radio
2×10^{-4}	10^{-10}	20	Quiet	Whisper at 1 m
	10^{-11}	10	Very faint	Silent garden
2×10^{-5}	10^{-12}	0	Threshold of hearing	Acoustical test room

Behaviour of Sound

Briefly summarised, sound is energy in the form of successive pressure waves transmitted through the air. As such it cannot pass through a solid obstacle such as a wall or a floor.

Most structures, however, are not fully airtight and sound travels easily through continuous pores, small holes and voids, cracks and joints.

Depending on the character of the surface texture, sound will be reflected and/or absorbed.

Dense smooth surfaces will reflect much of the striking sound while rough porous textures will absorb and dissipate much of it.

Part of the sound energy is transmitted by producing vibrations in the wall. In fact these vibrations are also sound waves but travelling through medium other than air.

After having been transmitted through the obstacle the vibrations regenerate sound waves in the air on the other side, but of lesser strength because of energy dissipated during transmission through the obstacle (***Figure 1***).

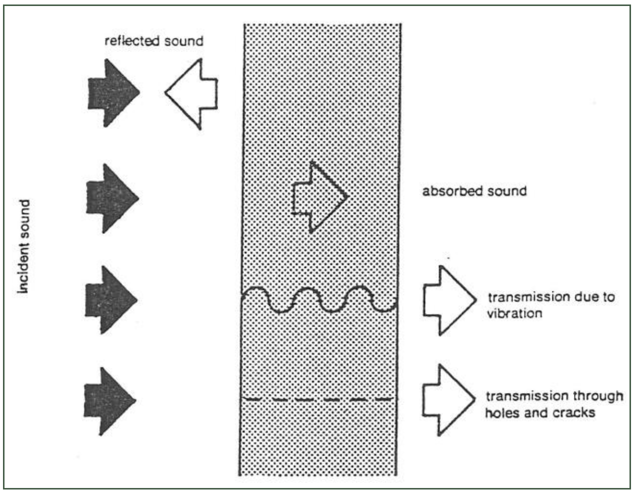


Figure 1: *The behaviour of sound when striking a solid object*

Sound Transmission

Speakers, singers, violinists and babies produce air-borne sound. They generate sound waves directly in the air. Upstairs neighbours walking on the floor, cellists, sanitary installations and wall mounted telephones produce air-borne sound and structure-borne sound. Vibrations are directly transferred to the structure.

To prevent these two types of sound passing through a wall or floor the same approach is followed for both, namely to try preventing the wall itself from vibrating as far as this is possible. This principle sound insulation is resolved in quite different ways, however, for the two types of sound.

To prevent the transmission of structure borne sound the best way is to absorb the vibration close to the source. Flexible mountings of sound emitting objects will absorb most of the vibrating energy and prevent it continuing into the structure. It must be noted that by proper design and detailing structure borne sound can be completely avoided in many cases.

The insulation of air-borne sound is achieved mainly by giving the wall or floor sufficient mass. The more mass the more energy is needed to vibrate this wall. According to the same physical law, more energy is required to vibrate the wall with a higher frequency. In theory the sound attenuation should rise by 6 dB every time the frequency doubles. In practice the sound attenuation graph follows the theoretical line only more or less (***Figure 2***). An important deficiency can be the resonance frequency of the wall itself.

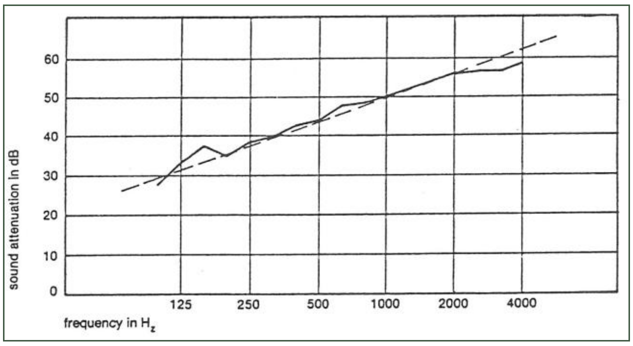


Figure 2: *Sound transmission loss of 200mm concrete masonry wall, ungrouted, compared with 6 dB octave slope*

The sound attenuation quality of a structure is expressed as Sound Transmission Class (STC) and is measured in dB's. This is derived from the attenuation figures measured in the different frequency ranges. The STC is a single figure rating based primarily on the relative importance of the different frequency ranges in achieving privacy of speech of sound.

Sound Attenuation of Concrete Masonry

Properly designed and constructed concrete masonry walls will provide good sound attenuation. A good sound insulating material requires properties much the opposite of the ones for sound absorption, low porosity, high density and a close surface texture are needed.

Transmission loss increases with wall thickness (mass) and increases further by having air spaces discontinuities across the thickness of the wall, such as open cores and wall cavities. Wall ties as used in cavity walls can act as sound bridges and reduce much of the acoustic value of the cavity. Plastering and/or painting one or both faces of the wall increases the sound attenuation.

Figure 3 illustrates in graphical form the relationship of single leaf block walls with a certain mass per unit area and the expected STC value. Table 3 gives the mass per m^2 of commonly used wall construction in New Zealand. When plastered on both sides walls can gain as much as 50 kg/m^2 in mass which adds considerably to the STC.

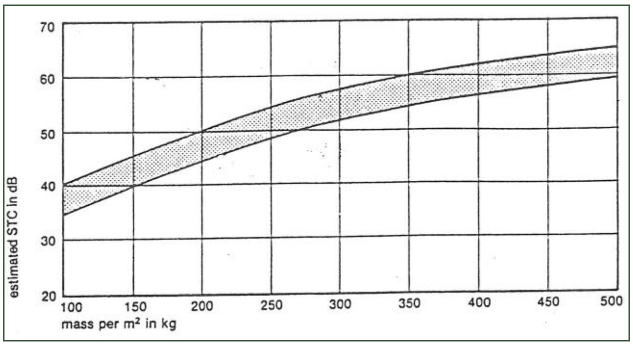


Figure 3: *The relationship of estimated SIC to the mass per m^2 of a single leaf concrete masonry wall*

Full scale testing was commissioned through the University of Auckland in 1988 and 1993.

In the first series of tests, a 15 series wall was built, reinforced and grout filled to produce a partially filled wall which would be typical of structural requirements in Seismic Zone C. The test results from the report pages 1-3 "Transmission Loss of a Masonry Wall Construction" April 1988 are attached.

In the second series of tests carried out in 1993 a block wall was constructed of 20 series blocks, but this time it was completely grout filled. The test results from the report pages 1-9, "Transmission Loss Measurements of a Reinforced Solid Core Block Wall" November 1993, are also attached.

The test method utilised by the Applied Research Office at the Acoustics Centre is outlined on Pages 4-7 for the first series and pages 9-12 for the second series. These pages are not included but copies of the full report are available on request from the Cement and Concrete Association of New Zealand.

Construction of the test wall for series one (1988) is shown in ***Figures 4 and 5***. The sound transmission coefficient based upon ASTM method of calculation was applied to the results obtained in Figure 1 of the attached report. The STC rating obtained for this wall construction was 42 but was close to achieving a 43 level. Bearing in mind, many masonry walls will be painted and the texture of the surface the test shows clearly that the predicted values of STC based on international results for masonry as shown in Figure 3 are substantiated for New Zealand materials. This is borne out in the second series of tests carried out in 1993. The wall was tested in both unpainted and painted conditions. The unpainted wall had a STC of 55 and the painted wall had a STC of 56 (see the attached results).

The summarised STC values for different masonry walls are set out in Table 1.

As far as ungrouted parts of the wall are concerned, the sound attenuation is very dependent on the workmanship of the block layer. A small gap in a mortar joint can cause a serious sound leak. When placing a closure block, especially, it is important to make a tight perpendicular joint. In cases where acoustical performance is of great importance, as in intertenancy walls, it is recommended to fill the space between the frogs on each side of a closure block with mortar. Plastering of the wall will give an extra safeguard against leaks and also adds mass.

Table 3 shows the result of acoustic tests on masonry walls of New Zealand made blocks. Although the test programme was limited they give a good indication of the sound attenuation qualities of concrete masonry. The STC values for the two single leaf walls conform very well with Figure 3 (estimated STC values). It must be noted that the test walls had none of their cores grouted. In the practical situation at least some cores are reinforced and subsequently grouted. This increases the wall mass but fills voids and will have an influence on the STC. Another point is the relative concentration of mass and its influence on the STC values. For these reasons it was decided to carry out the acoustic testing at Auckland University as described earlier.

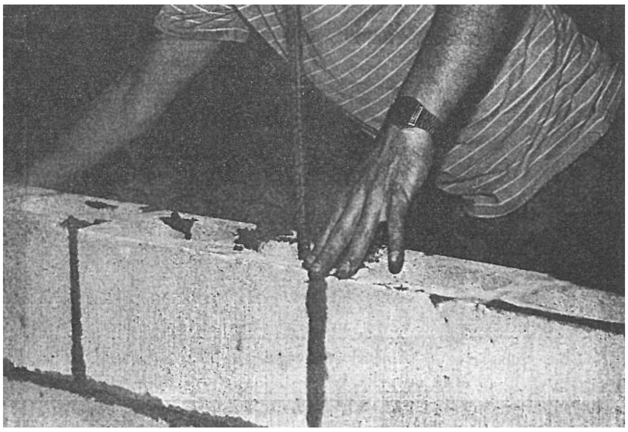


Figure 4: Laying up 15.05 units with reinforcing steel spaced at 800 mm centres

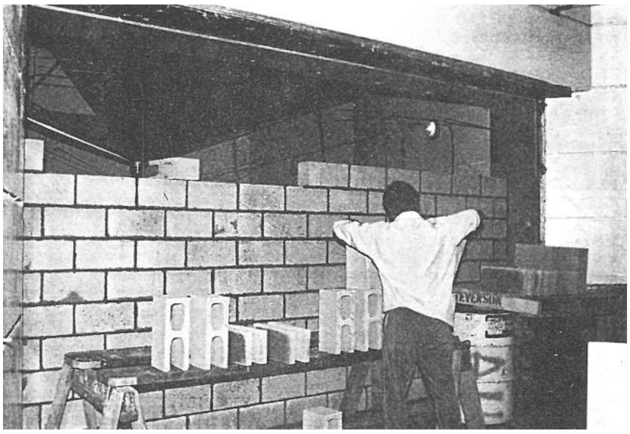


Figure 5: General view of wall under construction

Table 3: Sound transmission class of concrete hollow masonry in dB's as tested by BRANZ

STC (Median)	Wall Description
45	100 mm concrete block only
50	200 mm concrete block only
55	200 mm hollow concrete block + 50 mm cavity filled with R9 fibreglass + 100 mm concrete block + 13 mm Gib-board over timber furring
50	200 mm hollow concrete block + 50 mm cavity filled with R9 fibreglass + 100 mm concrete block

Classification in New Zealand Building Code G6

Many countries have standards and building codes covering sound attenuation of walls and floors in different situations and different types of buildings.

Released in July 1992, the new New Zealand Building Code and Approved Documents (NZBC) includes a section on airborne and impact sound. As with all other sections of the new code, it is in five main parts. The first three, the objective, the functional requirement, and the performance, are given below. The other two parts are the verification method, which is to be used in the case of specific design, and the acceptable solutions, which have been proven to meet the criteria and may be used without any specific design or testing. The first three main parts are as follows:

Objective

G6.1 The objective of this provision is to safeguard people from illness or loss of *amenity* as a result of undue noise being transmitted between abutting occupancies.

Functional Requirement

G6.2 *Building elements* which are common between occupancies, shall be constructed to prevent undue noise transmission from other occupancies or common spaces, to the *habitable spaces of household units*.

Performance

G6.3.1 ***The Sound Transmission Class* (STC) of walls, floors and ceilings, shall be no less than 55.**

G6.3.2 **The *Impact Insulation Class* (IIC) of floors shall be no less than 55.**

The verification method of the code states that any field tests carried out with respect to airborne sound insulation or impact sound insulation shall be within 5 dB of the performance requirement.

The acceptable solution states that sound transmission through building elements shall be minimised. To do this several options are listed.

Figure 6 is a schematic presentation showing the building elements which require noise control between separately occupied habitable spaces.

Whilst the New Zealand Code has some of the toughest compliance levels in the world, only a few situations are covered. Comparing the NZBC with other overseas regulations and practices clearly show these differences.

Typical overseas regulations and practices are summarised for some of the more common situations. However designers are advised to consult the primary reference sources to ensure a full understanding of their application.

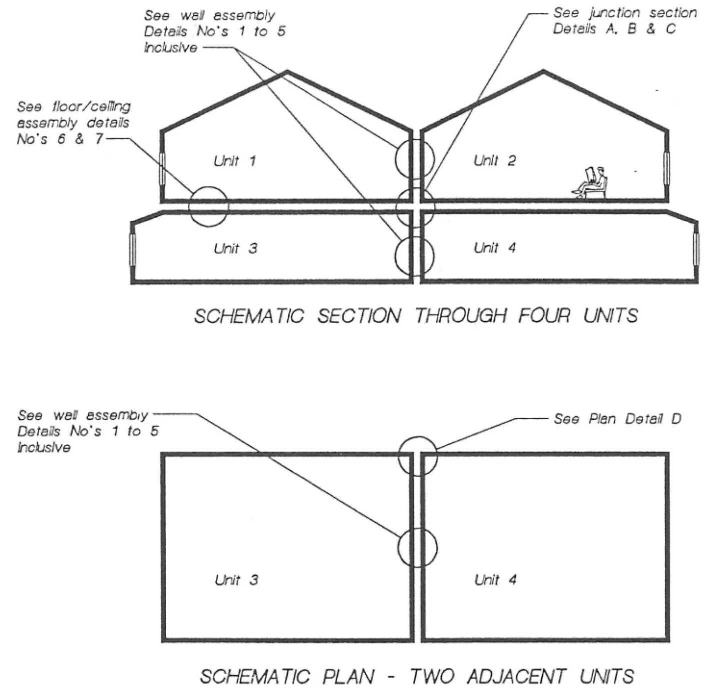


Figure 6: *Location of building elements requiring noise control. Note - references refer to NZBC Section G6 which is the source of this figure.*

Documents referred to are:

- BS 8233:1999 (UK) Code of Practice for Sound Insulation and Noise Reduction for Buildings.
- Sound Control for Homes, CIRIA Report 114, 1986.
- AS 2107:1987 (Australia) Acoustics - Recommended Design Sound Levels and Reverberation times for Building Interiors.

It is pertinent to refer first to CIRIA Report 114 because this gives:

(a) a summary of sound attenuation requirements

(b) a list of typical complying constructions

(c) a well-documented calculation method for dealing with external noise reduction through the building envelope.

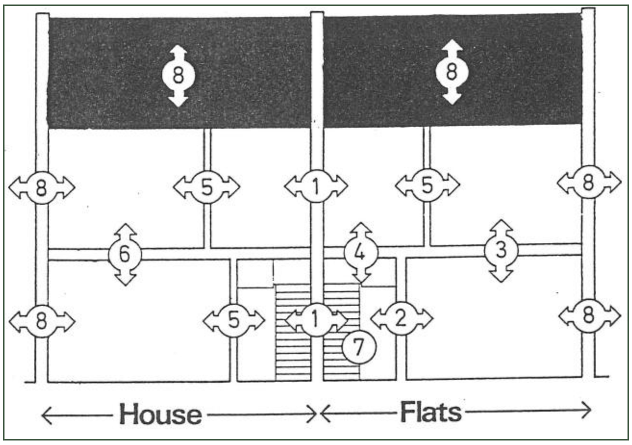


Figure 7: Typical construction separations taken from CIRIA Report 114.

Walls and Floors Covered by the New Zealand Building Code 1992

Wall 1 Separating wall between dwellings. Minimum sound insulation 55 dB.

Wall 2 Separating wall in communal lobby. Minimum sound insulation 55 dB.

Floor 3 Separating floor between dwellings. Minimum airborne sound insulation 55 dB. Minimum impact sound insulation 55 dB.

Floor 4 Take as 3.

For Walls 5 through 8 there are no requirements or recommendations.

Walls and Floors covered by UK Building Regulations 1985.

Wall 1 Separating wall between dwellings. Minimum sound insulation 52 dB.

Wall 2 Separating wall in communal lobby. Minimum sound insulation as for 1.

Floor 3 Separating floor between dwellings. Minimum airborne sound insulation 51 dB. Minimum impact sound insulation 62 dB.

Floor 4 Take as 3 but could relax impact sound requirement.

Walls and Floors not covered by Building Regulations. Suggested Standards.

Wall 5 Partition wall quiet 46 dB. Partition wall general 38 dB.

Floor 6 Partition floor
Above a quiet room airborne 46 dB.
Above a quiet room impact 68 dB.
Below a quiet room airborne 46 dB.
Below a quiet room impact 38 dB.

Area 7 References are made to special consideration for sound absorption surfaces.

Building Envelope 8

Building Envelope 8 is now discussed in more detail.

Designing the sound insulation of the building envelope requires the following information.

1. External noise level
2. Maximum design sound level inside the dwelling.
3. Surface area of the relevant portion of the building.
4. Sound absorption of receiving room.

The CIRIA Report 114 advises that a simplified approach can be used for housing. Items 3-4 can be ignored and a simplified external noise spectrum used.

When masonry walls are used it becomes only necessary to consider the detailing and construction of windows, doors, etc. Walls built of lightweight materials have a lower sound insulation value than masonry walls at low frequencies. Since road traffic noise peaks at low frequencies it follows that light weight construction is not as an efficient insulator as masonry.

The difference is very noticeable to residents changing from a traditional timber weatherboard house to a masonry house. Where traffic or external noise is likely to be a problem, the first step is to use concrete masonry walls as a first option followed by veneer construction. Then attention must be paid to the detailing of windows. A single glazed window has a dB rating of 24 when viewed against traffic noise (masonry typically 47, timber typically 34-39). Double glazing with 100 mm gap between panes produces 37 dB.

Typically a masonry wall with 20% openings, single glazed, would have rating of 32 dB against traffic noise. With double glazing as described a rating of 43 dB.

A working document, CIRIA Report 114, is very useful for planning and designing for noise control for domestic construction.

While domestic structures are also included in AS 2107 and BS 8233, these documents include wide ranging advice on insulation levels.

Important areas for using masonry are seen in education buildings where a recommended minimum sound reduction between classrooms is 45 dB (BS 8233). Note this can be achieved with 150 series all cells filled.

Where special rooms require low intrusion of external noise, such as libraries, the barrier wall is best served by masonry construction since usually a 45 dB level is required. For offices where privacy is required the sound insulation level should be a minimum of 45 dB (BS 8233). AS 2107 recommends slightly lower values, 35-40 and 40-45 respectively.

From this section it can be seen that not only is there attention paid to between dwelling noise transmittance, but also there is some attention paid to external noise. Hence, the designer of a structure which is to be built in New Zealand may be wise to take noise attenuation from external sources into account.

Sound Absorption

The application of sound absorbing materials is one of the most important tools with which the acoustic designer can influence the acoustic character of a room or hall. It leads to greater economy, of course, if the absorbing material is also a structural material. Concrete block is one of the few building materials which combines acoustic and structural qualities.

Sound absorbing materials are commonly used to quieten noisy rooms such as airport lounges, hotel bars, open plan offices and manufacturing plants. These materials are also used to adjust the reverberation time (echo characteristics) of auditoria, theatres or concert halls. By applying the right degree of absorption satisfactory clarity and volume of sound is achieved for the type of performance for which the room is designed. In rooms designed for speech the potential for generating echo needs control as this will interfere with understanding. Not all reflected sound is undesirable. A room in which all sound generated is immediately absorbed can be unpleasant and could be called acoustically dead.

The sound absorbing qualities of concrete blocks depend on their surface texture. This is determined by the texture of the aggregates and the mix design. Also the frequency spectrum of the incident sound plays a role because there is a relationship between the wave length of the absorbed sound and pore dimensions. Rough and open surface textures generally absorb sound well, while smooth dense surfaces such as a plastered wall reflect much of the sound.

Absorbent surfaces allow the pressure waves to enter the pores in which they are reflected many times and become widely dispersed. The rough surface of the pore walls causes resistance to the airflow and the sound energy is transferred in to heat. The highest absorption is attained with concrete units using lightweight aggregates or with dense aggregates with an open surface structure and high internal porosity such as no fines concrete. For special acoustical applications custom designed surface shapes can add considerably to the absorption.

Masonry units and walls can be custom designed to achieve desired sound absorption properties. Examples of such special design are reported in the Concrete Masonry Association of Australia's publication "Concrete Masonry for Acoustic Control", MAPR 9 and "Tuning Concrete Masonry for Transformer Enclosures" MAPR 21. In both these cases, cavity walls were used incorporating an inner leaf of sound absorbing masonry and an outer leaf of sound attenuating material. The functions of sound absorption and sound attenuation were thus combined in a single structure.

It is believed that sound absorbing materials add to the acoustical insulation of a wall or floor by damping the coupling air space particularly in framed constructions. This is certainly true for materials which have absorbent qualities and sufficient mass, such as concrete block.

Sound absorbing qualities can be compared by means of absorption coefficients which are measured at bands within the audible spectrum. The average value over the spectrum for a material is known as its Noise Reduction Coefficient (NCR). Such coefficients range from 0.03 (3%) for hard plaster to 0.8 (80%) for special absorbent materials. The sound absorption coefficient is derived as a proportion of an open window, assumed to be 1 (100%) because there is no reflection at all - which equals total absorption.

Any material with an NRC of 0.15 or higher is considered useful for sound control. According to ***Figure 8***, the NCR of concrete block walls ranges from 0.06 to 0.68 (untreated). This places the majority of blocks in the category of useful for sound control.

The table in ***Figure 8*** (page 10) is based on international data because absorption figures for the full range of New Zealand produced blocks are not yet available. It is likely that they will differ from plant to plant since each plant uses different aggregates and different mix designs. ***Figure 8*** supplies some guidance for general cases but in special situations, tests should be carried out on the actual materials used.

A typical comparison illustrates the potential role of masonry in a sound absorption role. Consider a room 4 m square, ceiling height 2.4 with 2 m^2 glazing, 1.5 m^2 timber door, walls built in light weight (1,850 kg/m^3) masonry.

	m^2		**Absorption Coefficient**		
Concrete ceiling	16.00	x	0.04	=	0.64
Masonry walls (38.40 - 3.50)	34.90		0.40	=	13.96
Door	1.50		0.06	=	0.09
Glazing	2.00		0.02	=	0.04
Carpet floor	16.00		0.45	=	7.20
	70.40			=	21.93

$$\text{Average NRC} = \frac{21.93}{70.40} = 0.31$$

Repeat but with timber panelling

	m^2		**Absorption Coefficient**	
Concrete ceiling	16.00	x	0.04	0.64
Panelling	34.90		0.06	2.09
Door	1.50		0.06	0.09
Glazing	2.00		0.02	0.04
Carpeted Floor	16.00		0.45	7.20
	70.40			10.06

$$\text{Average NRC} = \frac{10.06}{70.40} = 0.14$$

For noise reduction comfort the sound absorption for a typical room as a whole should be between 0.2 and 0.5.

Rooms with special requirements for music, singing and speaking need a careful balance of reflection and absorption which may on occasions be obtained by different decorative wall treatments on masonry walls.

Unrendered and unpainted concrete masonry absorbs more sound than surface treated walls. Paint applied by brushing tends to seal the outer pores, reducing sound entry and dissipation, although light spray painting reduces sound absorption only marginally. This painting does of course increase sound insulation. A point explained in the earlier part of the acoustic section.

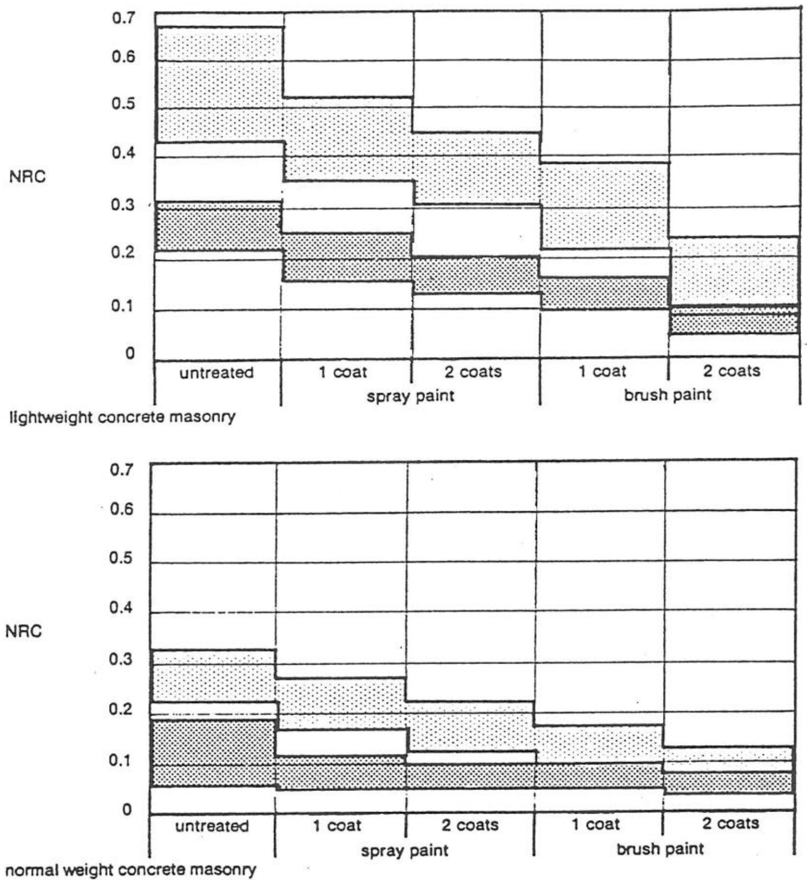


Figure 8: Sound absorption of concrete masonry expressed as Estimated Noise Reduction Coefficient (NRC).
(This information is based on a large number of tests carried out in the USA by the PCA and the NCMA)

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Further Reading

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ARO/3012

88T02

TELARC

REPORT ON TRANSMISSION LOSS OF A MASONRY WALL CONSTRUCTION

Report prepared for: N. Z. Concrete Research Association
Private Bag
PORIRUA

Report prepared by: Acoustics Testing Service
School of Architecture
UNIVERSITY OF AUCKLAND

APPLIED RESEARCH OFFICE
UNIVERSITY OF AUCKLAND
PRIVATE BAG
AUCKLAND

APRIL 1988

MEASUREMENT:	Transmission loss of a Masonry wall construction.
STANDARDS FOLLOWED	ISO 140 parts 1, 2 and 3 and ISO 717 part 1.
PLACE OF MEASUREMENT:	Reverberation chambers C and A, Acoustics Research Centre, University of Auckland.
DESCRIPTION OF SAMPLE AND INSTALLATION:	The wall construction was built in the test opening between chambers C and A. Before construction a timber surround of 18 mm customwood was bolted over the gap between the chambers, of width 500 mm. The masonry wall was built on the chamber A side of the surround. The wall consisted of 13 courses of blockwork using 140 mm blocks ("15" series blocks). The wall was partially filled and reinforced using one D16 bar in the top and bottom course of the wall and 6 vertical D12 bars in every second column of blocks. (See figure 2).
RESULTS:	These are shown in figure 1 and the accompanying table.

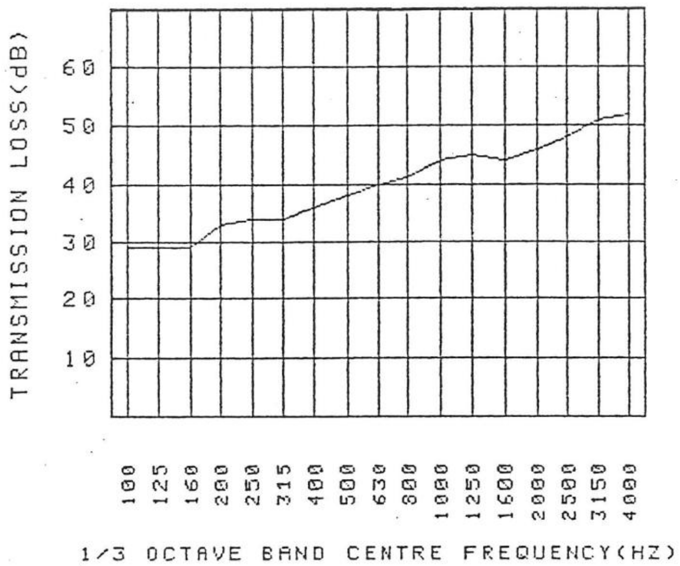
George Dodd
G. Dodd

Mark Poletti
M. Poletti

88T02

ACOUSTICS TESTING SERVICE
16-2-88

FIGURE 1: TRANSMISSION LOSS OF MASONRY WALL



1/3 OCTAVE CENTRE FREQUENCY (HZ)	TABLE 1 TRANSMISSION LOSS (dB)	STANDARD ERROR (dB)
100	29	1
125	29	1
160	29	1
200	33	1
250	34	<1
315	34	<1
400	36	<1
S00	38	<1
630	40	<1
800	41	<1
1000	44	<1
1250	45	<1
1600	44	<1
2000	46	<1
2500	48	<1
3150	51	<1
4000	52	<1

SIC = 42 Rw = 42

WALL AREA = 12.341056 M^2

RECEIVING ROOM = CHAMBER C VOLUME 209 M^3 AREA 214 M^2

METEOROLOGICAL CONDITIONS:

REVERBERATION MEASUREMENT = 24.5 DEGREES C 76.0 % RH 1000 MILLIBARS

LEVEL MEASUREMENT = 24.5 DEGREES C 76.0 % RH 1000 MILLIBARS

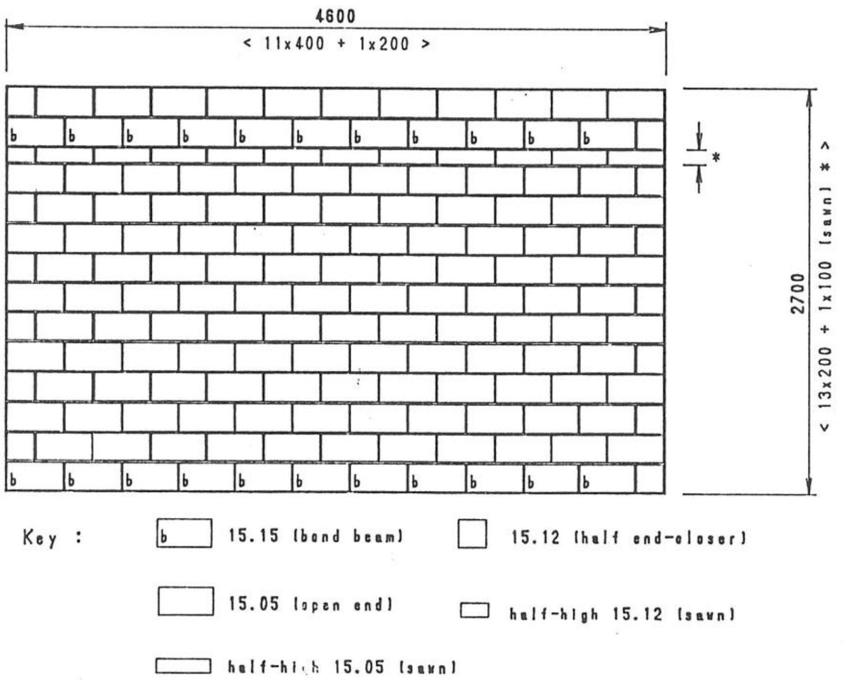
88T02

ACOUSTICS TESTING SERVICE
16-2-88

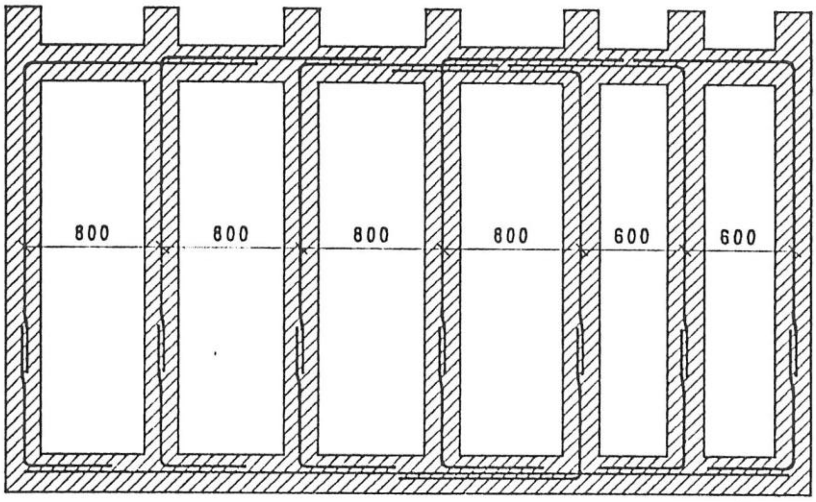
NZCMA ACOUSTICS TEST PROGRAMME (Feb. 1988)

15 SERIES WALL DETAILS

(a) Block Layout



(b) Grout/Reinforcement Profiles



All reinforcement Grade 275 MPa, D12

Grout proportioned to yield a standard compressive strength rating of between 17.5 - 20 MPa during the acoustic tests.

FIGURE 2

TRANSMISSION LOSS MEASUREMENTS OF A REINFORCED SOLID CORE BLOCK WALL

Report prepared for: Cement & Concrete Association
P.O. Box 47-277
Ponsonby
Auckland

Report prepared by: Acoustics Testing Service
Department of Architecture
THE UNIVERSITY OF AUCKLAND

AUCKLAND UNISERVICES LTD
THE UNIVERSITY OF AUCKLAND
PRIVATE BAG 92019
AUCKLAND

November 1993

Dr G. Dodd
Mr J. Irons
Acoustics Testing Service

REGISTERED LABORATORY

All tests reported herein have been performed in accordance with the laboratory's terms of registration

REGISTERED TESTING LABORATORY No. 268

MEASUREMENT:	Measurement by Laboratory Method of Transmission Loss of a reinforced solid core block wall.
STANDARDS FOLLOWED:	ISO 140/III.1978 ASTM E413-87
PLACE OF MEASUREMENT:	Reverberation Chambers A and C, Acoustics Research Centre, The University of Auckland.

INSTRUMENTATION:

EQUIPMENT	TYPE/SERIAL No.
Brüel & Kjær	
CHAMBER A EOUIPMENT	
1" Microphone	4145/1503121
Preamplifier	2619/945922
Power Supply	2807/888602
Rotating Boom	3923/936497
CHAMBER B EOUIPMENT	
1" Microphone	4145/1503121
Preamplifier	2619/945922
Power Supply	2807/888602
Rotating Boom	3923/936497
COMMON EQUIPMENT	
Noise Generator	1027/882108
Noise Generator	1402/352500
Measuring Amplifier	2636/936030
Filter Set	1617/887800
Pistonphone	4220/1048366
MEASUREMENT SOFTWARE	
"Reverb/SV2"	version 2.01
"Lread/SV2"	

NOTE: Where names are given in capitals in this report, they refer to proprietary products, which can be referred in appropriate product catalogues.

DESCRIPTION OF SAMPLES: A TRI-BOARD collar was installed in the chamber C part of the test corridor between chambers A and C. This collar was made of 50 mm TRI-BOARD, and was bolted in place, then sealed to the structure of chamber C with USG ACOUSTICAL SEALANT, giving a sealed edge flush with the chamber (ref ATS/ I302). A further TRI-BOARD collar of 43 mm thickness was then fitted in the chamber A side of the test corridor, bridging the separation between the two chambers. The two collars were vibration isolated from each other by means of an intervening layer of SELLOTAPE 3507 rubber tape which also sealed the gap between them.

SAMPLE No.1: On the collar in Chamber C, a concrete block wall was erected using type 20:16 blocks. The blocks were mortared with DRICON "TRADE" Mortar. The nominal weight of each block was 15 kg. The wall was constructed as a hollow core structure, with D12 reinforcing rods placed horizontally and vertically. When the wall was completed to within 60 mm of the top of the collar, grout was poured into the internal cavities, filling the existing structure to the top. To seal the wall, concrete capping slates were mortared and slipped into position on top of the blocks. See Drawings 1 and 2.

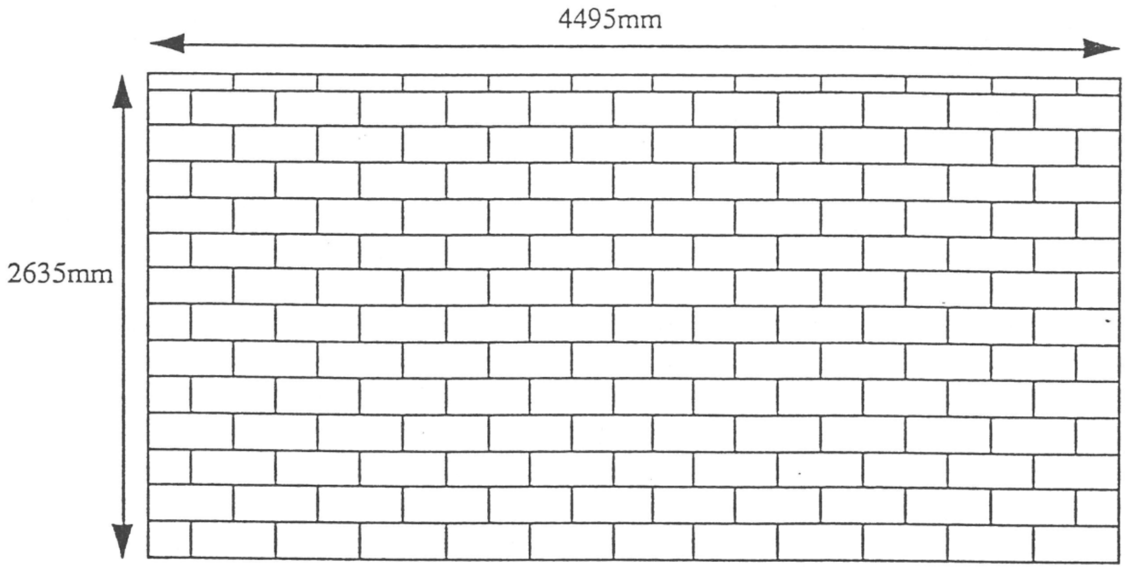
SAMPLE No. 2: The wall described above was painted on each face with two coats of acrylic paint to seal the faces.

Mr J. Irons
Testing Officer

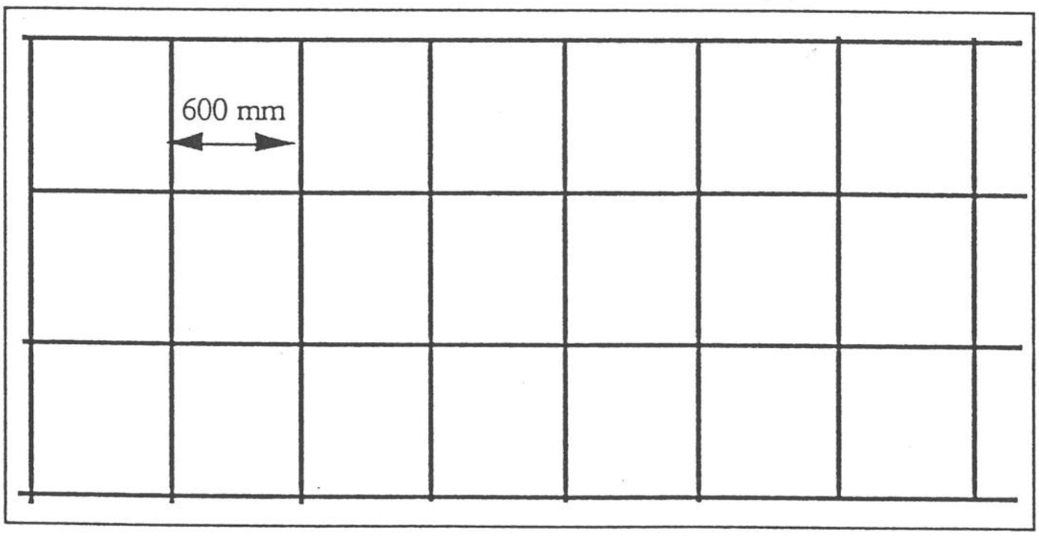
George Dodd

Dr G. Dodd
Head, ATS

DRAWING 1: BLOCK LAYOUT OF WALL



DRAWING 2: PLACEMENT OF D12 REINFORCING RODS IN WALL



DRAWING 3: SECTION OF WALL SHOWING BLOCK LAYOUT

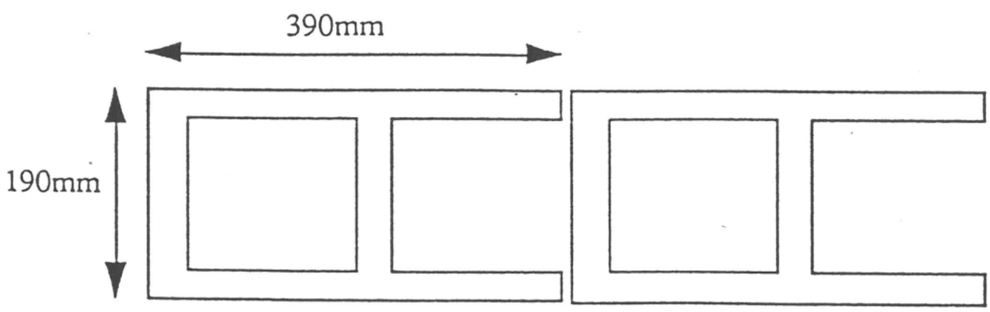


TABLE 1: Reverberation Times in Chamber A for bare block wall.

THIRD OCTAVE BAND CENTRE FREQUENCY (HZ)	R.T WITH SAMPLE (Seconds)	STANDARD ERROR
100	5.37	0.05
125	4.88	0.03
160	5.09	0.03
200	5.62	0.02
250	6.02	0.02
315	5.99	0.01
400	6.20	0.01
500	6.49	0.01
630	6.64	0.01
800	6.42	0.01
1000	5.99	0.04
1250	5.51	0.03
1600	5.02	0.02
2000	4.39	0.02
2500	3.85	0.02
3150	3.44	0.02
4000	2.98	0.01

METEOROLOGICAL CONDITIONS:

TEST ID	TEMPERATURE DEGREES C	RELATIVE HUMIDITY	ATMOSPHERIC PRESSURE (mB)
T334R01	20	90%	1011

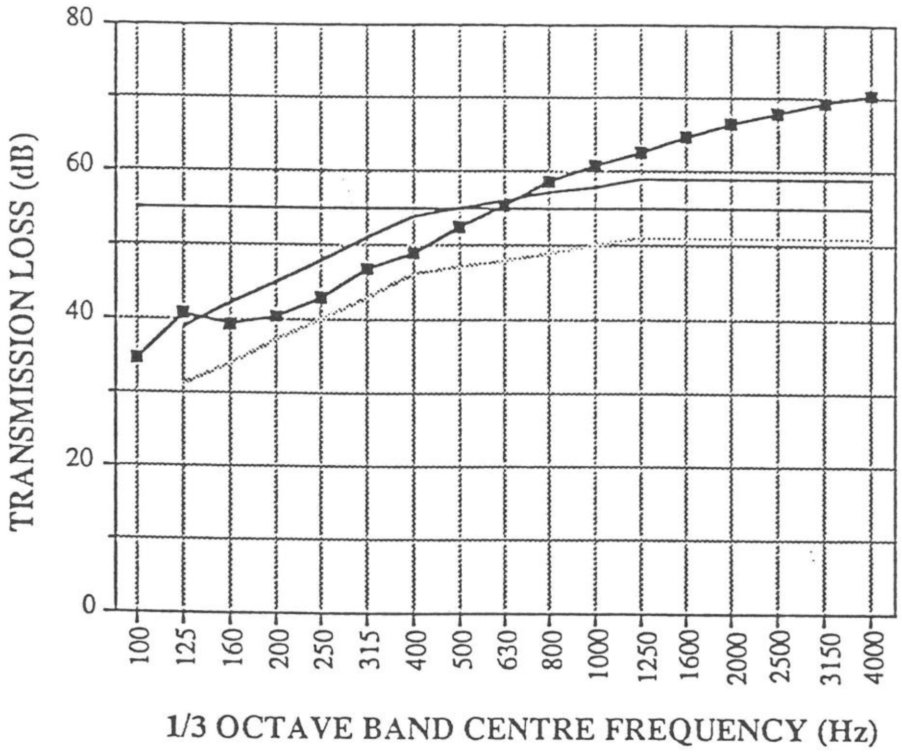
TABLE 2: Sound Pressure Levels measured for the determination of TL of bare block wall. Chamber C Source, Chamber A Receiving.

THIRD OCTAVE BAND CENTRE FREQUENCY (Hz)	LEVEL, (dB) SOURCE CHAMBER	STANDARD ERROR	LEVEL, (dB) RECEIVING CHAMBER	STANDARD ERROR
100	94.6	0.9	62.8	0.9
125	95.6	0.8	57.5	0.5
160	99.7	0.7	63.0	0.4
200	102.1	0.4	64.7	0.4
250	102.8	0.4	63.2	0.5
315	102.1	0.3	58.7	0.3
400	102.0	0.4	56.7	0.3
500	102.5	0.3	53.7	0.3
630	102.4	0.3	50.9	0.3
800	102.5	0.3	47.4	0.3
1000	101.1	0.3	43.7	0.3
1250	100.0	0.3	40.3	0.3
1600	98.6	0.3	36.7	0.3
2000	98.7	0.3	34.4	0.3
2500	98.2	0.3	31.8	0.3
3150	96.7	0.3	28.3	0.3
4000	97.2	0.4	27.0	0.2

METEOROLOGICAL CONDITIONS:

TEST ID	TEMPERATURE DEGREES C	RELATIVE HUMIDITY	ATMOSPHERIC PRESSURE (mB)
T334L01	20	90%	1010

GRAPH 1: Transmission Loss of Bare Block Wall



STC = 55
Rw = 55

TABLE 3: Transmission Loss of Bare Block Wall

THIRD OCTAVE BAND CENTRE FREQUENCY (Hz)	TRANSMISSION LOSS (dB)	STANDARD ERROR	DEVIATION BELOW RATED STC CURVE
100	35	1	-
125	41	1	0
160	39	1	3
200	40	1	5
250	43	1	5
315	47	<1	4
400	49	1	5
500	52	<1	3
630	55	<1	1
800	59	<1	0
1000	61	<1	0
1250	63	<1	0
1600	64	<1	0
2000	66	<1	0
2500	68	<1	0
3150	69	<1	0
4000	71	<1	0

SAMPLE AREA = 11.84 m^2
RECEIVING ROOM = CHAMBER A
VOLUME = 202 m^3 AREA = 203.6 m^2

Total Points Below STC Curve = 26

Table 4: Reverberation Times in Chamber A for painted block wall

THIRD OCTAVE BAND CENTRE FREQUENCY (Hz)	R.T. WITH SAMPLE (Seconds)	STANDARD ERROR
100	5.78	0.04
125	4.92	0.03
160	5.25	0.03
200	5.88	0.02
250	6.49	0.01
315	6.39	0.01
400	6.73	0.01
500	7.25	0.01
630	7.46	0.01
800	7.28	0.01
1000	6.90	0.06
1250	6.29	0.05
1600	5.65	0.02
2000	4.90	0.02
2500	4.22	0.02
3150	3.72	0.02
4000	3.19	0.01

METEOROLOGICAL CONDITIONS:

TEST ID	TEMPERATURE DEGREES C	RELATIVE HUMIDITY	ATMOSPHERIC PRESSURE (mB)
T334R02	20	67%	1014

Table 5: Sound Pressure Levels measured for the determination of TL of painted block wall. Chamber C Source, Chamber A Receiving.

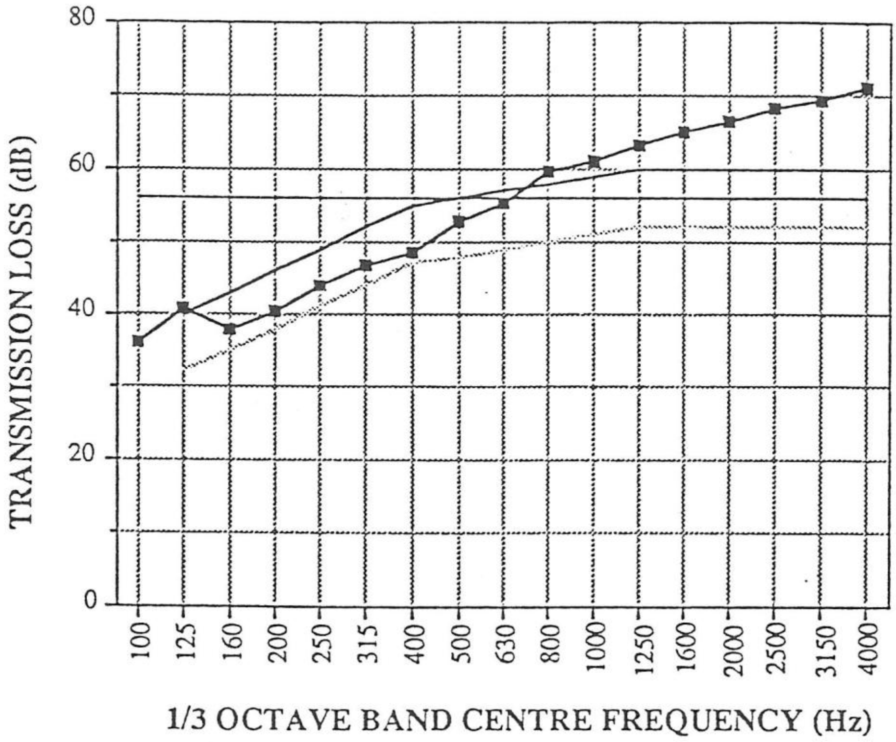
THIRD OCTAVE BAND CENTRE FREQUENCY (Hz)	LEVEL, (dB) SOURCE CHAMBER	STANDARD ERROR	LEVEL, (dB) RECEIVING CHAMBER	STANDARD ERROR
100	97.1	0.8	64.2	0.7
125	96.4	0.7	58.3	0.5
160	100.1	0.4	64.9	0.5
200	102.5	0.4	65.4	0.5
250	104.0	0.4	63.6	0.4
315	103.2	0.4	60.1	0.3
400	102.7	0.3	57.8	0.4
500	103.4	0.3	54.8	0.4
630	103.3	0.3	52.1	0.3
800	103.6	0.3	48.3	0.3
1000	102.1	0.3	44.8	0.3
1250	101.4	0.3	41.7	0.3
1600	99.6	0.3	37.7	0.3
2000	99.6	0.3	35.5	0.3
2500	99.1	0.4	32.7	0.3
3150	97.2	0.3	29.3	0.4
4000	97.3	0.4	26.8	0.2

METEOROLOGICAL CONDITIONS:

TEST ID	TEMPERATURE DEGREES C	RELATIVE HUMIDITY	ATMOSPHERIC PRESSURE (mB)
T334L02	19	66%	1013

PROJECT T334 PAGE 8 OF 12

Graph 2: Transmission Loss of Painted Block Wall.



STC = 56
Rw = 55

Table 6: Transmission Loss of Painted Block Wall.

THIRD OCTAVE BAND CENTRE FREQUENCY (Hz)	TRANSMISSION LOSS (dB)	STANDARD ERROR	DEVIATION BELOW RATED STC CURVE
125	41	1	0
160	38	1	5
200	40	1	6
250	44	1	5
315	47	1	5
400	49	1	6
500	53	1	3
630	55	<1	2
800	59	<1	0
1000	61	<1	0
1250	63	<1	0
1600	65	<1	0
2000	67	<1	0
2500	68	1	0
3150	69	1	0
4000	71	<1	0

SAMPLE AREA = 11.84 m^2
RECEIVING ROOM = CHAMBER A
VOLUME = 202 m^3 AREA = 203.6 m^2

Total Points Below STC Curve = 32

3.1 Basic Masonry Construction

Introduction

The design of reinforced concrete masonry walls is controlled primarily by:

- NZS 4230 *Design of reinforced concrete masonry structures;*
- NZS 4229 *Concrete masonry buildings not requiring specific engineering design;*
- and some further non-specific designs set out in NZS 3604 *Timber Framed Buildings.*

All these listed standards require Materials and Workmanship to follow the provisions of NZS 4210:2001 *Masonry construction: Materials and workmanship.*

The principal contents of Section 3 of the Masonry Manual relate to the provisions of NZS 4210 concerning construction, reinforcement and grouting. Information has been provided on manufacture of masonry, basic blocklaying and mortar joints in Section 1.

Section 4 will deal with aspects of design.

Workmanship and Supervision

It is important to recognise that the Design Standard NZS 4230 places a significant importance to the overseeing of construction work under the term ‘observation’.

It outlines three types of observation – A, B and C – and designates permitted design strengths to each observation level. The full specification details and contained in Table 3.1 of NZS 4320.

A summary is as follows:

Observation		Nominal Design MPa
C	No construction observation by design engineer	4
B	Inspection by design engineer or mason deemed to meet the competenacy requirements of NZS 4210	12
A	Additional levels of observation, see NZS 4230	>12

The implications of this were that Non-specific engineering design details of NZS 3604 are based on 4 MPa, and NZS 4229 are based on 8 MPa (but will be raised to 12 MPa at next amendment). Most specific engineered designed reinforced concrete masonry will be Type B.

The important correlation is that the New Zealand Masonry Trades Registration Board had examined masons in their competency understanding of NZS 4210 and hence using a Registered Mason for construction met the requirements of Type B observation.

At the time of writing, the Department of Building and Housing (DBH) is introducing the Licensing Building Practitioner (LBP) scheme wherein the competency check on the structural mason will be done by DBH (see Section 1.6).

In effect, a LBP registered in *Bricklaying and Blocklaying 2: Structural Masonry* will be the equivalent envisaged by NZS 4230 for a registered mason under the previous arrangement.

The DBH is introducing the term ‘Restricted Work’ but for the purposes of this industry section it will mean all structural masonry elements contained in NZS 3604, NZS 4229 and NZS 4230.

Blocklaying Tolerances

While the full list of tolerances is contained in NZS 4210, Table 2.2, the principal values are set out here:

1. Deviation from position — 15 mm
2. Vertical plumbness — 10 mm in 3 metres
3. Total verticality in building height — 20 mm
4. Deviation from plan line under 10 m in length — 5 mm
5. Displacement in vertical alignment between load bearing walls — 5 mm
6. Displacement in vertical alignment between masonry courses:
   (a) Nominated fair face (one side only) — 3 mm
   (b) Structural face — 5 mm

Other tolerances in joints are discussed in Section 1, Part 1.5.

It is worth pointing out that in many of the architectural applications, a tighter tolerance is desirable than the 3 mm specified in NZS 4210.

For such applications, the project specification needs to override NZS 4210. Typical value would be 1-1.5 mm.

Grout Specification

There are two types of grout specified in NZS 4210, one for use in standard concrete masonry and one for use mainly in between skins of brick or blockwork.

The coarse grout specification can be summarised as a blend of concreting sand, 13.2-4.75 coarse aggregate (approximately 50% of sand quantity) and Portland cement.

The basic strength requirement is 17.5 MPa at 28 days but where construction is exposed and in Zone C (NZS 3604), 20 MPa strength is specified. Zone D (NZS 3604) require 25 MPa strength.

Currently NZS 4210 has a different coastal durability designation where in 25 MPa is the default value where 17.5 MPa is not acceptable.

The workability of the grout is measured by the spread test (see NZS 3112, Part 1, Section 11). The spread is required to be 450 mm to 530 mm for the grout to comply with NZS 4210. Refer to CCANZ Information Bulletin 50 *Spread Test* found at the end of this section.

It is strongly recommended that the gout should be specified to include an expansive admixture as defined in NZS 4210, clause 2.8.2.1(c).

The expanding admixture is to have 2-4% expansive properties occurring within four hours of dosing.

While the supply of grout will normally be via the ready mixed concrete industry via the basic specification, i.e. 17 MPa (or 25 MPa) at 28 days with spread value of 450-530 mm, the dosage of the expanding admixture must be done at site immediately prior to discharge into the masonry wall.

It should NOT be dosed at the ready mixed concrete plant.

The two photographs demonstrate the importance of using the expanding agent in terms of achieving a homogeneous infill.

Figure 1 shows a cross-section through a section taken from a full height wall using the admixture.

The top reinforcement is fully encapsulated and face shells remain bonded to the core.

Figure 2 shows a similar specimen but with no expanding admixture used. Air gap is visible under the steel and face shells have been lost.

Figure 1

Figure 2

The other important reason to use the expanding admixture relates to the method of filling and consolidation of the grout in the wall. This will be described in Section 3.3.

Testing Requirements

Where buildings have specific engineering design it will be usual to require the following tests to be executed and/or results supplied:

1. Compressive strength of grout.
2. Spread of grout.

The compressive strength of grout will often be available from the ready mixed concrete supplier. However, if grout is site tested as described in NZS 4210, it must be sampled BEFORE any expanding admixture is added.

Other testing mentioned in NZS 4210 relates to mortar compressive and bond strengths which generally are of little significance to the overall performance of the reinforced masonry.

The expanding admixture testing has been superseded by manufacture performance criteria, i.e. the product is produced and warranted to give 2-4% expansion usually within some temperature limits.

Bond strengths are a critical requirement of veneer performance and are discussed in Section 5.

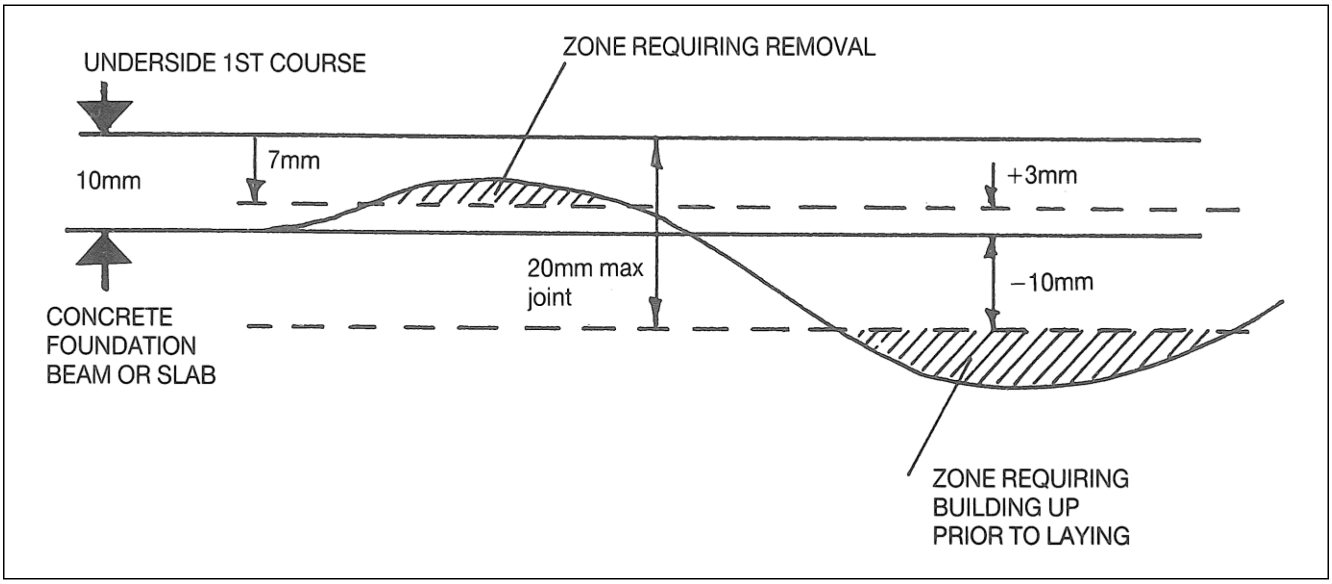


Figure 3

For non-specific engineering construction, the mason needs to retain records of the supply of materials and any supplier tests available.

Concrete Foundations

Concrete foundations to receive a masonry wall must comply with the various design provisions from NZS 4230, NZS 4229 and NZS 3402.

Their construction, materials and workmanship are controlled by NZS 3109. As discussed in Section 1.4 *Mortar and Mortar Joints* the foundation surface level must be with the tolerances shown in Figure 3.

The surface should be cleaned prior to laying the first course of blocks (see Figure 4).

This should be the work of the foundation contractor as it is more easily done while the concrete is still green.

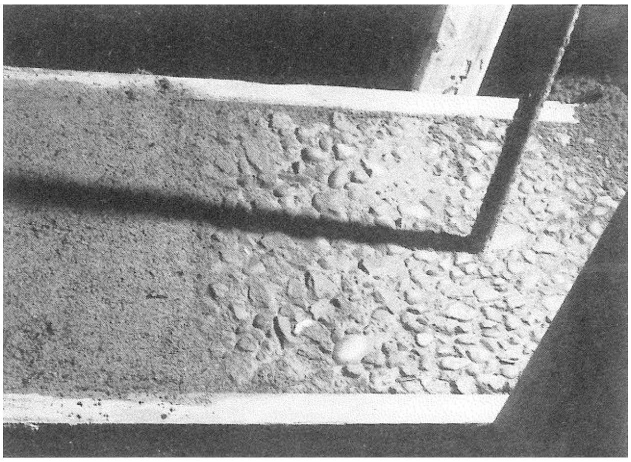


Figure 4

The first course should not overhand the supporting foundation by more than 20 mm. However, structural alignment restrictions may well rule on the issue in which case the overhang may have to be limited to 5 mm.

Reinforcing starter bars need to be positioned to suit the masonry block modules. Refer to CCANZ Information Bulletin 47 found at the end of this section.

Figure 5 (page 4) illustrates a typical wall elevation showing that starter bars are required at the jambs of windows and door openings which may not be on the regular spacing grid, i.e. the wall elevation drawing is needed before the starter bar positioning in the foundation can be decided.

The position of the starter bars needs to be checked immediately prior to laying the first course.

Where bars are missing or in the wrong position, additional steel will need to be inserted. S cranking of out of place steel is not permitted (see Figure 6, page 4).

The remedial work will require the specific permission of the designer.

The steel fixing tolerances are discussed in Section 3.2.

Intersecting Partition Walls

For tying partition walls to other walls, strips of galvanised metal bonding, metal lath or 6 mm mesh galvanised netting are placed across the joint between the two walls.

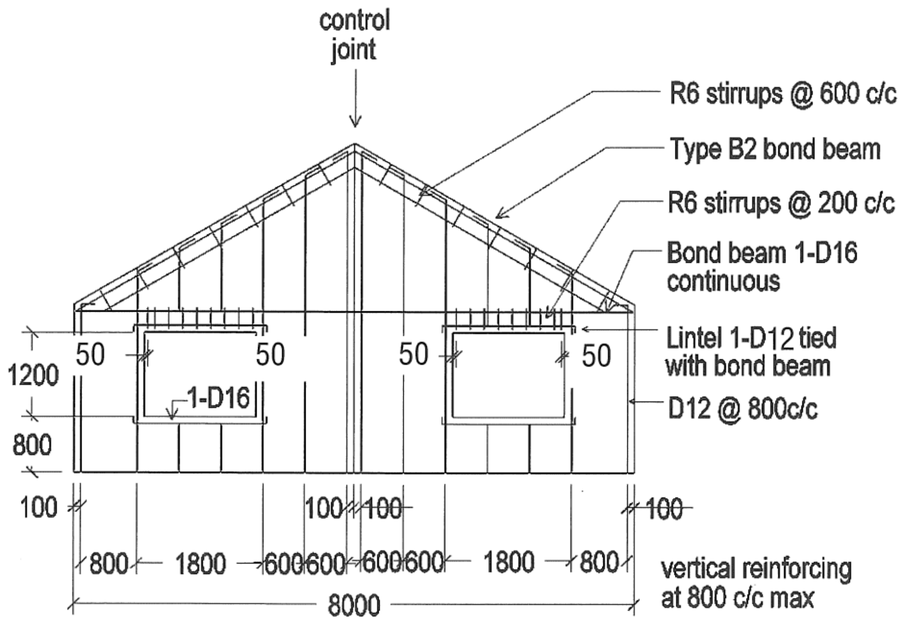


Figure 5

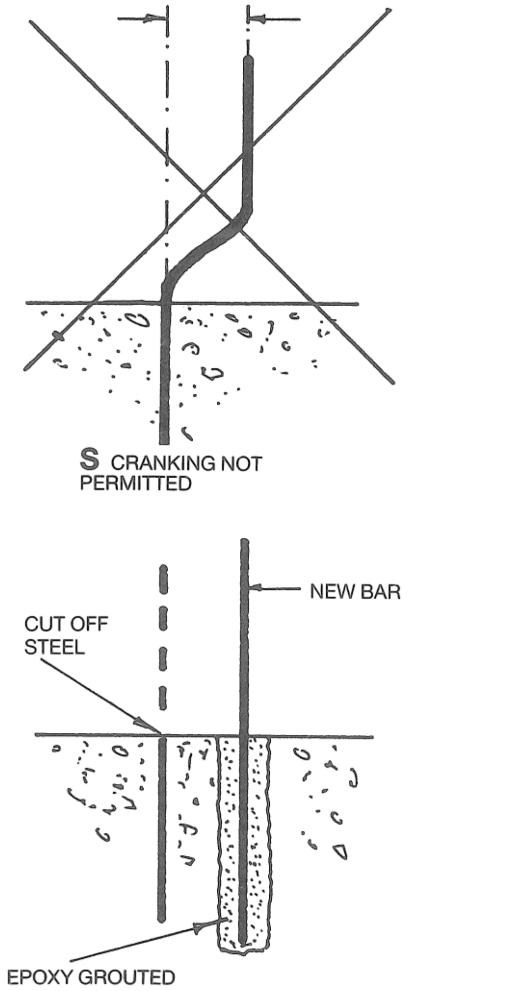


Figure 6

The metal strips are placed in alternative courses in the walls.

When one wall is constructed first, the metal strips are built into the wall and later tied into the mortar joint of the second wall. See Figure 7.

Intersecting Masonry Walls

Intersecting concrete masonry walls may be bonded together as indicated in the accompanying details, provided the intersecting wall is not longer than 1.2 m or has been structurally considered and specifically designed.

It is more common practice for the intersecting wall to terminate at the face of the other wall with a control joint in the intersecting wall approximately 1.2 m from the junction.

For lateral support, bearing walls are tied together by reinforced bond beams. See Figures 8 and 9.

In minor structures where there might not be an intermediate bond-beam, it is recommended that the intersecting walls be tied together with galvanised metal bonding or lath or netting as specified above for intersecting partition walls.

If the joint at the intersection of the two bearing walls is to be exposed to the weather, it should be constructed and sealed with a caulking compound as described under Control joints.

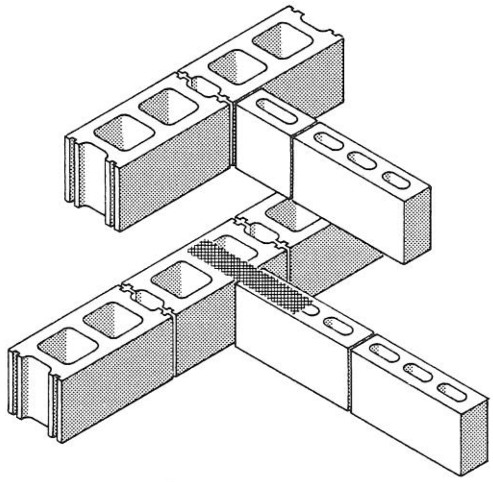


Figure 7: Intersecting Partition Walls

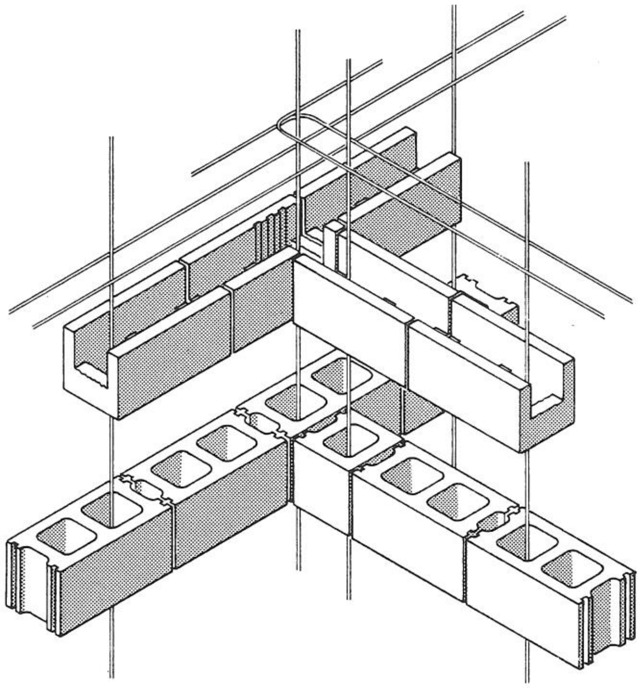


Figure 8: Intersecting Masonry Walls

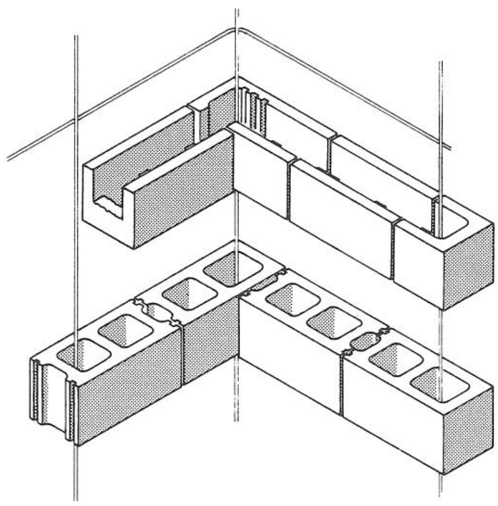
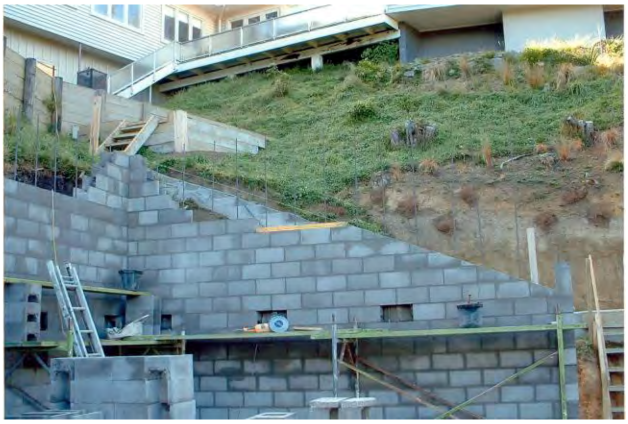


Figure 9: Intersecting Masonry Walls




Bonding Details

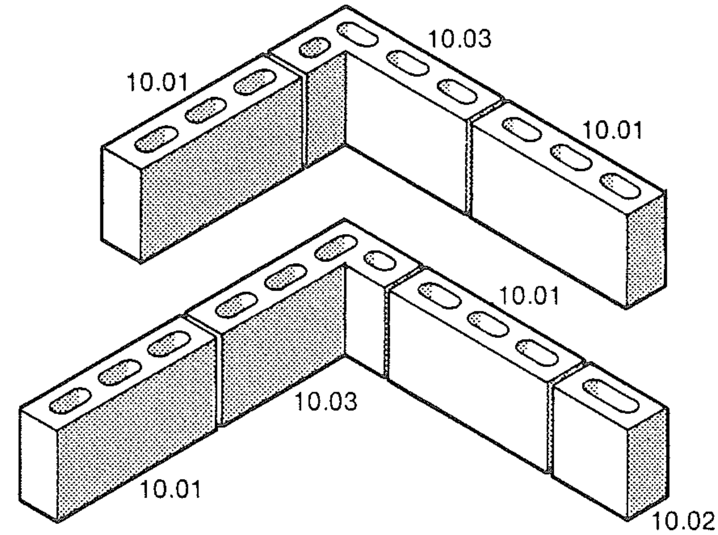
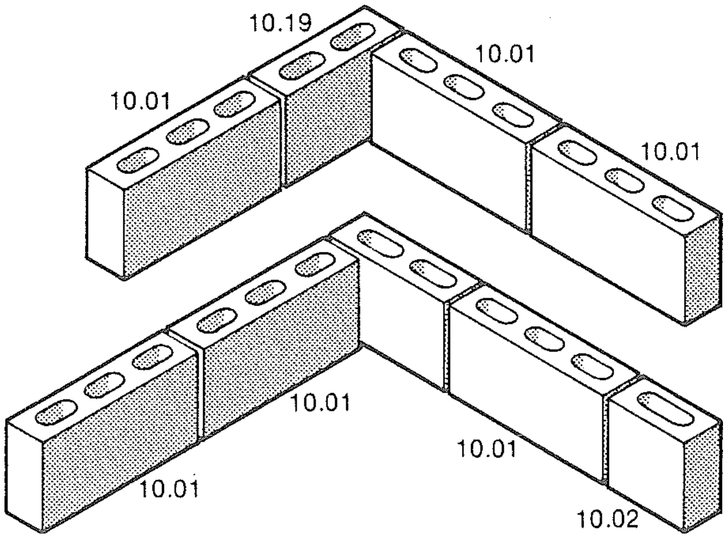
The following diagrams (page 7-17) illustrate the use of standard blocks in constructing corners, columns, cavity walls and intersections where walls of the same or different thicknesses meet. The illustrations are not intended as constructional details in themselves; rather they are guides to how construction details can be developed with standard modular units.

There might be some structural considerations that would suggest that some corners should be un-bonded, perhaps more so in cases where the walls' thicknesses are markedly different (e.g. 10 to 25 series). Such corners would have to be specially considered and designed.

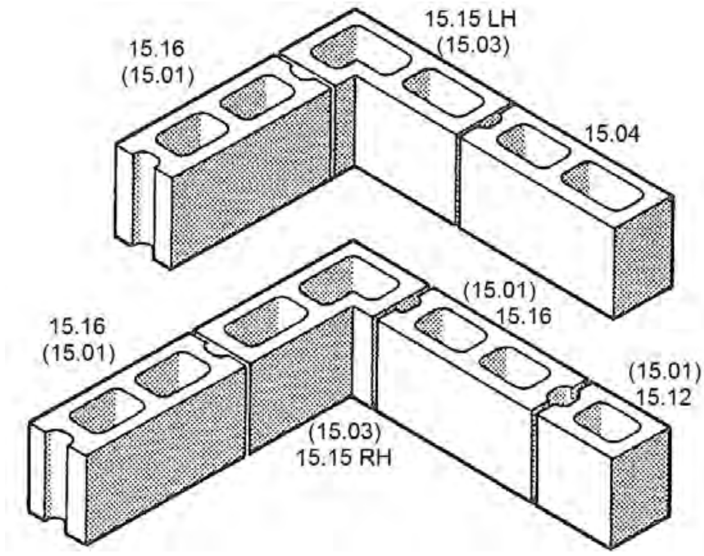
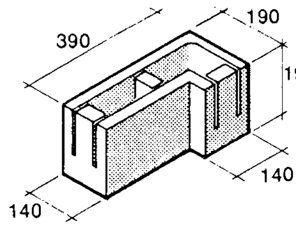
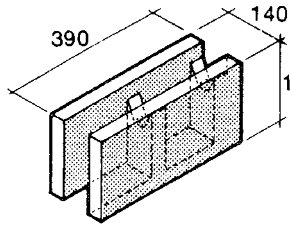
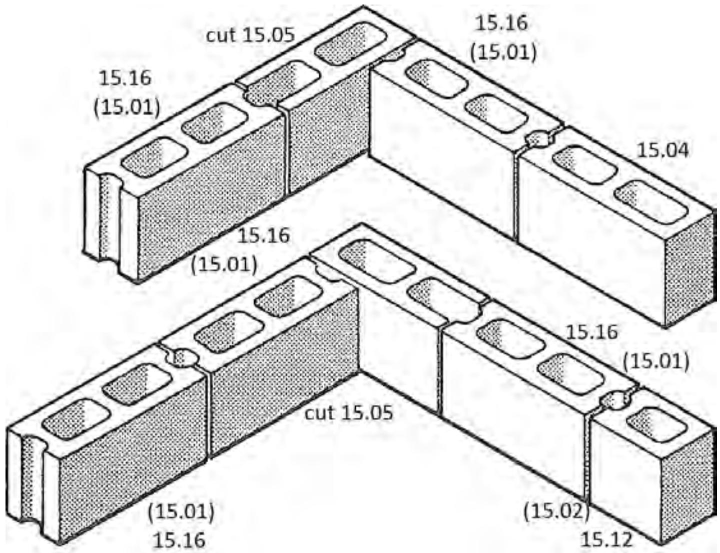
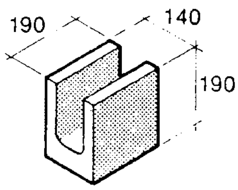
In most cases, except for when the 10 series is used as a masonry veneer supported by either a structural masonry or timber wall, there will be a need to tie intersections with horizontal reinforcement. Where this is required, it will be necessary to substitute the referenced block with a block designed to accommodate horizontal reinforcement using either a depressed web or knock out web section. For example, a 15.01 could be replaced by a 15.14 or 15.16, a 20.01 replaced by 20.14 or 20.16, a 25.01 replaced by a 25.14 or 25.16 and a 30.04 replaced by a 30.14.

In some cases, there is no direct substitute block and it will be necessary to modify the referenced blocks by masonry saw cutting prior to laying to create a horizontal pathway for the reinforcement.

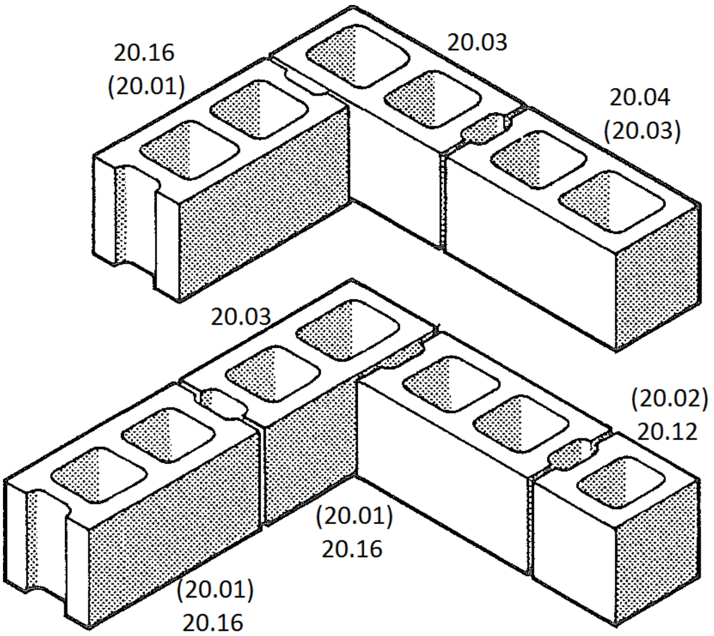
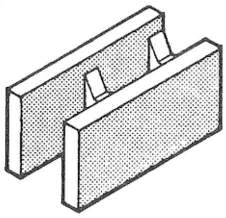
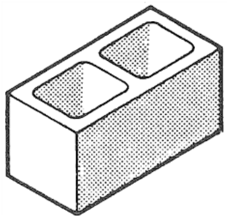
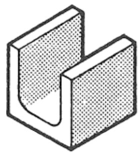
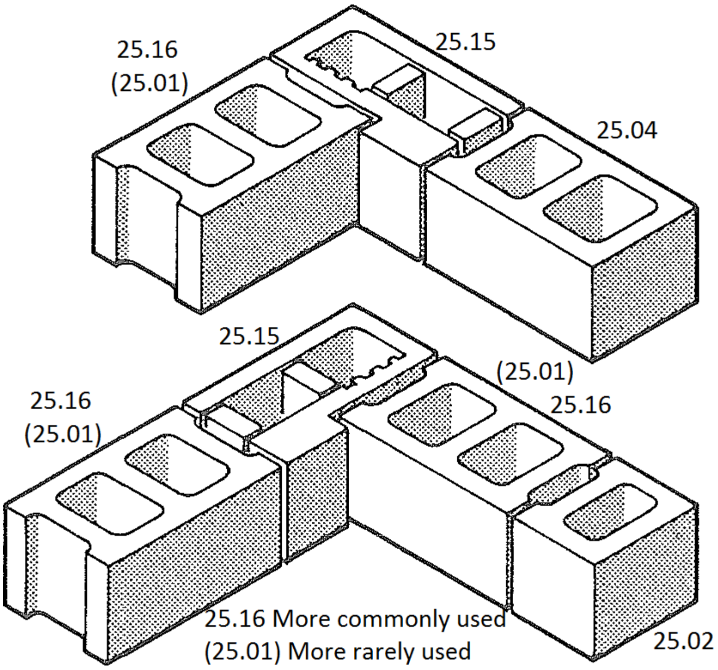
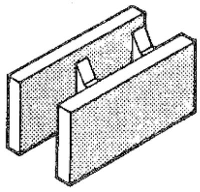
10 Series Corners

	200 mm module on outer face	200 mm module on inner face	Cores in line	Cores not in line	Common Blocks in Use and Other Remarks
Normal System 	•			•	Use of two cores units or of 10.05 units instead of 10.01 units would align cores.
		•		•	Use of two cores units or of 10.05 units instead of 10.01 units would align cores. Recommended for use only where 200 mm module required on inner face. Also recommended bonding for half-high split-face and similar units.

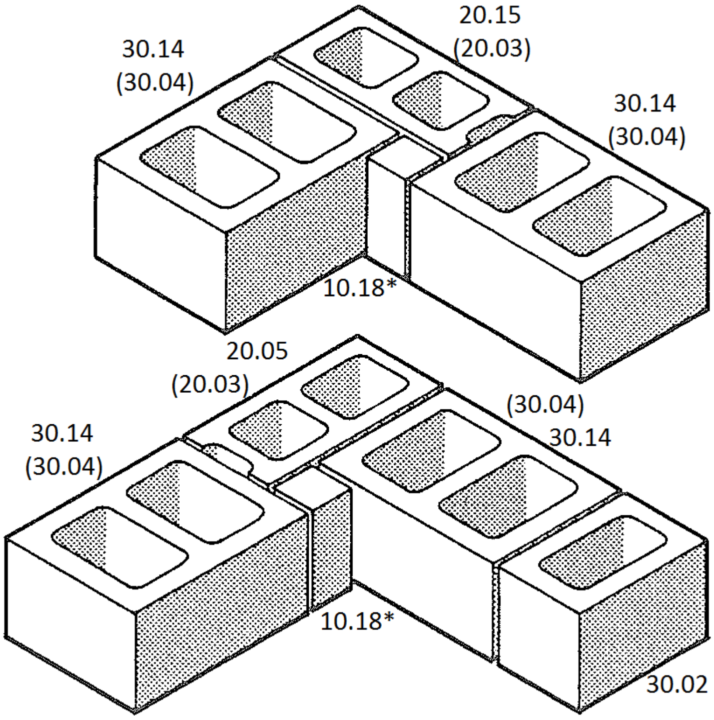
15 Series Corners

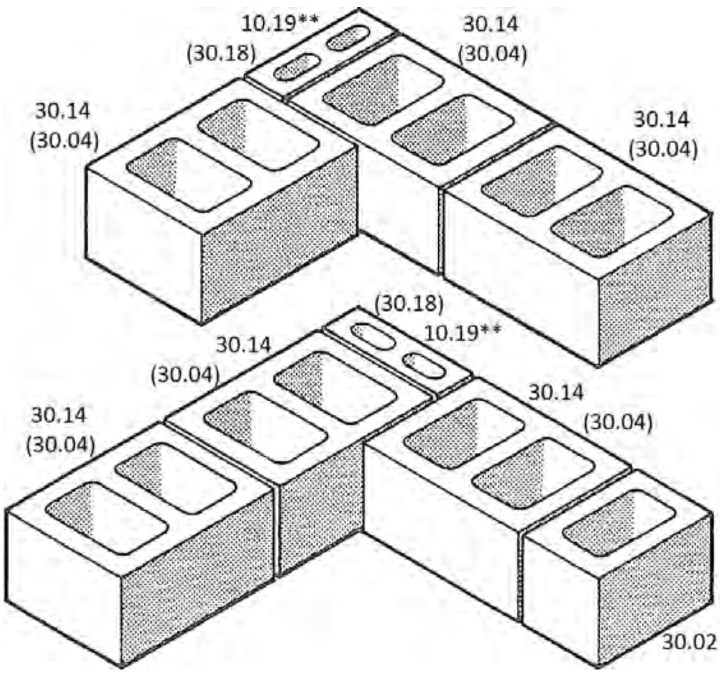
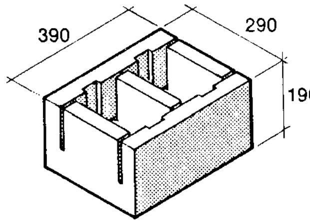
	200 mm module on outer face	200 mm module on inner face	Cores in line	Cores not in line	Common Blocks in Use and Other Remarks
	•		•		15.15 RH 15.16
		•	•		The corner blocks are 15.05 units cut on a table saw to 340 mm actual length. Recommended for use only where 200 mm module required on inner face. 15.12 ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

20 and 25 Series Corners

	200 mm module on outer face	200 mm module on inner face	Cores in line	Cores not in line	Common Blocks in Use and Other Remarks
20 Series Corner 	•	•	•		 20.16  20.04  20.12
25 Series Corner 	•		•		 25.16 ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

30 Series Corners

<table>
<tr><th></th><th>200 mm module on outer face</th><th>200 mm module on inner face</th><th>Cores in line</th><th>Cores not in line</th><th>Common Blocks in Use and Other Remarks</th></tr>
<tr><td>
</td><td>•</td><td></td><td>•</td><td></td><td>* 10.18 could be cut from 10.17 solid

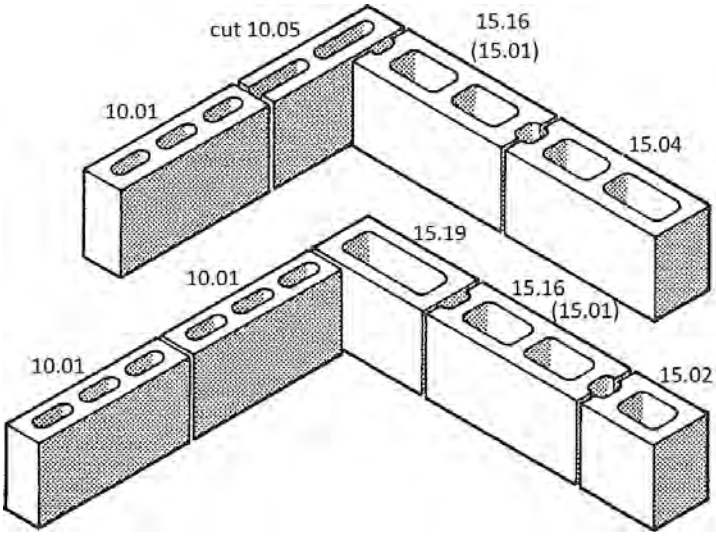
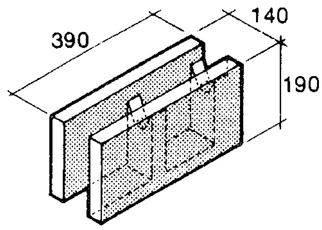
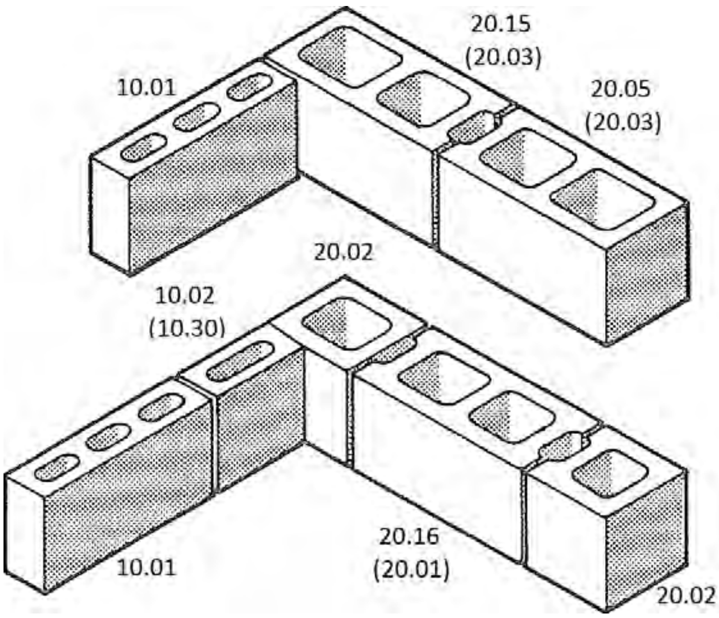
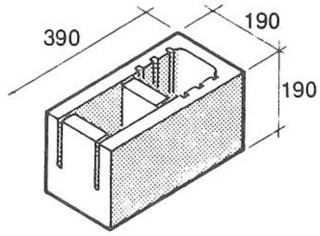
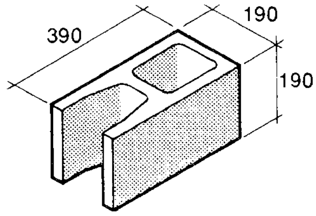
** 10.19 cut from 10.01.</td></tr>
<tr><td>
</td><td></td><td>•</td><td>•</td><td></td><td>Recommended for use only when 200 mm module required on inner face.


30.14

30.18 is also known as 10.19.

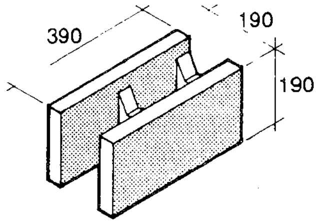
Note: Modifications to standard blocks can also be made by blocklayer using a masonry saw.</td></tr>
</table>

10/15 and 10/20 Series Corners

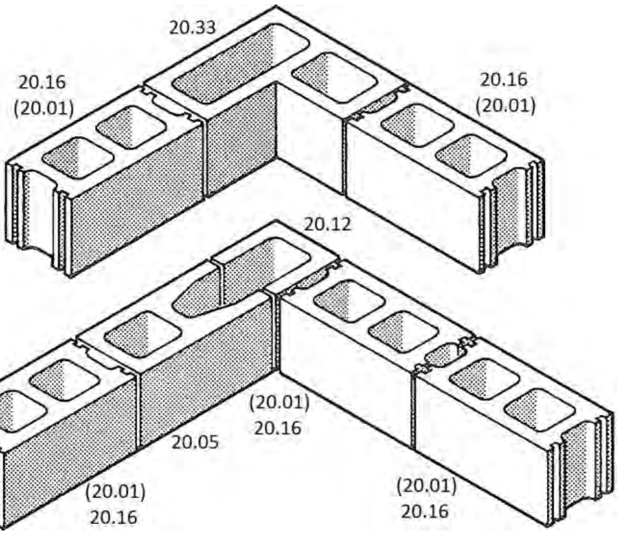
	200 mm module on outer face	200 mm module on inner face	Cores in line	Cores not in line	Common Blocks in Use and Other Remarks
10/15 Series Corner  		•		•	10.05 unit cut to 340 mm actual length. Use of two cores units or of 10.05 units instead of 10.01 units would align cores.  15.16
10/20 Series Corner  	•			•	Use of two cores units or of 10.05 units instead of 10.01 units would align cores.  20.15  20.05 ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

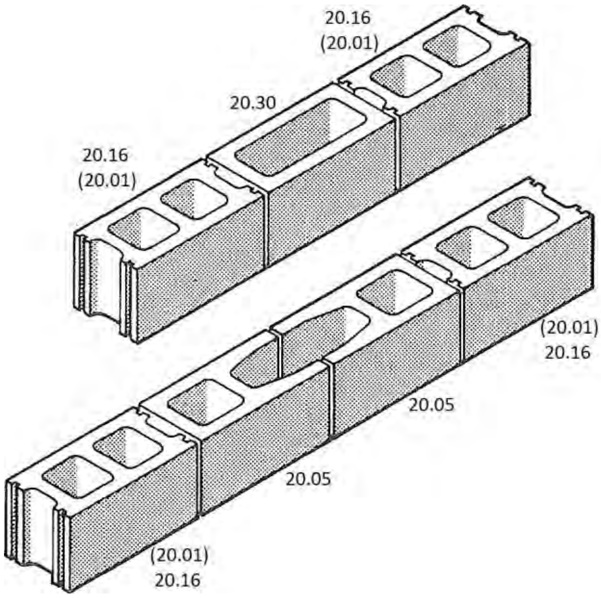
Flush Columns in 20 Series Walls

Common Blocks in Use and Other Remarks

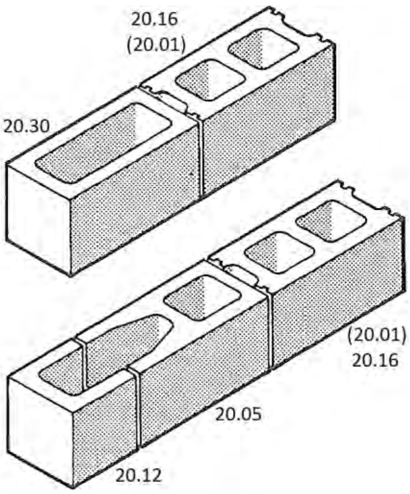


20.16



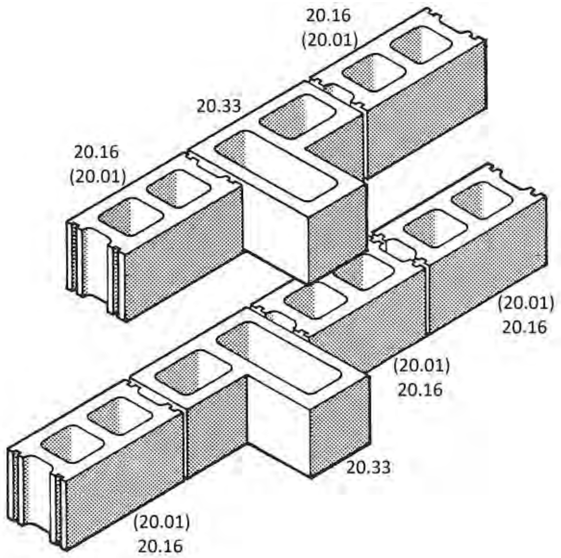
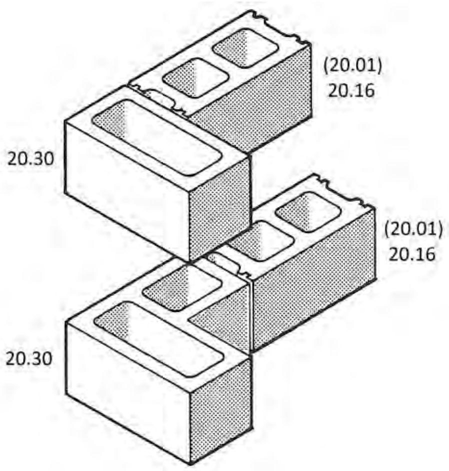
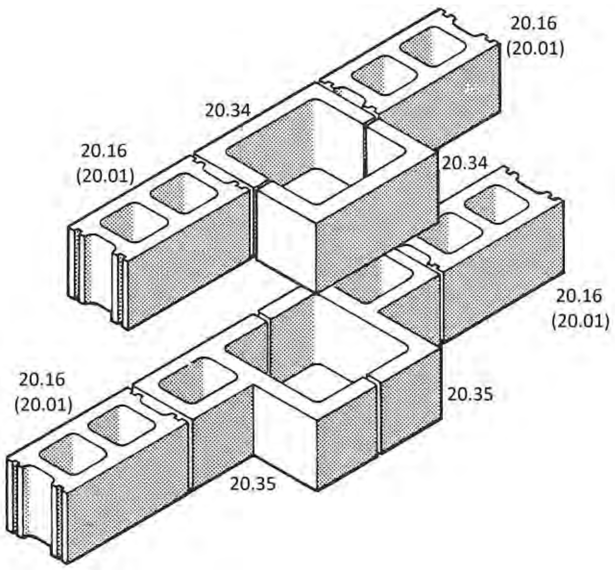
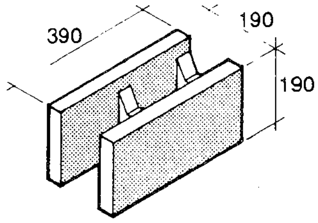


Bonded intersecting walls should not exceed 1.2 m in length unless specially designed.

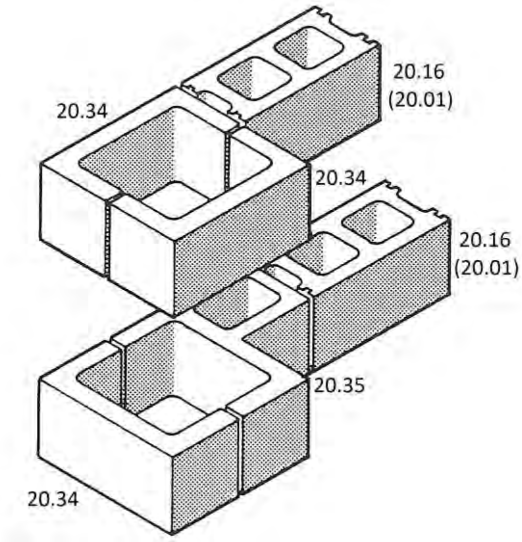
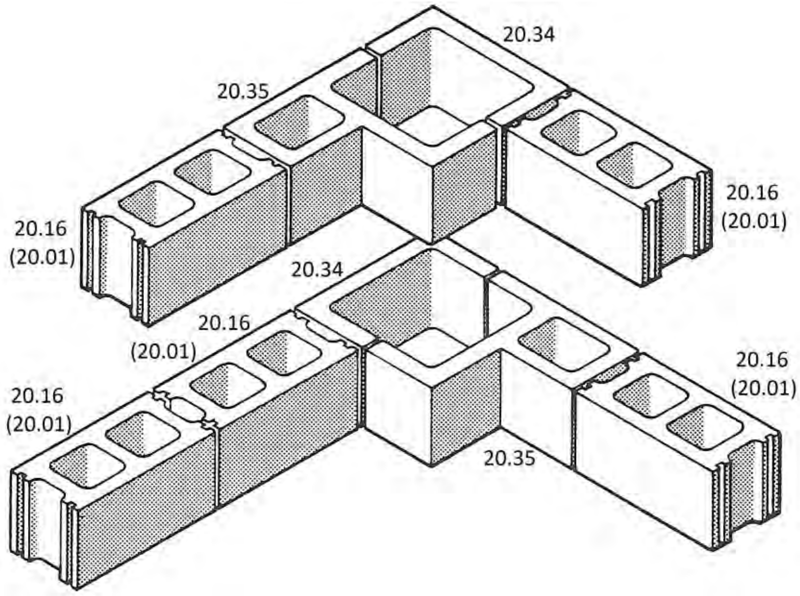
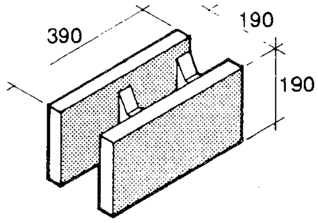


Note: *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

Columns in 20 Series Walls

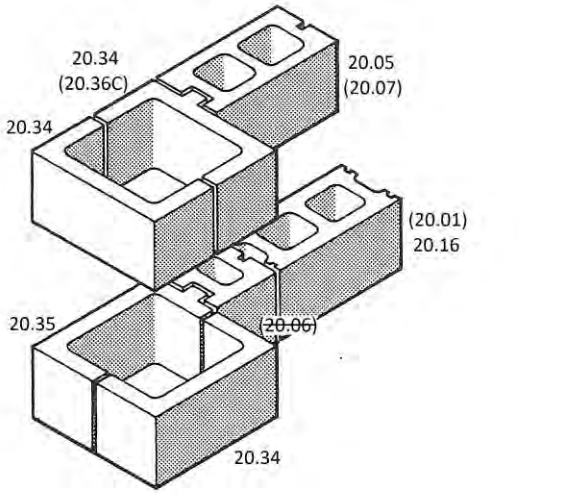
	Common Blocks in Use and Other Remarks
  	 20.16 ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

Columns in 20 Series Walls continued

	Common Blocks in Use and Other Remarks
 	 20.16 ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

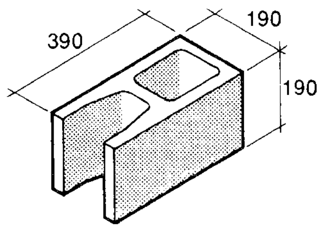
Columns in 20 Series Walls continued

Common Blocks in Use and Other Remarks

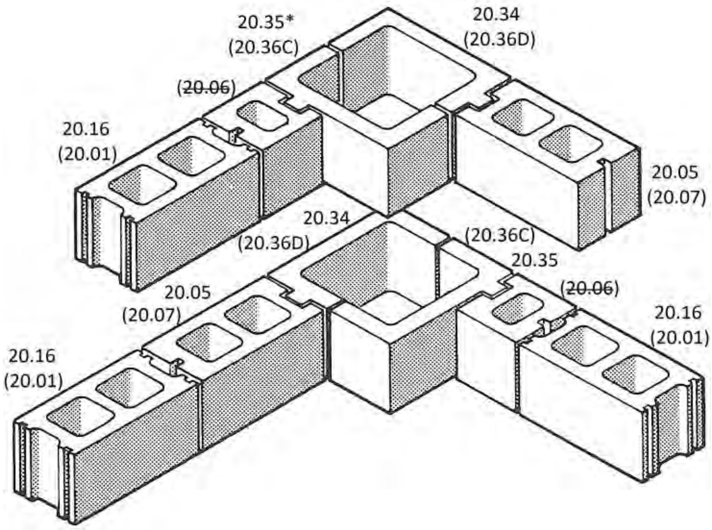


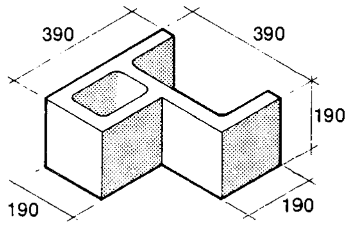
20.36 is superseded by 20.34.

20.07 is superseded by 20.05.



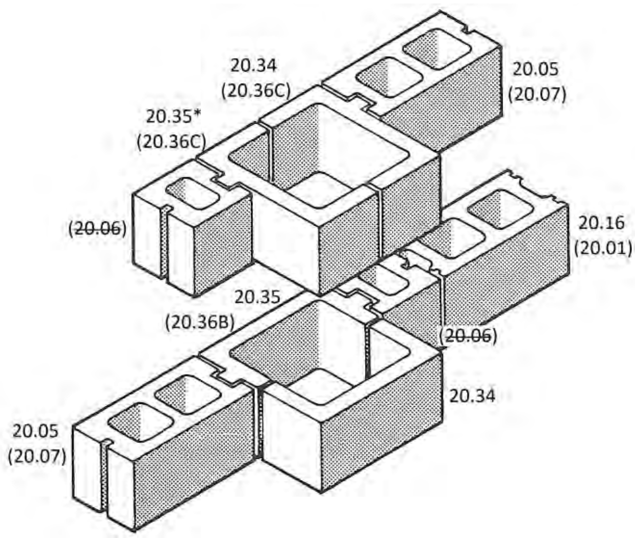
20.05





20.35

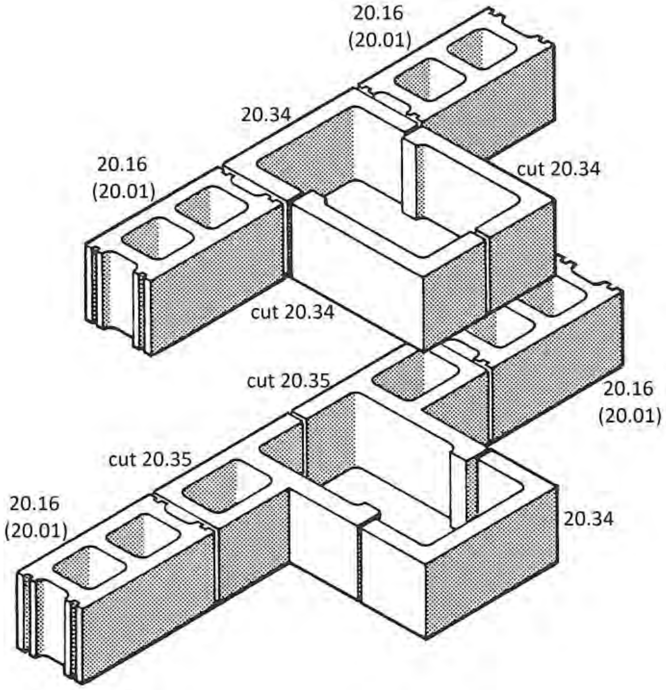
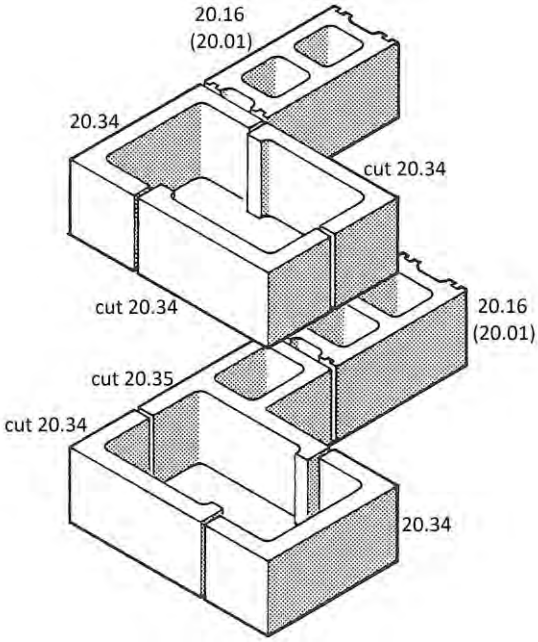
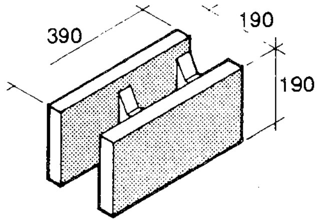
* 20.35 is a composite of 20.36 and 20.06.



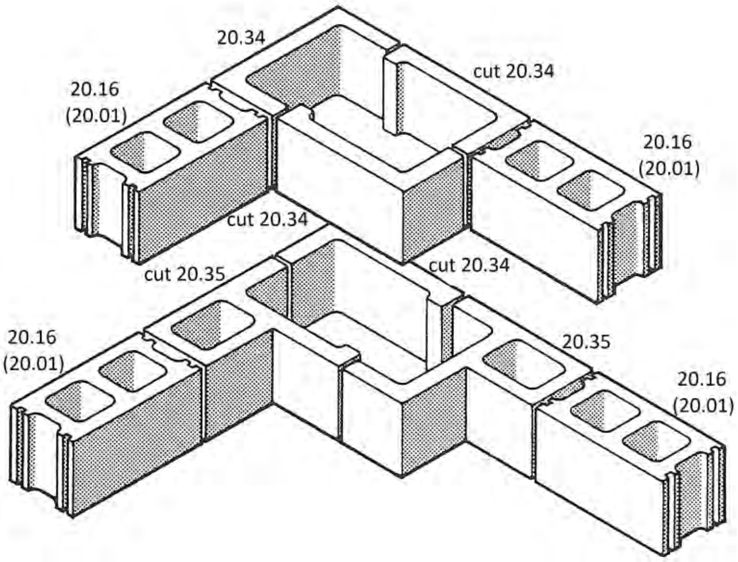
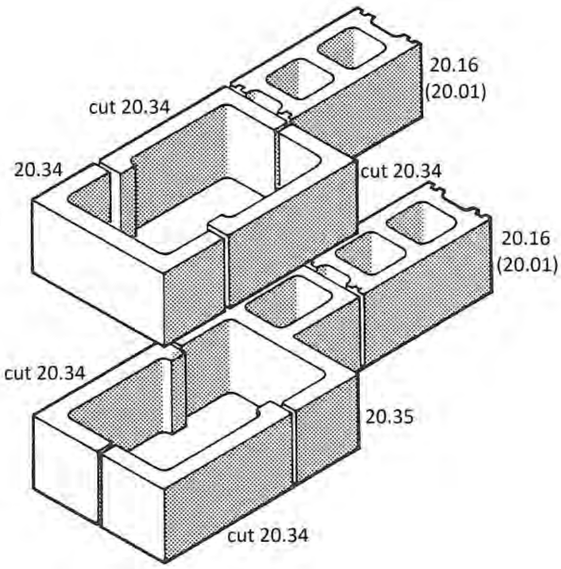
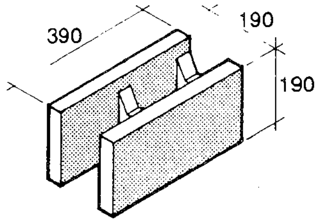
These column details facilitate control joints alongside the columns.

Note: *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

Columns in 20 Series Walls continued

	Common Blocks in Use and Other Remarks
 	 The shell faces of these column units must be supported by strapping and strutting during placing of grout filling and until grout has set. ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

Columns in 20 Series Walls continued

	Common Blocks in Use and Other Remarks
 	 20.16 The shell faces of these column units must be supported by strapping and strutting during placing of grout filling and until grout has set. ***Note:*** *Modifications to standard blocks can also be made by blocklayer using a masonry saw.*

CCANZ
concrete answers

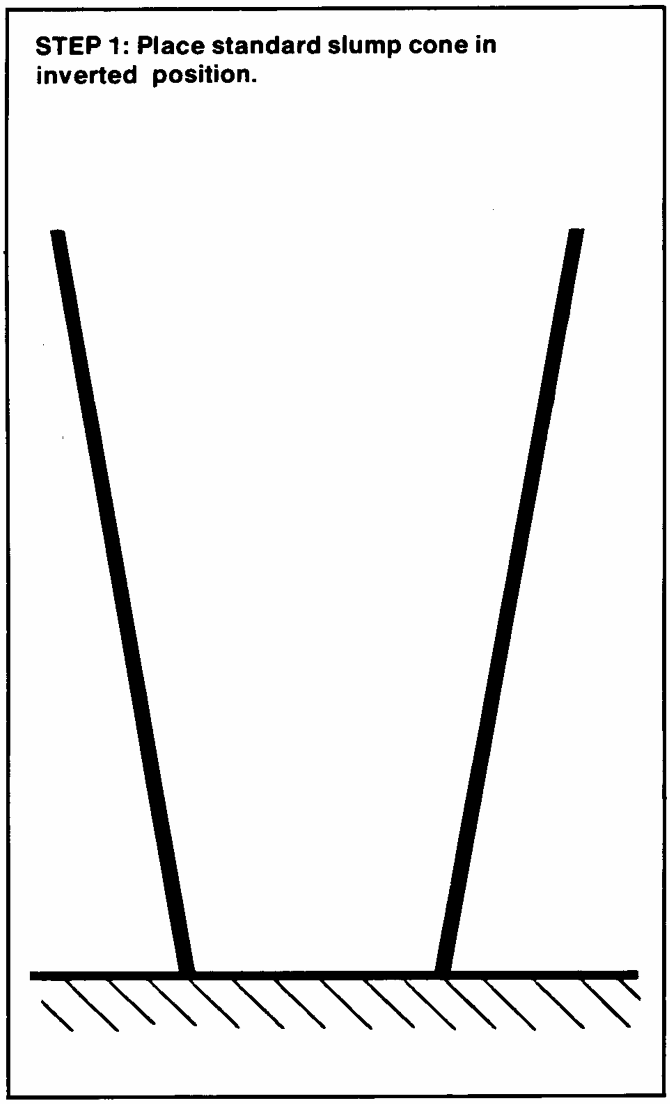
INFORMATION BULLETIN: IB 50

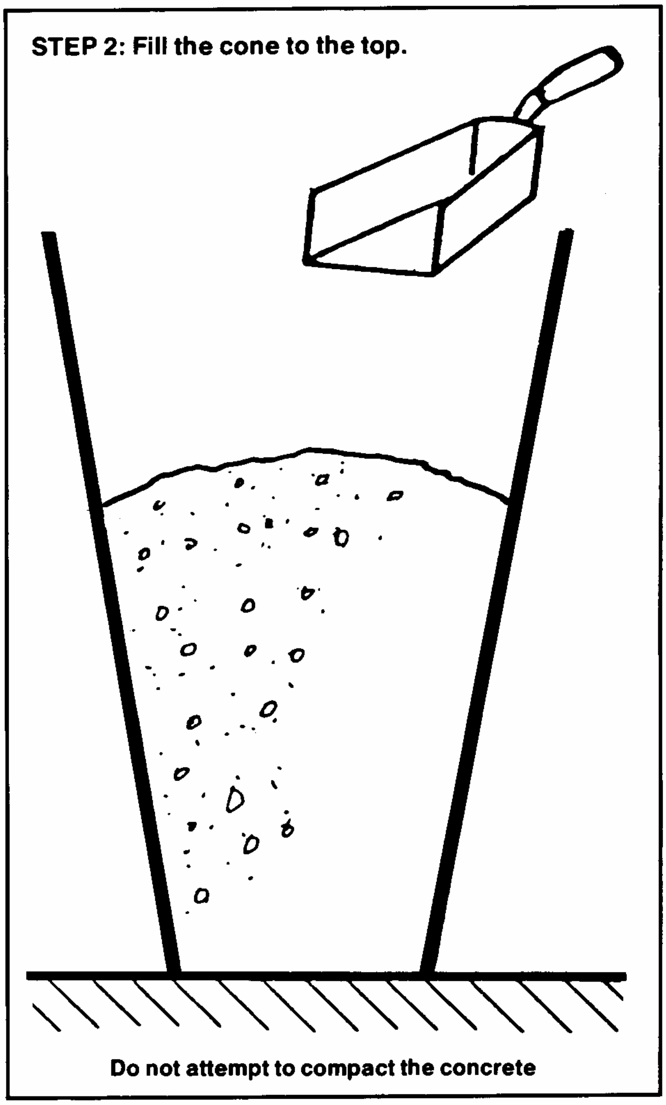
Spread Test (As per NZS 3112, Part 1:1986)

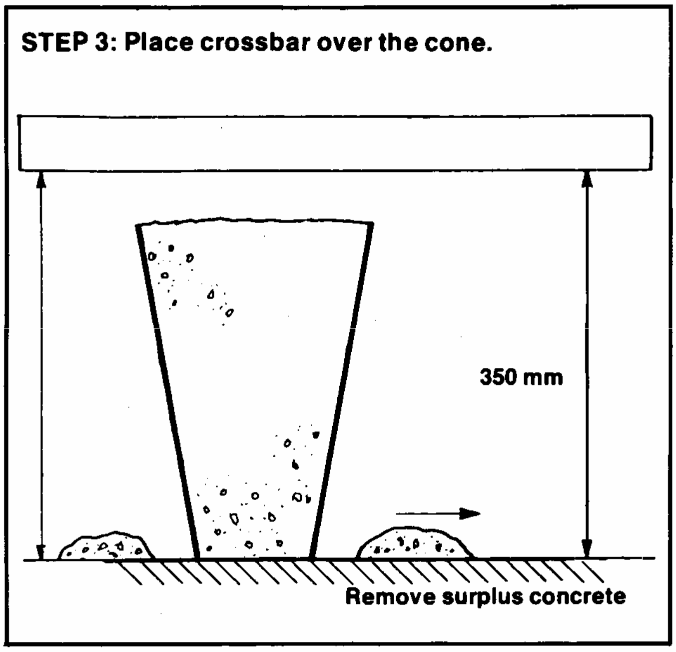
INTRODUCTION

The spread test gives a quick assessment of the fluidity of high-slump concrete mixes used for filling the cavities in reinforced masonry construction. (Of course, fluidity is not the only practical criterion which such mixes should satisfy; thus, for example, maximum aggregate sizes must be compatible with the presence of reinforcement within cells of particular dimensions). Equipment for the spread test comprises a suitable base, a standard slump cone, metal scoop, metric rule, and some means of restricting cone uplift to 50 mm. The base should be clean, flat, smooth-surfaced, rigid, and non-absorbent, with a lateral dimension of not less than 600 mm. It should be level and free from vibration during the test. Where a smooth concrete slab or floor area (without surface flaws) is used as the base plate, this must be moistened prior to the test to inhibit absorption of fresh concrete. The limiting of cone uplift to 50 mm can be achieved by the provision of an upstand and cross bar arrangement.

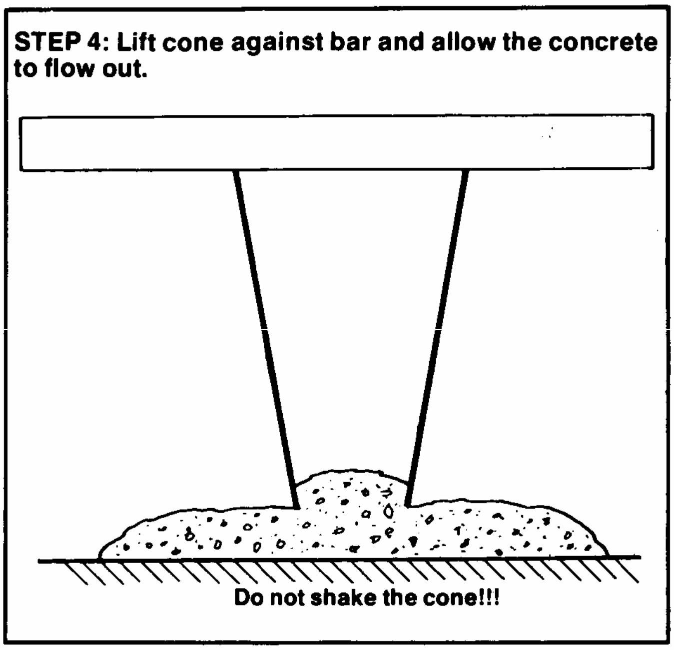
TEST PROCEDURE



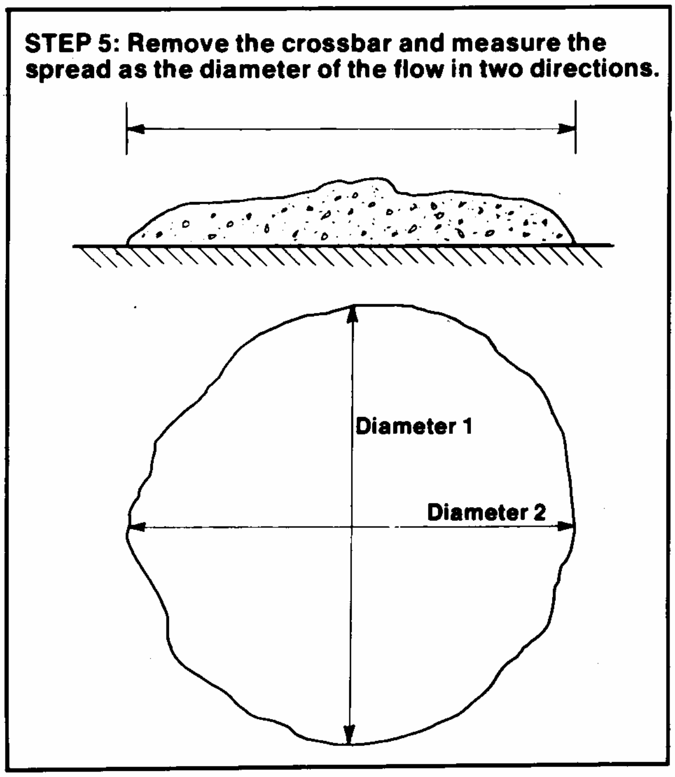




STEP 3: Place crossbar over the cone.



STEP 4: Lift cone against bar and allow the concrete to flow out.



STEP 5: Remove the crossbar and measure the spread as the diameter of the flow in two directions.

Place the moistened slump cone, narrow end downwards, on the center of the base plate.

Step 1: Remix the concrete sample thoroughly; and then fill the slump cone taking care to ***avoid segregation and/or compaction.***

Step 2: Set up the stop device for limiting cone uplift, providing a clear height of 350 mm above the surface of the base plate.

Step 3: Subject the cone to a quick vertical lift of 50 mm. Hold the upper end of the cone against the stop device until the flow of concrete ceases.

Step 4: Remove the slump cone and stop device, and measure the "spread" of the concrete across two diameters at right angles.

Step 5: Average the two measurements. Report spread to the nearest 10 mm.

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INFORMATION BULLETIN: IB 47

Starter Bars – Concrete Masonry

INTRODUCTION

Starter bar positions for concrete masonry are a vital part of the wall construction.

Errors in position can result in the wall being understrength and in many cases unable to be built without the replacing of the-starter bars by drilling and epoxy grouting steel in the correct position. This is time consuming and wasteful, since errors are often only discovered when the blocklayer lays out the first course.

This design note is applicable to 100, 150, 200 and 250 series walls built to modular pattern.

REQUIREMENTS

To be able to place the starter bars in the correct position, the contractor responsible for the foundation or floor construction must have the following information in addition to foundation dimensions:

1. Size and spacing of vertical bars.

Note: For non-specific design masonry buildings with solid filled walls:

Running bond: vertical bars will be D12 at 600 mm centres in earthquake zone A, and 800 mm centres in earthquake zones B and C.

Stack bond: vertical bars will be D12 to 600 mm centres in all zones. For partial filling, vertical bars will be D12 at 600 mm centres in earthquake zones B and C only, and constructed in running bond.

Bar size limited to D10 in 10 series walls.

2. Thickness of masonry walls to be built.
3. Edge support detail of masonry wall.
4. Positions of doorways.
5. Positions of windows.
6. Position for cut block on non-modular construction, if this is known.

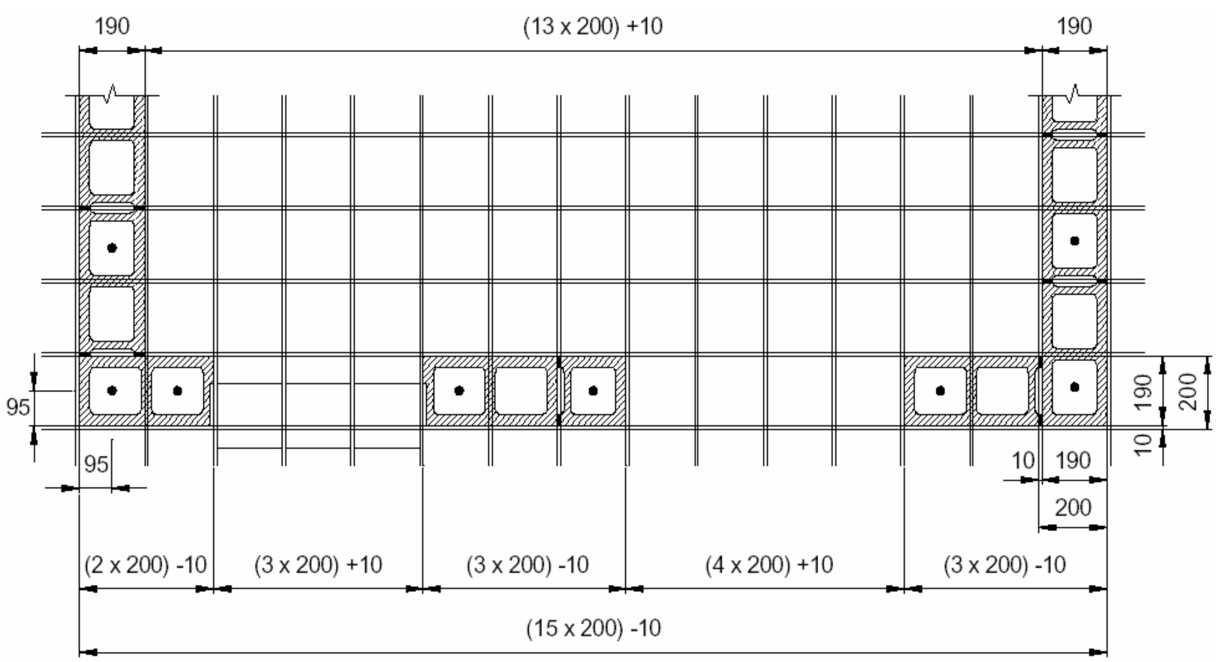


Figure 1: Planning on the standards concrete masonry grid.

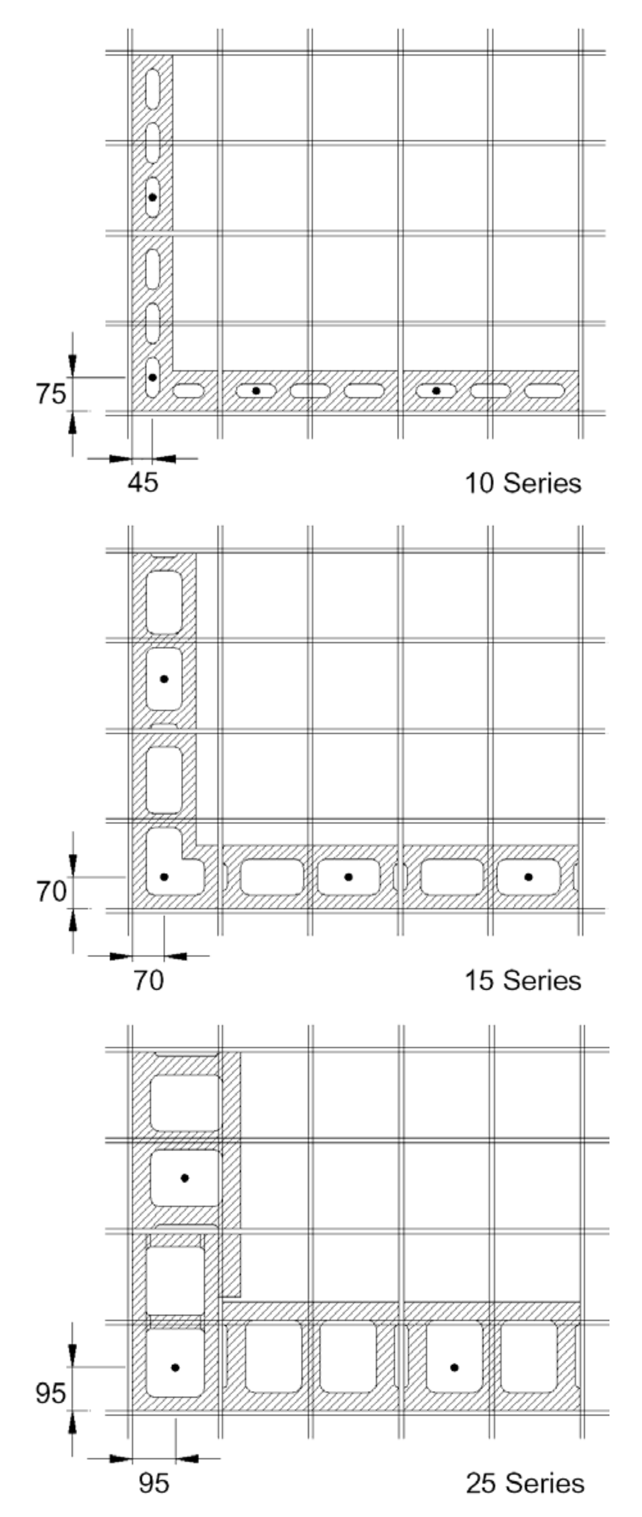


Figure 2: Modular corner constructions

SETTING OUT

The details in this document are based upon the modular concept of 200 mm at each corner and intersection. Masonry units are produced in different thicknesses, yet may be put together to meet the modular concept (Figures 1 and 2). There are a number of basic rules which will apply to ALL masonry construction.

1. Every corner will be solid filled with grout and will contain a vertical reinforcing bar.
2. Every bonded T intersection will have a solid filled core and will contain a vertical reinforcing bar.
3. Each masonry cell immediately adjoining a door opening will be solid filled and will contain a vertical reinforcing bar.
4. Each masonry cell adjoining a window will be solid filled and contain a vertical reinforcing bar.

The starting point for setting out will be the corners of the building.

Step 1:

Determine measuring position at a corner.

With modular construction for masonry walls of nominal 100 (2 holes), 150, 200 and 250, the position from the corner will be 95 mm.

If the block wall is to overhang the foundation then the overhang dimension must be subtracted from the 95 mm (see figure 3).

Put nail in timber shutter or mark steel shutter. The 100 series which has 3 holes per unit will have a starting dimension of 75 mm.

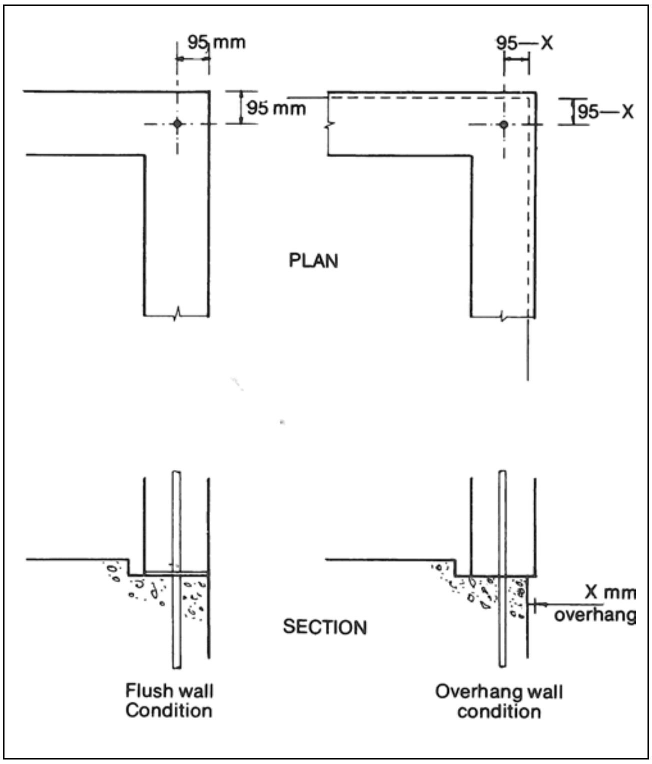
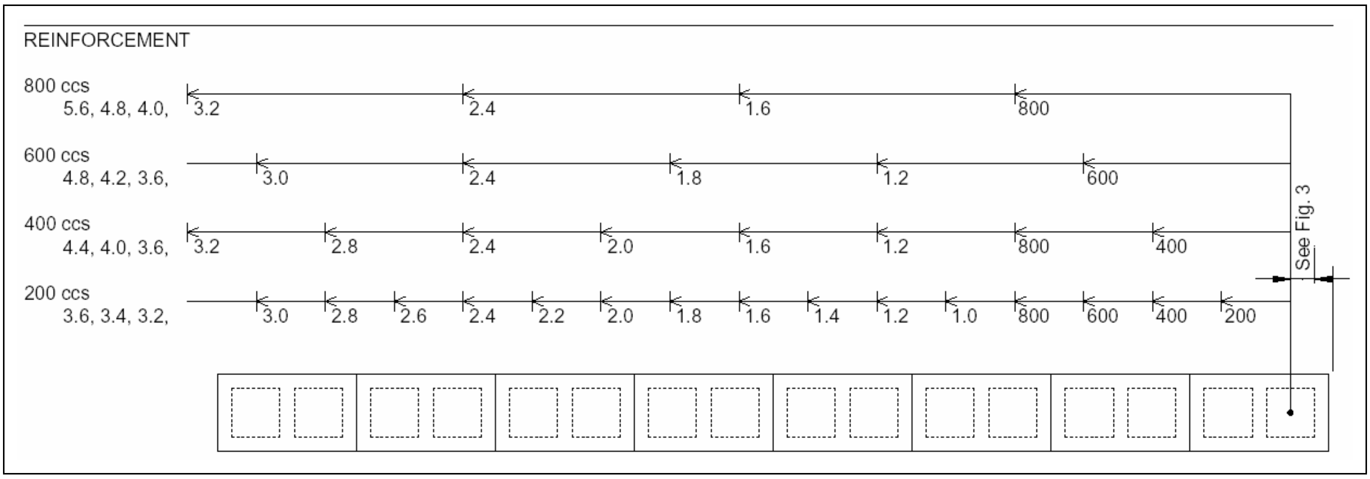


Figure 3: Foundation corner detail 200 series

Table 1:
Cumulative dimensions for start bar positions



Step 2:

From this comer position, using Table 1, mark the running dimension positions on the shutter, depending upon the nominal centres of steel required. If the wall length is non-modular (i.e. not a multiple of 200 mm), it is recommended that bar positions be marked out from each corner (see step 3). A tape hooked on a nail or a special long gauge rod are the simplest ways of measuring out.

Step 3:

From the opposite corner bar position, mark the running dimensions positions on the shutter until reaching the original set, Step 1. These may not match, so include an additional bar (figure 4).

Step 4:

Mark positions of windows and doorways and ensure there is a reinforcing bar in the adjoining cell. Extra bars may be needed to cover this case (figure 5).

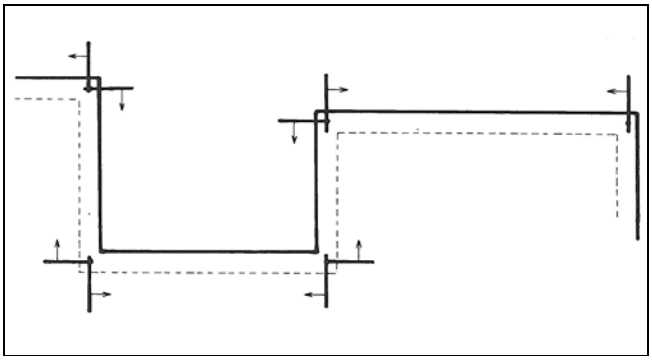


Figure 4: Work dimensions from each corner

Step 5:

Positions of starter bars in the opening of doorways can be omitted (figure 5).

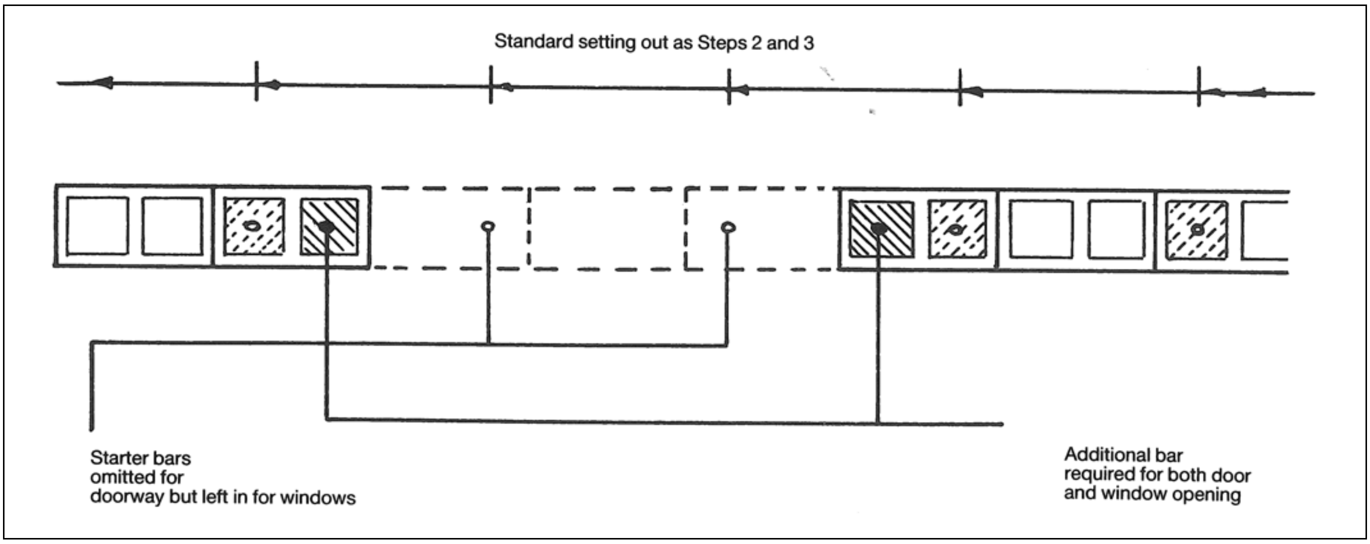


Figure 5: Example of Steps 4 and 5

Step 6:

The position of the bar from the face of the block depends upon the wall thickness and bar size.

The dimension from the shutter face will therefore be figures from Table 2 if the block is to be placed flush with the foundation (see figure 6). If an overhang detail is used, the overhang dimension must be subtracted from the figures in Table 2 (figure 6). Note that the corner starter bar lies on centre line of wall (figure 7). This only corresponds with the setting out reference position on the 200 series.

Table 2:
Position of reinforcement from block face

Nominal Wall Thickness (mm)	Face of block to	
	Centreline of bar (mm)	Face of 10/12 mm bar (mm)*
100	45	40
150	70	65
200	95	90
250	120	115

D12 bars are used in the non-specific design code NZS 4229.

D10 bars maximum size in 100 mm (10 series).

* Dimensions have been rounded to nearest 5 mm

Step 7:

Fix starter steel to shape and dimensions required. Typical details for D12 bars are shown in figure 6.

Step 8:

Immediately after concreting, make a running dimension check to see that steel is in correct position. Adjust as necessary, but make sure concrete around the starter is recompacted.

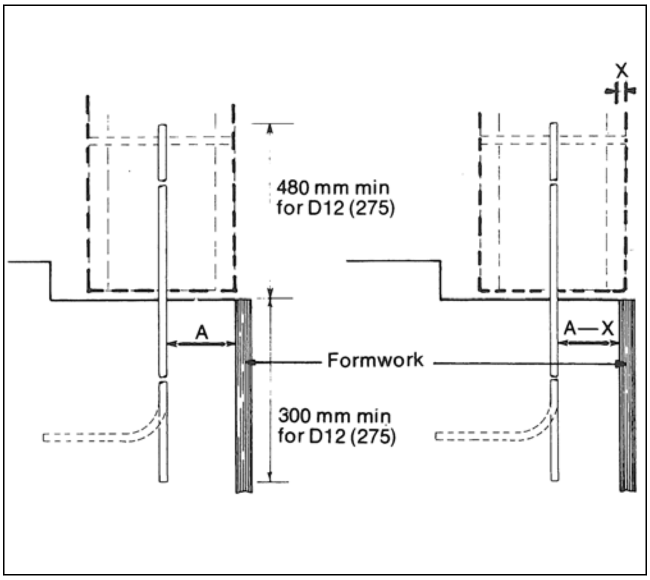


Figure 6: Position of bar from face of foundation

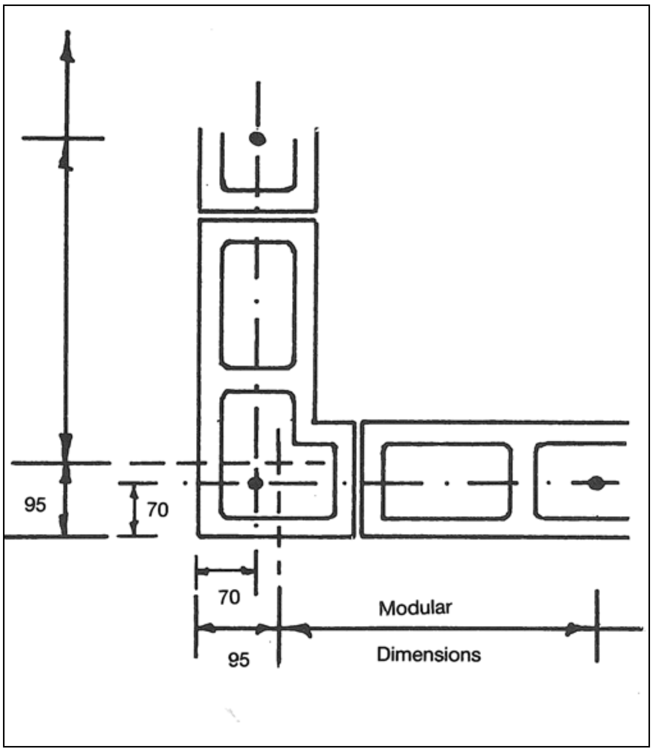


Figure 7: Positioning corner bar on 150 series

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3.2 Reinforcement for Masonry

Introduction

The reinforcement distribution within a masonry wall is determined by specific engineering design requirements of NZS 4230 or by specified requirements of NZS 4229 or NZS 3604. The reinforcement used must comply with steel Class E of AS/NZS 4671.

There are two grades of steel available 300 MPa and 500 MPa. The most common situation is to use 300 plain round bar for all links/ties and to use 500 deformed bars for main steel. Because of the restricted nature of a masonry block cell, the maximum size bar used is restricted to 16 mm.

The lap lengths for splicing two bars are 40 db for 300 MPa steel and 70 db for 500 MPa steel. It should be noted that where horizontal steel is in the top 300 mm of a wall and the grout does not contain an expanding agent there is a 1.3 times penalty on the lap lengths i.e. 52 db and 91 db respectively.

The reinforcement distribution from NZS 4229 is as follows:

(a) **Solid Filled**

Vertical bars D12 at 800 mm centres
Horizontal bars D12 at 1200 mm centres

(b) **Partially Filled**

Vertical bars D12 at 800 mm centres
Horizontal bars D16 at 2800 mm centres

The workmanship and installation of the reinforcement follows the provisions of NZS 4210.

Bending Provisions

The bending provisions of NZS 4210 which are contained in Appendix 2D, set minimum criteria for diameter of bends.

Application	Bar Diameter	Minimum Diameter of Bend	
		Plain	**Deformed**
Stirrup and Links	6-20	2db	4db
	25-32	3db	6db
Main Steel	6-20	5db	5db
	25-40	6db	6db

Note for masonry construction maximum bar size is 16 mm

Fixing Requirements

Reinforcement must be fixed:

(i) Within ± 6 mm in the width of the wall or column

(ii) Within ± 50 mm in the case of the length of the wall or one quarter the length of an individual cell whichever is smaller see Figure 1.

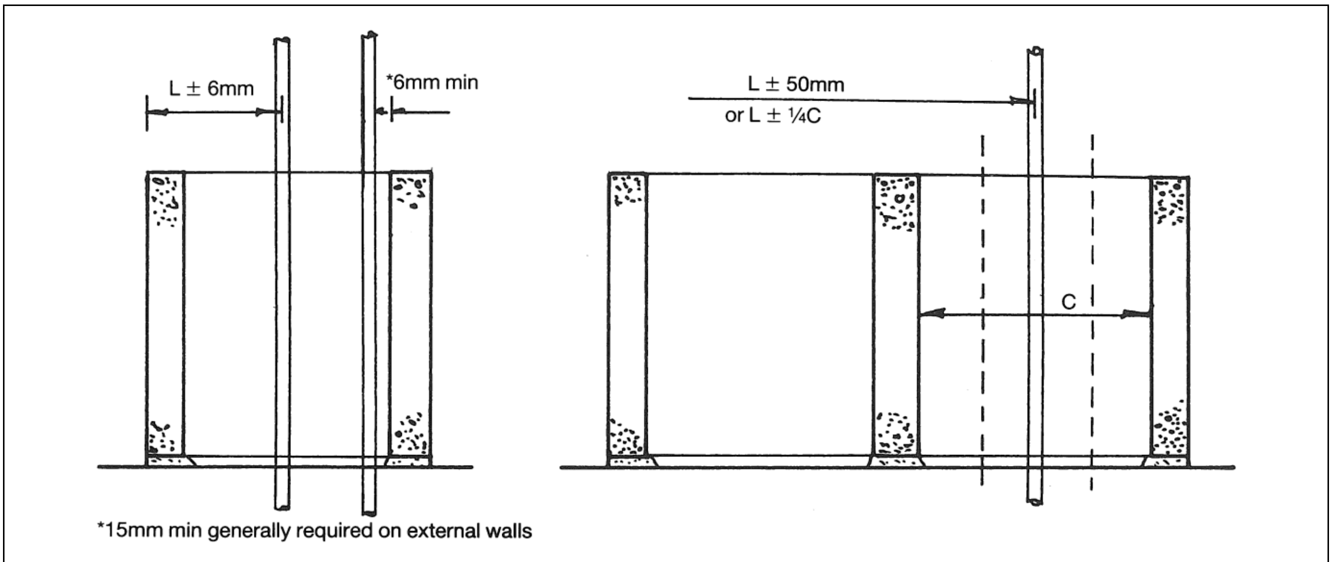


Figure 1: Fixing Tolerances for Reinforcement

In addition reinforcement must be maintained at least 6 mm from the face shell of the block. However where masonry is in an external situation, a higher value of cover to the face shell will be required, i.e. ranging from 15 mm to 30 mm depending on the category of exposure.

In order to ensure that reinforcement remains in position during the grouting operation, the reinforcement must be adequately tied. First to the starter bars and then at intervals in the height see Figure 2.

From Figure 2, it will be seen that the D12 bar does not require tying between the nominal storey lift of 2.4 m. Hence it is unusual to use D10 as vertical steel. The vertical bar is assumed to be in the centre of the cell unless the designer specifies differently. The important construction use, where off setting from the centre line will occur, is retaining walls.

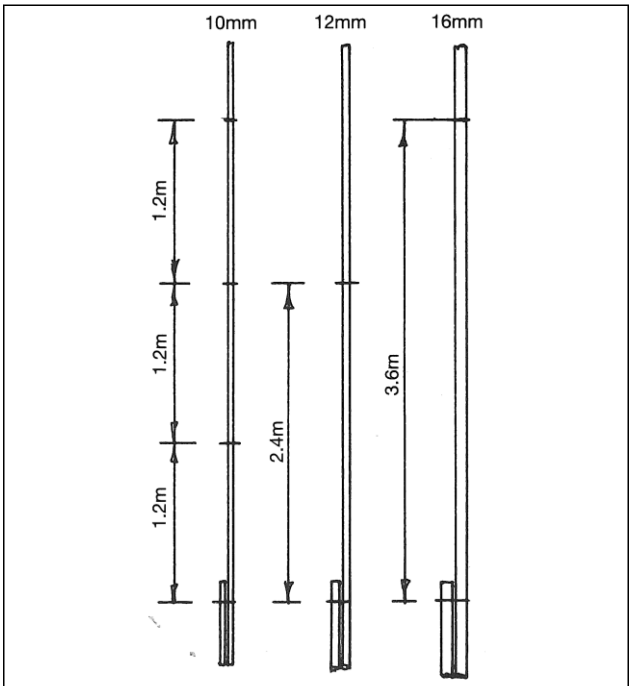


Figure 2: Maximum Spacing of Vertical Bar Fixings

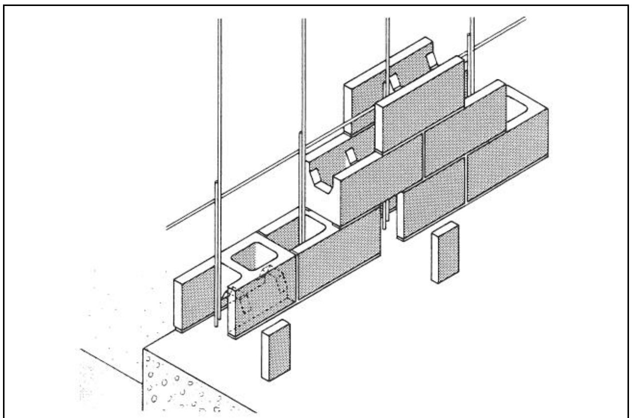


Figure 3: First Course Clean-out Pockets

Fixing Operation

The vertical steel is generally prefixed to starter bars before the laying of blocks start. However in some configurations, steel may have to be fixed after laying. Where this becomes the case, a clean out opening, discussed in Part 3.3, must be provided at each starter bar to allow the vertical bar to be tied to the starter. See Figure 3.

The horizontal steel must be placed as laying work proceeds since apart from the top course there is no way of inserting the steel once the blocklaying work is completed.

It is important to ensure that horizontal trimming bars under openings are correctly fitted as the work proceeds. Typical arrangement for wall detailing and trimming openings are shown in Figures 4 and 5.

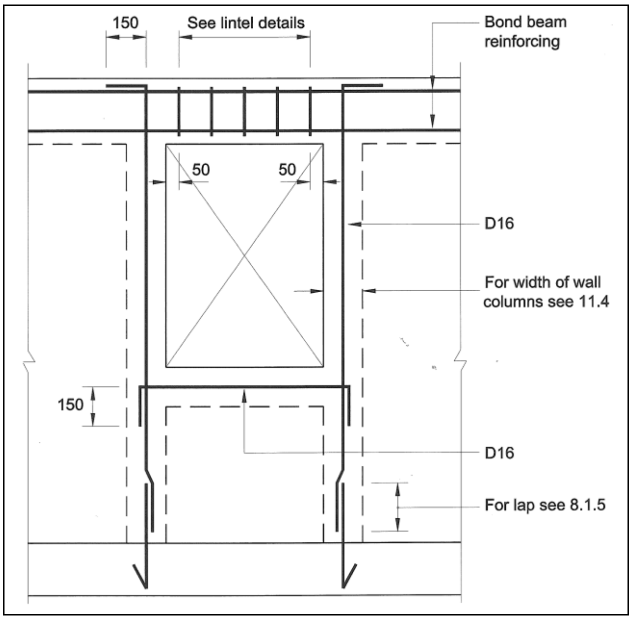


Figure 4: Reinforcement Above and Below Openings (Figure 8.1, NZS 4229)

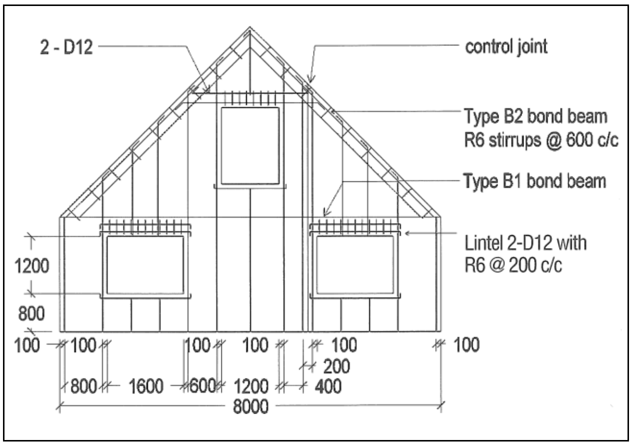


Figure 5: Bond Beam and Lintel Reinforcement for West Wall (Bracing Line B)

3.3 Wall Construction

Introduction

Sections 3.1 and 3.2 deal with the specifics of laying and the fixing of reinforcing steel.

This section primarily looks at the grouting and general construction of the wall.

Clean-out Openings

For most concrete masonry walls, clean out openings will be required.

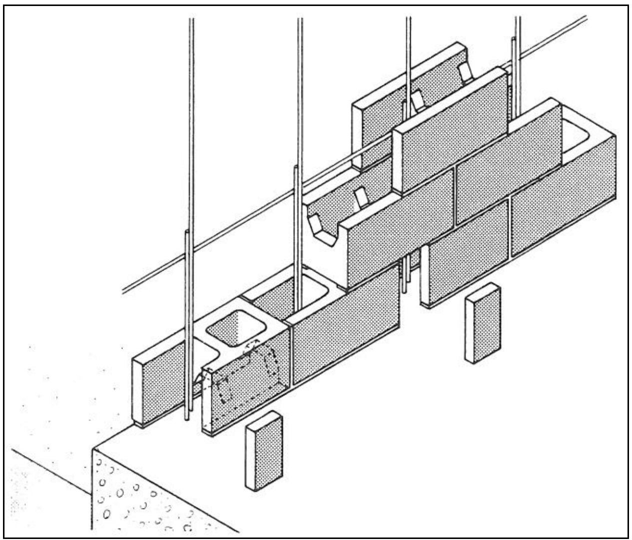


Figure 1: Clean-out Units

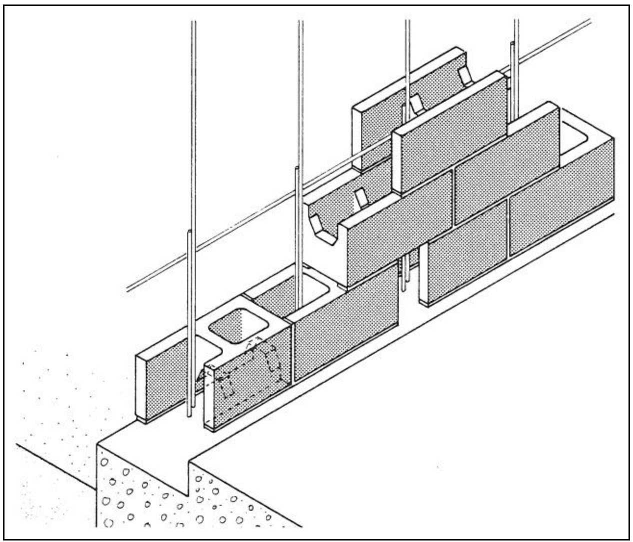


Figure 2: Clean-out Units in Rebated Base

It is usual to cut a face shell creating a 75 mm – 100 mm wide opening the full height of the block. See Figures 1 and 2. Note that for partial fill construction, an edge rebate is required to comply with weathertightness requirements.

This is to enable any debris/loose mortar etc., to be cleaned out from the base of the wall before grouting.

The clean outs should be positioned at every bar position for partially filled masonry or at 800 mm centres for solid filled walls.

Where fixing of vertical steel has to be carried out after block laying, the clean outs have to match the steel positions.

To facilitate cleaning out for solid fill walls the first course is an inverted 2016 (1516) block with a layer of sand spread on the concrete foundation surface before further courses are laid.

Cleaning Out

Once the block laying for the wall is completed, the base course is cleaned out usually by water jet aided by compressed air.

After an inspection to check cleanliness and adequate steel tying, the clean out pieces are mortared back into place where a fair face finish is required or a temporary plywood shutter is braced over the opening or tied to steel.

Grouting

There are four different methods described in NZS 4210 Clause 2.11.6, one relates to low lift (1.2 m high) and three relate to high lift systems up to 3.6 m.

The most structurally efficient way of grouting is the High Lift Grouting with Expansive Admixture.

This is the method specified in the Manual and is illustrated by the following sequence of steps, where the scaffolding shown is diagrammatic:

Step 1:

Clean-out grout space and remove all debris and loose material from the construction joint. See Figure 3, page 2.

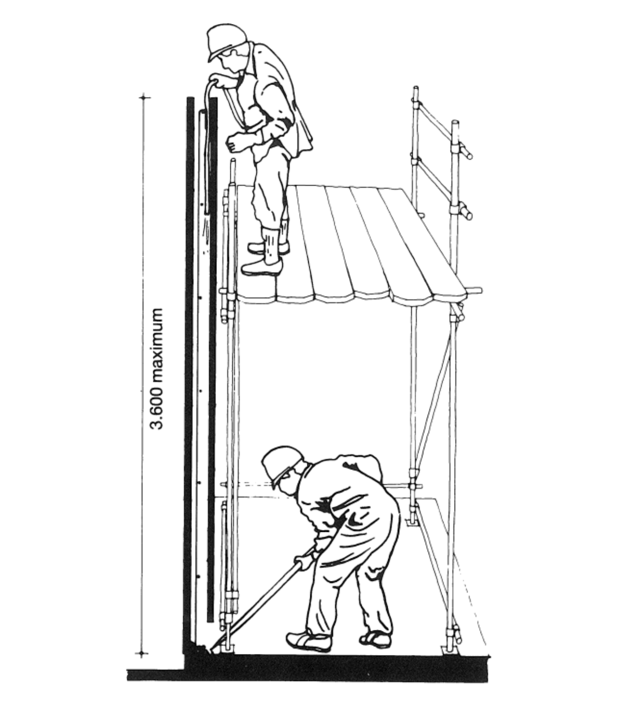


Figure 3: Step 1

Step 2:

Grout the wall in a semi continuous operation to the top. See Figure 4.

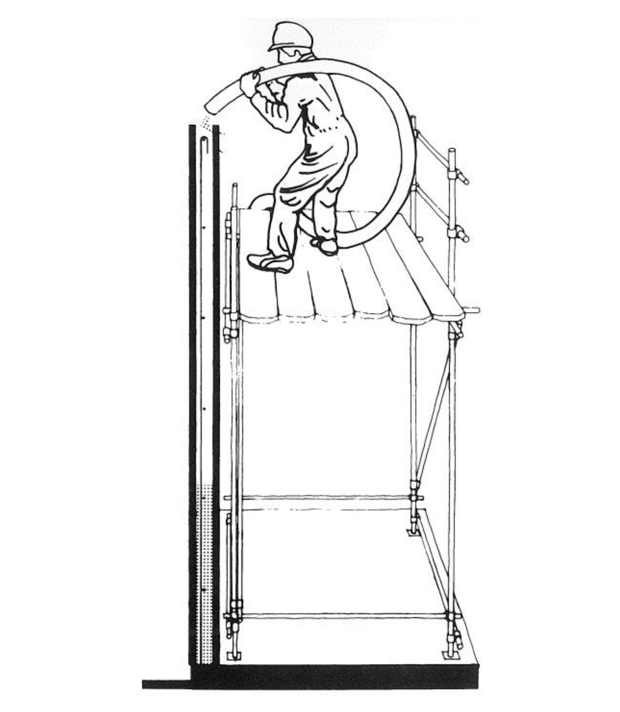


Figure 4: Step 2

Step 3:

Consolidate the grout using a bar or rod minimum diameter 16 mm. This can be done by full depth rodding or by using a 25 mm pencil vibrator. When dealing with window openings it is important to use a 25 mm vibrator in the cells adjoining the opening to ensure grout flow along the sill line. See Figure 5.

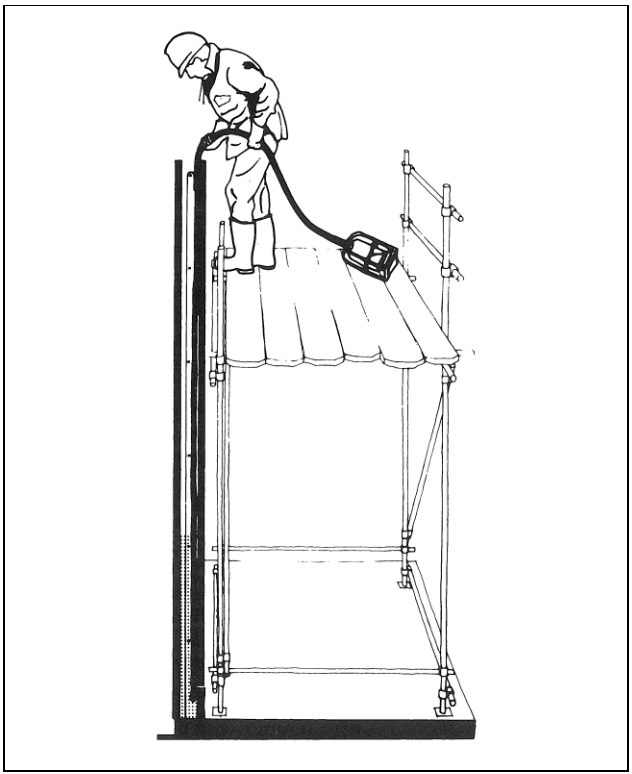


Figure 5: Step 3

Step 4:

Trowel down the top of the wall after grout expansion has taken place. See Figure 6.

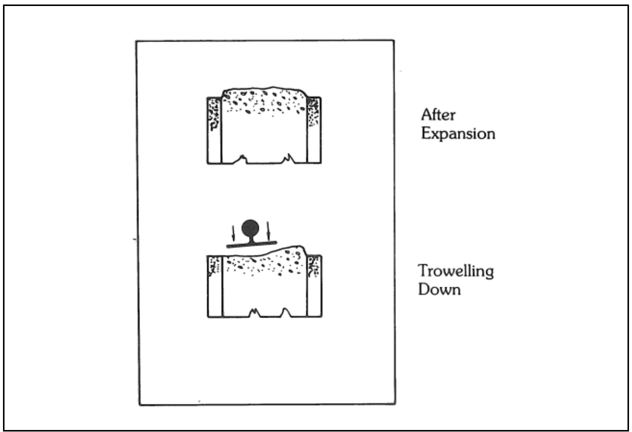


Figure 6: Step 4

Evidence that the expansion agent is working is often accompanied by the outside of the masonry shells taking on a "wet appearance". Testing has shown that significantly improved bond between face shells and the grout core is experienced when using the agent.

Control Joints

Horizontal wall movements mainly associated with shrinkage may require the use of control joints. Since the movements are influenced by various design factors such as the amount of horizontal steel, the positioning of the joints is the responsibility of the designer.

Control joints are described differently in various New Zealand Standards and review of those specifications with particular reference to NZS 4229 is now contained in a new Section 6.3 Control Joints in the Manual.

The control joint spacing may not exceed 6 m in NZS 4229 and its positions in the building are shown in Section 6.3. Commercial work would normally accept up to 8 m spacing.

The joint itself can be formed using closed end blocks or open-ended blocks but using a filler strip which is 30 mm narrower than the wall thickness – see Figure 7. The use of control joint blocks shown in Figure 8 has largely been superseded.

The matter of reinforcement details adjoining or going through the joint is contained in Section 6.3.

On a practical note of weatherproofing the control joint, it is recommended that masking tape be applied to the face of the concrete masonry on both sides of the joints before applying the filler/sealant, thereby confining the filler/sealant to the joint and preventing the marking of the block face. The masking tape is removed after the surface of the filler/sealant has become firm.

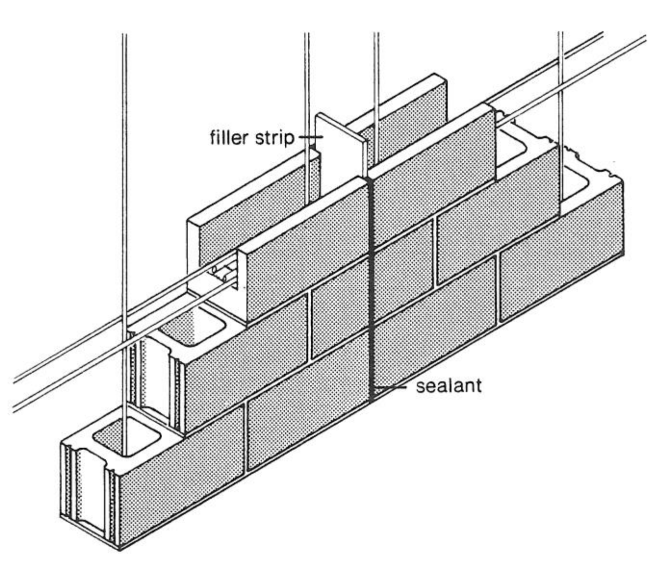


Figure 7: Control Joint Using Standard Whole and Half Unit

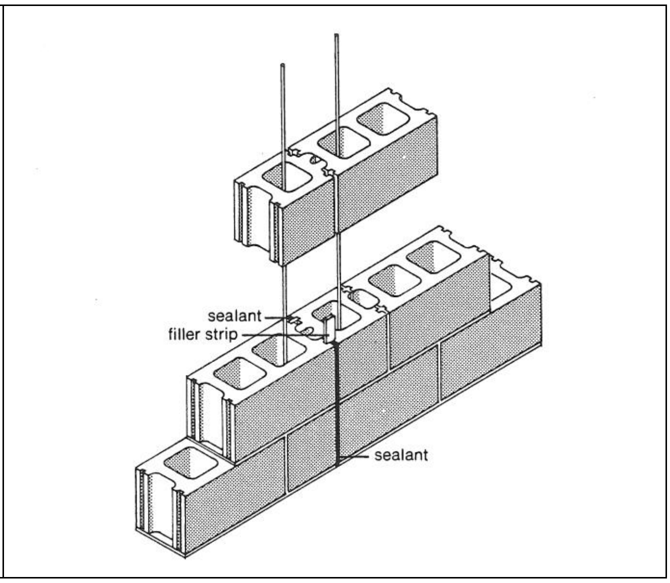


Figure 8: Control joint using control joint units

Construction Joints

Construction joints may be required between different masonry wall lifts. A horizontal construction joint will occur on the top of the uppermost masonry unit. The level of the construction joint, however, should not be lower than 20 mm from the top of this unit. See Figure 9.

The horizontal construction joint should be roughened to remove laitance and any loose matter lying on the surface of the hardened grout. This is a similar process to the preparation of the construction joint between the masonry wall and its supporting concrete beam or foundation. It is generally easy to wash and brush the joint a few hours after the grout has hardened to provide a clean surface ready for the next lift. See Figure 10.

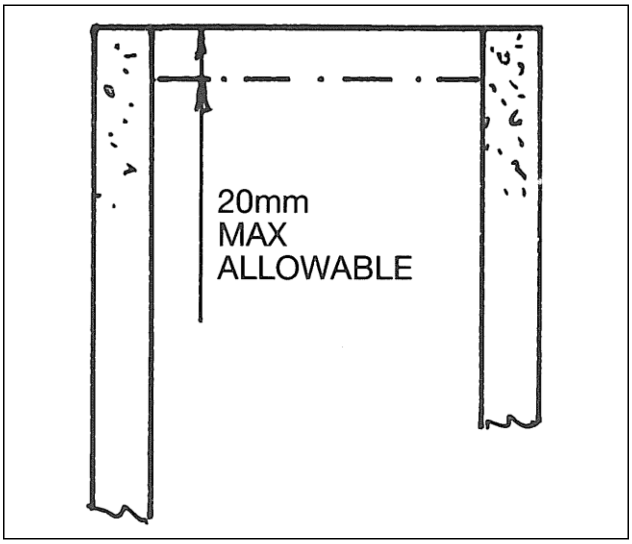


Figure 9: Horizontal Construction Joint

At the position of this intermediate horizontal construction joint it will, of course, be necessary to form clean out ports, as was required for starting off at the ground level. A sand covering of the cleaned surface will keep droppings from sticking to the surface. Cleaning out of the horizontal joint is required before placing the next lift.

Figure 10: Construction Joint Preparation

Temporary Bracing

A point which can often be overlooked is the need for temporarily bracing the concrete masonry during construction. An ungrouted wall is very susceptible to failure from strong winds. Typically, a wall over 1 metre in height is at significant risk. See Figure 11.

It is important to take some measures to brace the wall in order to prevent its premature failure. Typically bracing at 3 metre centres is recommended.

Cold Weather Construction

The precautions for cold weather construction are shown:

(a) Water used for mixing mortar shall be heated;

(b) Masonry shall be protected for not less than 24 hours after laying by covers, blankets, heated enclosures, or the like to ensure that the mortar can gain strength without freezing or harmful effects from cold winds; and

(c) No frozen materials or materials containing ice shall be used.

Hot Weather Construction

We have just discussed precautions to be taken during cold weather. It is also necessary to consider problems that might occur during hot weather construction. Generally, when the air temperature rises above 27°C, or there is a drying wind even though the temperature may be lower, it is necessary to take some additional precautions.

These are shown below:

(a) Masonry units may be lightly dampened before laying;

(b) Mortar shall be kept moist and not spread the wall more than two unit lengths ahead of the units being placed;

(c) The mortar shall be prevented from drying so rapidly that it cannot cure properly. This may be done by applying a very light fog spray several times during the first 24 hours after laying or by some other protective measures over the same period; and

(d) Grout shall be protected from too rapid drying.

Various research tests have shown advantages in carrying out these recommended practices.

Weathertightness

The basic requirements for achieving weathertight-ness are fully discussed in the CCANZ publication *CCANZ CP01: Code of Practice for Weathertight Concrete and Concrete Masonry Construction*, which is an approved approach for E2/AS3 of the New Zealand Building Code.

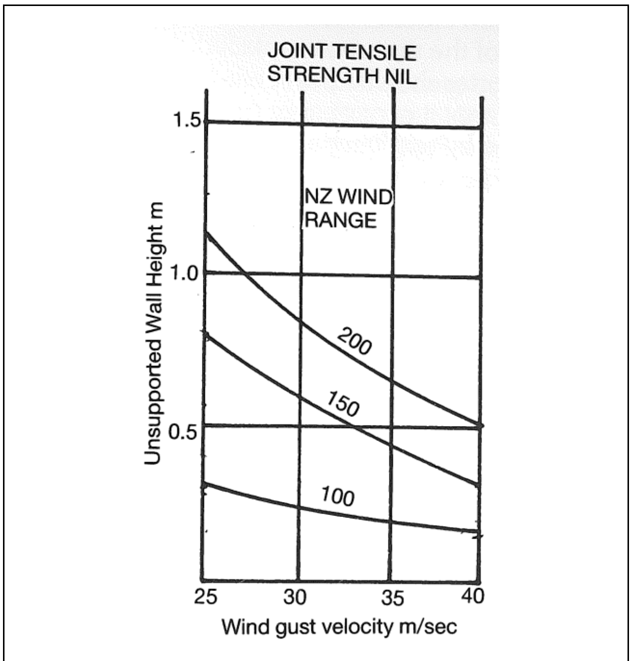


Figure 11: Maximum Unsupported Height of Ungrouted Masonry During Construction

Basic Construction Details

The concept details on the following pages, indicate basic uses of concrete masonry in building construction. They are presented as guides to detailing and construction and not as finite solutions to a wide variety of situations and conditions.

The details are primarily based upon NZS 4229 and NZS 3604. Reference to these two New Zealand Standards is often necessary to check the specific application of the detail to the job under design.

The exact size, number and positioning of reinforcing bars must be considered in every case. Wall thicknesses, footing widths and other matters must likewise be considered in each and every case.

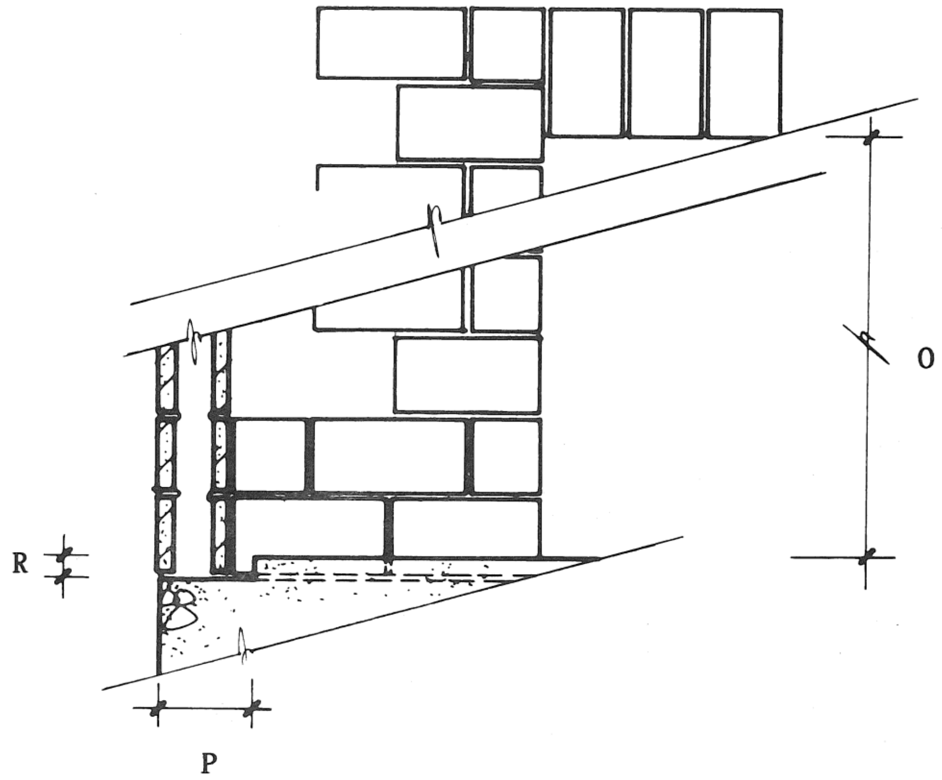
Certain site conditions might require that masonry or insitu concrete footings should be treated with a damp proof course to prevent ground moisture rising up into masonry walls. This matter is to be individually considered bearing in mind site conditions, structural design, construction procedure and other relative factors. Damp proof courses can be provided by painting the top of the footing with a bituminous or similar emulsion, but such a coating would prevent a full bond between mortar and footing and between grouted cores and footing.

A full mortar bed of waterproof mortar at the top of the footing might be acceptable, but in any and every case the matters of damp proofing and structural bond must be individually considered.

Cavities of cavity walls or veneer walls must be drained and ventilated as indicated, and moisture must be prevented from rising from a cavity into the roof space. This matter is more fully described under the Veneer walls section of this manual.

F0 FOUNDATION

CONCRETE MASONRY WALL
CONCRETE SLAB
PERIMETER STEPDOWN 20-50 mm



F0/A

F0/A

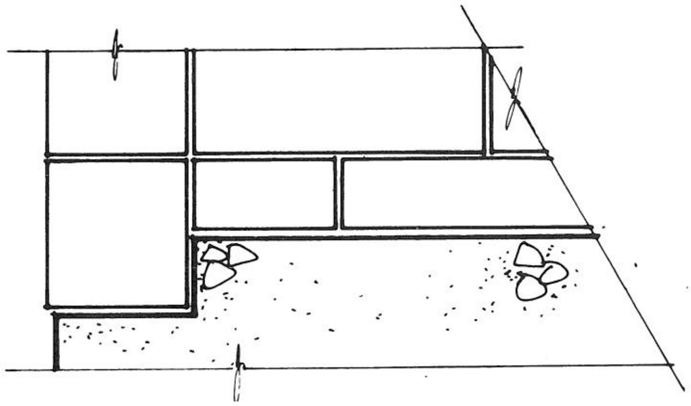
A nominal stepdown to improve weathertightness of the structure at floor level can be used – 20-50 mm. The following consequences on the modular construction must, however, be considered:

1. Rebate required for any bonded internal walls.
2. Opening dimension from floor to modular head position will be reduced by the depth of rebate chosen. To maintain a satisfactory opening size and block coursing, it will be necessary to trim standard lintel blocks.
3. Width of rebate should be wide enough to permit the satisfactory provision of clean-out ports and their subsequent reinstatement.
4. Provisions to drain unfilled cells by way of weep holes should be made.

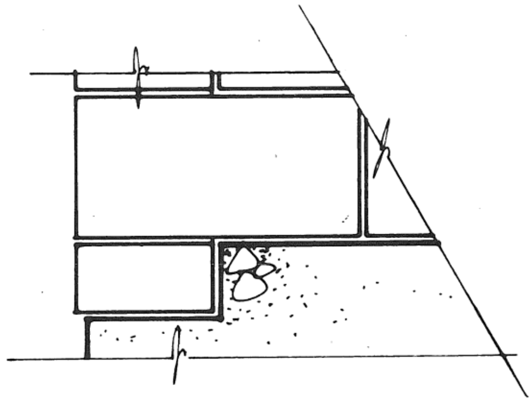
See also Modular Masonry Section

R = Rebate Depth: 20-50 mm
0 = Clear Opening: To suit individual requirements
P = Width of rebate to allow for cleanout port

F0



F0/B



F0/C

F0/B and F0/C

A half high modular step down to improve the weathertightness of the structure at floor level can be used.

The following consequences on the modular construction must, however, be considered:

F0/A

1. No rebate is required for internal walls by using a half high course of masonry.
2. For partial fill, clean-out ports at each reinforcing bar position in the half high course should be provided.

   For solid fills, the internal half high course should be grout filled after laying prior to proceeding further. An inverted 20.16 unit would then be laid incorporating clean-outs – see F1 (or 15.16).
3. Door openings - see Note F0/A2.
4. Drainage of unfilled cells - see Note F0/A4.

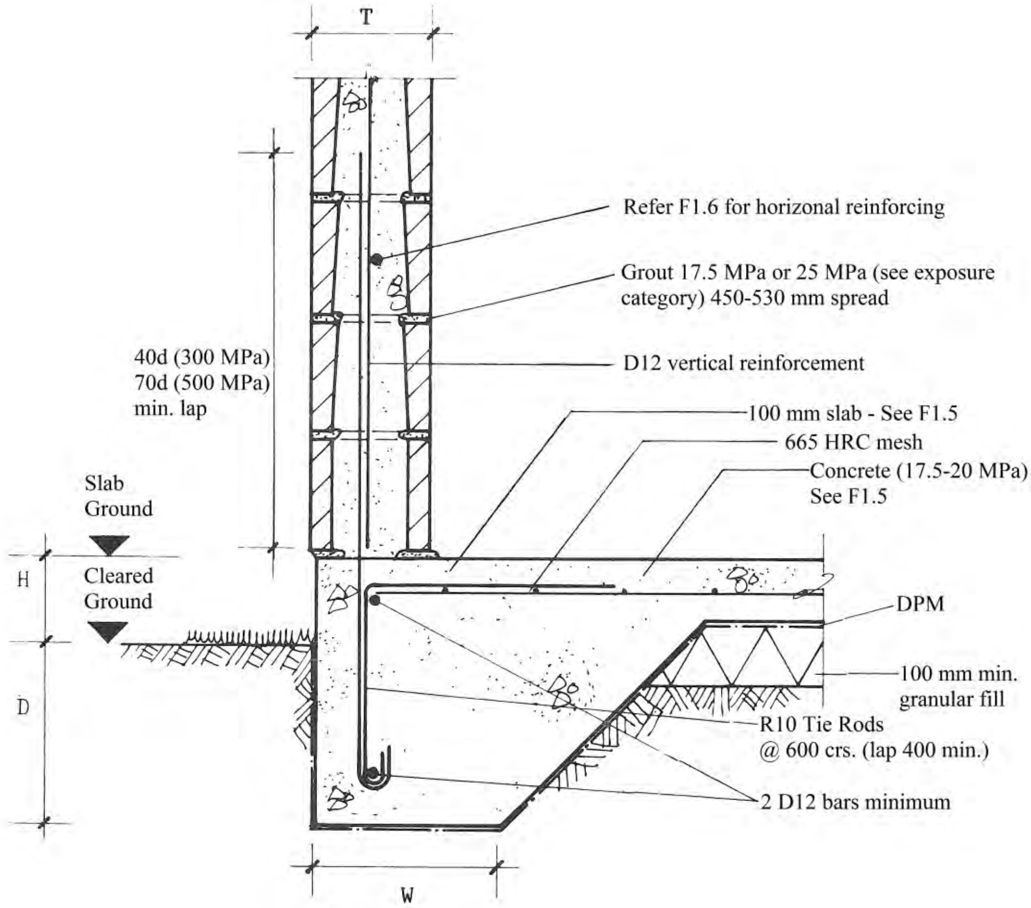
F0/B

1. No rebate is required for internal walls by using a half high course of masonry on the external wall.
2. For partial fill see F2 details.
3. Door openings are related to modular sizes.
4. Drainage of unfilled cells - see F0/A4.

See also Modular Masonry Section.

F1 FOUNDATION

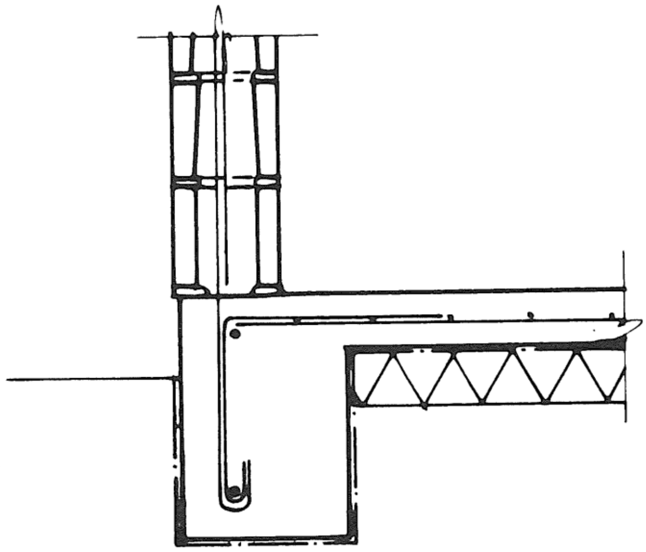
CONCRETE MASONRY WALL
SOLID FILLED CONSTRUCTION
CONCRETE SLAB
CONCRETE FOOTING



F1/A SLOPING FOOTING

H = Height of slab above ground:	100 mm above paved surface. 150 mm above unpaved surface.
D = Minimum depth of footing:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Footing width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
T = Masonry Thickness:	140 mm for 15 series 190 mm for 20 series 240 mm for 25 series

F1



—·—·— D.P.M.

F1.1 Principal Masonry units used in this detail are:

- Type 16; 05.

F1.2 Bottom course formed by using inverted 20.16 blocks enabling full length rodding to remove mortar droppings from slab interface (or 15.16).

F1.3 Cleanout ports at 800 mm crs usually formed by cutting outside face from bottom course, to be replaced and firmly wedged after cleanout and vertical steel in position.

F1.4 Setting the masonry into a rebated foundation detail can improve the weathertightness of the wall at that joint. The use of a rebate alters the notional modular door opening size. See F0.

F1.5 Concrete slab strength 17.5-20 MPa quoted is appropriate to domestic and non-direct wearing light duty floors. Commercial floors should have strength selected from 25-40 MPa as appropriate to condition. Floor thickness of 100 mm is taken from NZS 4229 for domestic loading. The spacing of control joints determines the size of mesh used in the floor.

F1.6 Steel spacing for non-specific design is as follows:

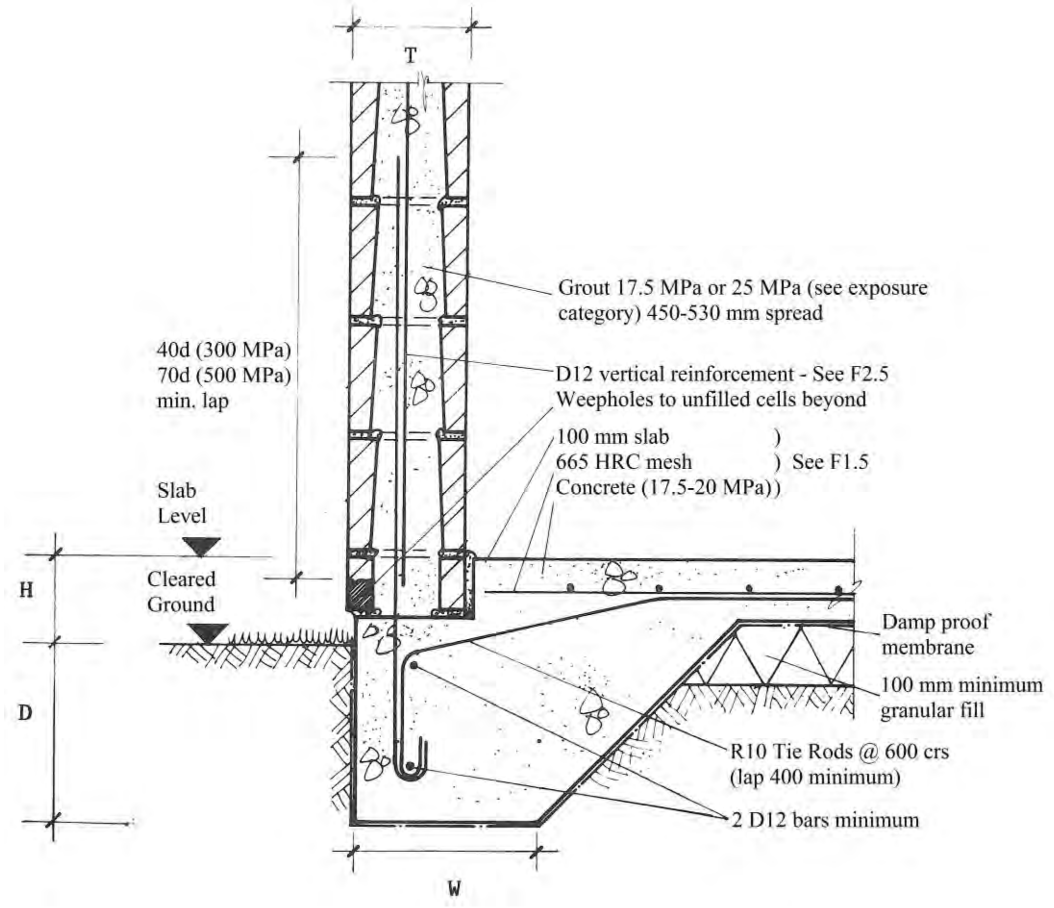
Seismic Zone	Horizontal Spacing of Vertical Steel (D12) mm	Vertical Spacing of Horizontal Steel (D16) mm
3(A)*	600	600
2(B)	800	800
1(C)	800	1,200 (2 No. D16)

F1.7 Refer NZS 4229 Sections 4, 5, 6.

*( ) Zone references prior to 2010.

F2 FOUNDATION

CONCRETE MASONRY WALL
PARTIAL FILLED CONSTRUCTION
CONCRETE SLAB
CONCRETE FOOTING



F2/A SLOPING FOOTING

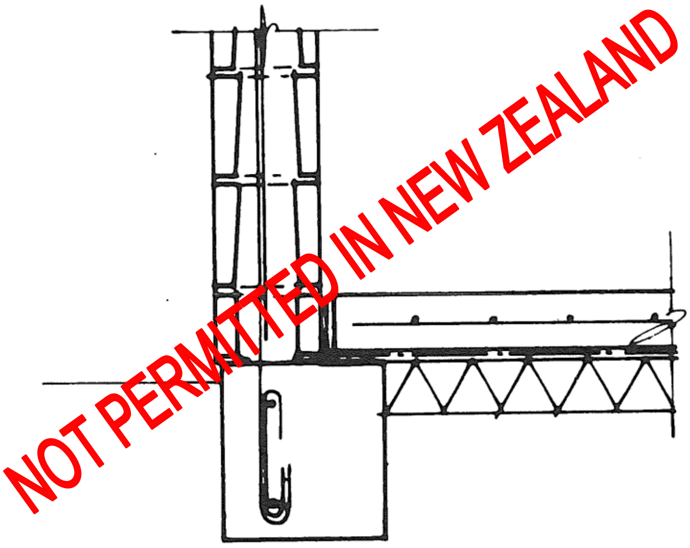
H = Height of slab above ground: 100 mm above paved surface.
150 mm above unpaved surface.

D = Minimum depth of footing: Typically 300 mm depending on soil type. See Section 3 NZS 4229.

W = Footing width: Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.

T = Masonry Thickness: 140 mm for 15 series
190 mm for 20 series
240 mm for 25 series

F2



F2/B FREE SLAB

—·—·— D.P.M. 0.25 mm polythene or multi-laminate polythene sheet

F2.1 Principal Masonry units used are:

- Type 01; 05; 14; etc.

F2.2 Provide weepholes in vertical joints midway between filled cells.

F2.3 Provide cleanout ports at vertical bar locations for cleanout and vertical lap tie.

F2.4 Provide a stepdown on external wall from the slab to the masonry bed to permit moisture egress.

On a perimeter wall, where obstructions prevent external cleanouts, use internal cleanouts in the second course.

F2.5 Horizontal spacing for vertical steel (non-specific design) is:

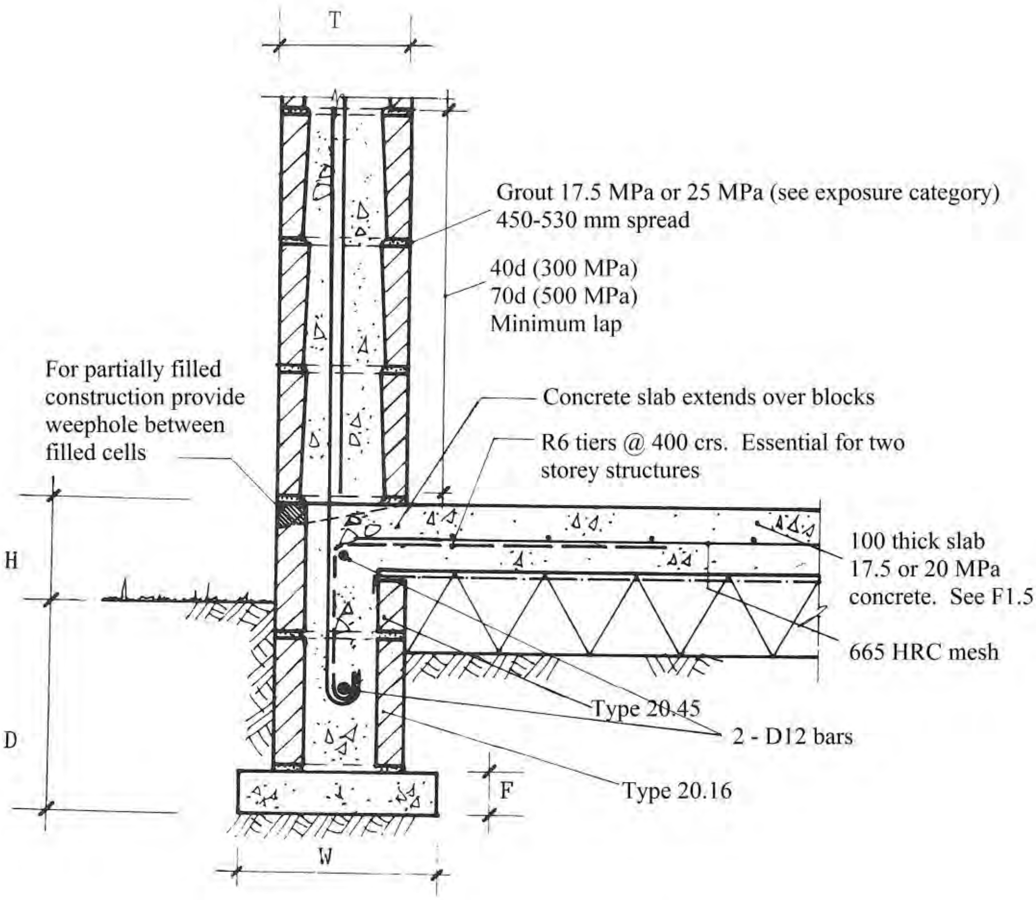
- 600 mm Zone B
- 800 mm Zone C

Note: Horizontal steel not required where bond beam is within 2.8 m maximum height.

F2.6 Detail may be used for solid filled construction - Refer F0 Section B.

F3 FOUNDATION

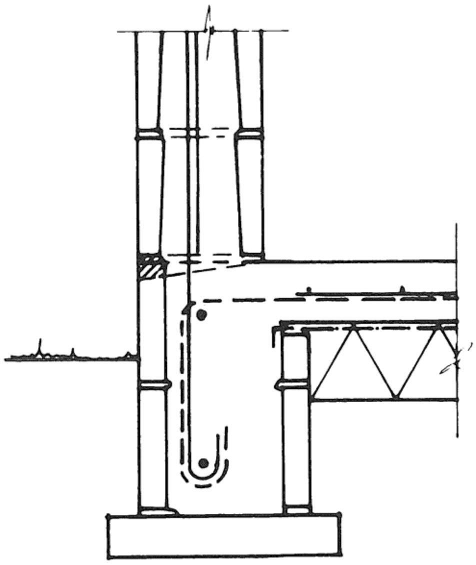
CONCRETE MASONRY WALL
CONCRETE FLOOR SLAB



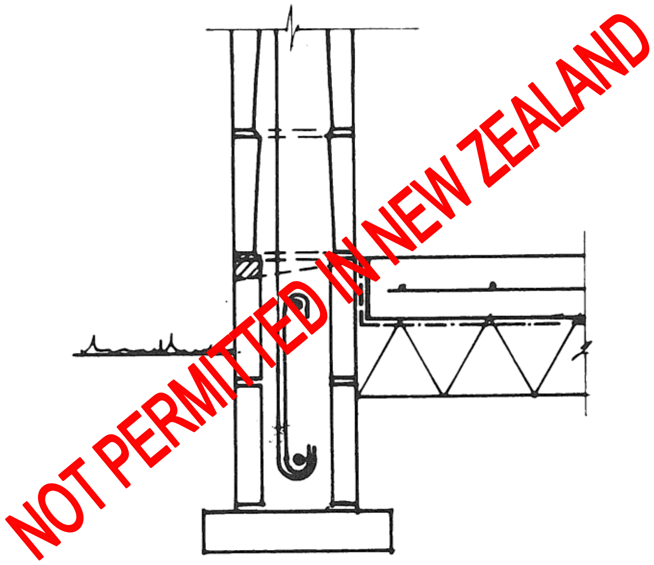
F3/A UNIFORM FOOTING TIED SLAB

H = Height of slab above ground:	100 mm above paved surface. 150 mm above unpaved surface.
D = Minimum depth of footing:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Screed width:	Minimum wall thickness + 100 mm. 17.5 MPa concrete. See Section 6 NZS 4229.
F = Screed Thickness:	60 mm minimum
T = Masonry Thickness:	140 mm for 15 series 190 mm for 20 series 240 mm for 25 series

F3



F3/B WIDE FOOTING



F3/C FREE SLAB

—·—·— D.P.M. typically 0.25 mm polythene or multi-laminate polythene sheet

F3.1 Principal masonry units used in this detail are:

- Footings - Type 16; 45
- Partially filled construction - Type 01; 05 wall etc.
- Solid filled construction - Type 16; 05 wall etc.

F3.2 For foundations less than 1.2 m high, inverted bottom course sand cleanouts are not necessary.

F3.3 Fill foundation core as casting floor slab**.**

F3.4 Where available, use Type 45 units as top of foundation and run D.P.M. over face shell to permit full integration with slab.

F3.5 In solid filled construction use inverted Type 16 units above slab level to permit nodding of slab junction.

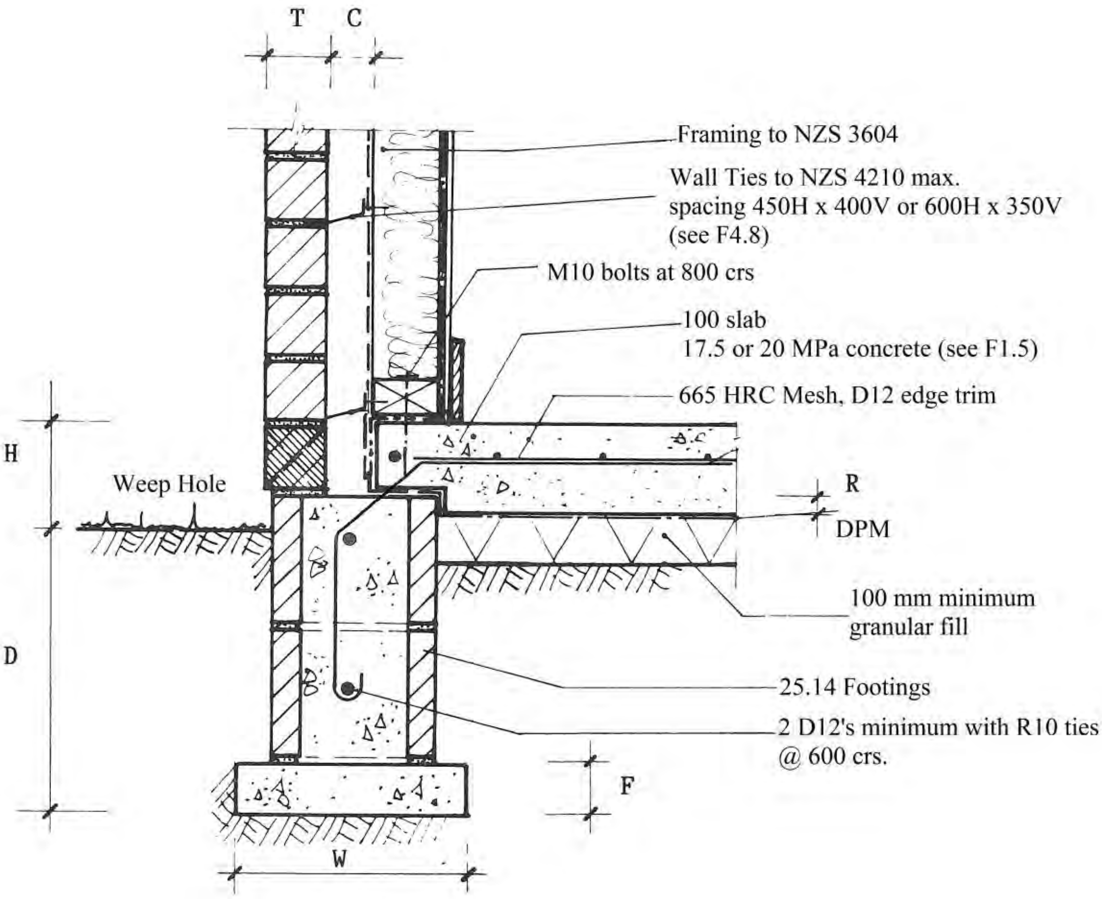
F3.6 In partially filled construction provide weepholes in vertical joints midway between filled cells and cleanouts at vertical base. These to be provided at slab level.

F3.7 All foundations must be excavated to horizontal benches. Soil conditions dictate the width of footings or sub-footings. See Section 4, 5 and 6 of NZS 4229.

F3.8 For horizontal spacing of vertical steel see F1 and F2 details.

F4 FOUNDATION

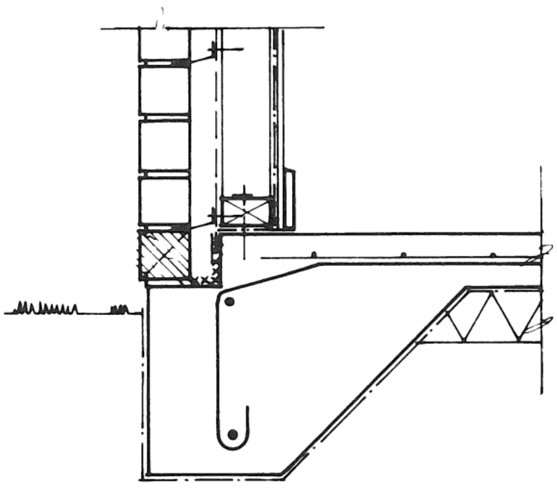
MASONRY VENEER CLADDING
TIMBER FRAME WALL
CONCRETE SLAB



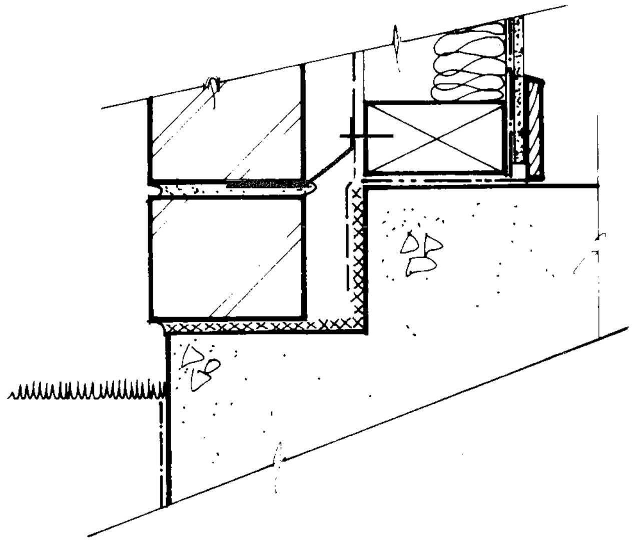
F4/A MASONRY FOOTING

H = Height of slab above ground:	100 mm above paved surface. 150 mm above unpaved surface.
D = Minimum depth of footing:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Screed Width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
F = Screed Thickness:	60 mm minimum.
T = Veneer Thickness:	70 mm minimum.
C = Cavity Width:	40 mm minimum NZS 4210. 75 mm maximum.
R = Rebate:	25 mm

F4



F4/B CONCRETE FOOTING



F4/C BUILDING MEMBRANES

—·—·— D.P.M. ——— Building Paper —··— Malthoid D.P.C. XXXXXX Bitumen Coating

F4.1 Principal Masonry Units used in this detail:

- Footing - Type 25.14

F4.2 Ventilation of cavity to exterior at top and bottom is required.

F4.3 Ensure cavity is free from pipes and services which would pass moisture from the rear face to the framing.

F4.4 Provide weepholes in vertical joints of bottom course at approximately 800 mm crs.

F4.5 Minimum step of 50 mm required between veneer seat and slab level.

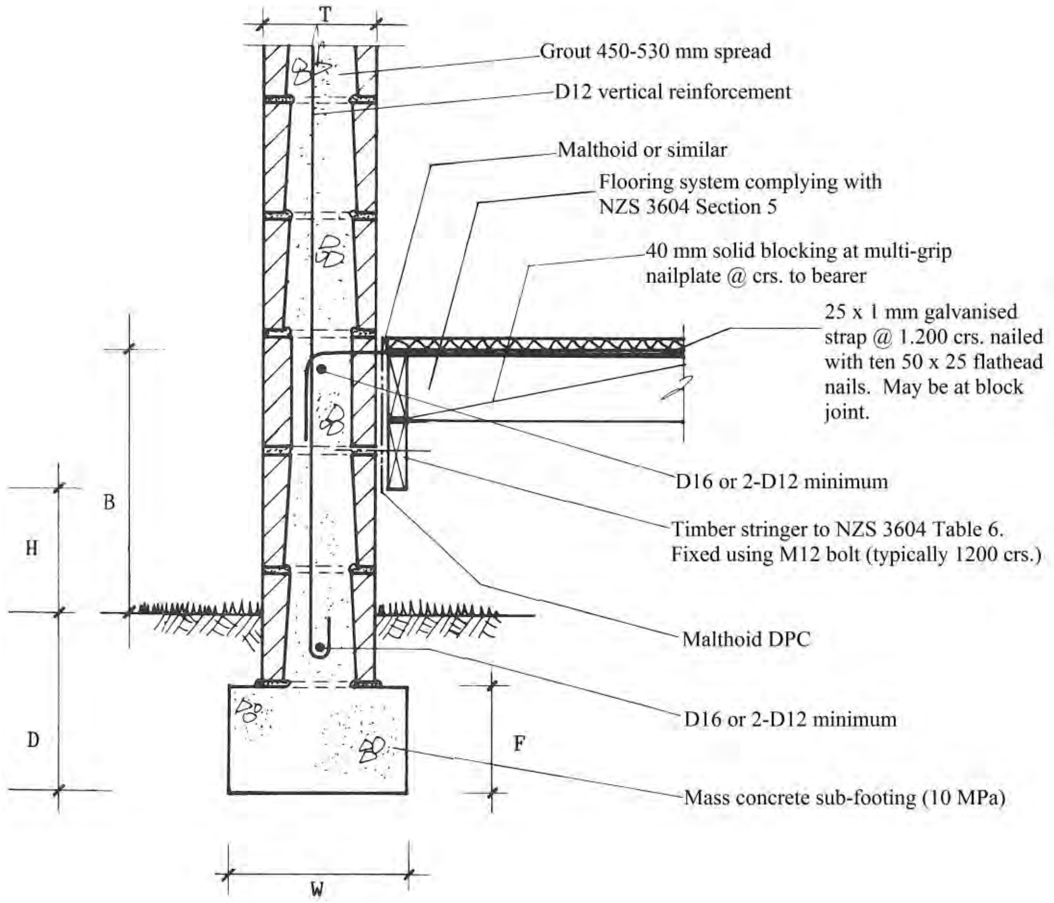
F4.6 Where practical bring slab D.P.M. up the face of framing and fasten behind wall building paper. Overlap by at least 75 mm.

F4.7 Where integral slab prevents D.P.M. from folding up face of slab, paint edge of slab and masonry seat with bitumen emulsion to within 50 mm of footing face.

F4.8 Wall tie spacing relates to 87 mm veneer construction. Special tie conditions apply for 70 mm construction.

F5 FOUNDATION

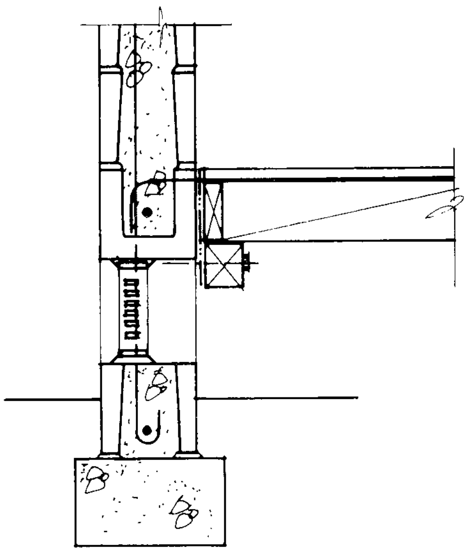
CONCRETE MASONRY WALL
TIMBER FLOOR DIAPHRAGM
MASONRY FOOTING



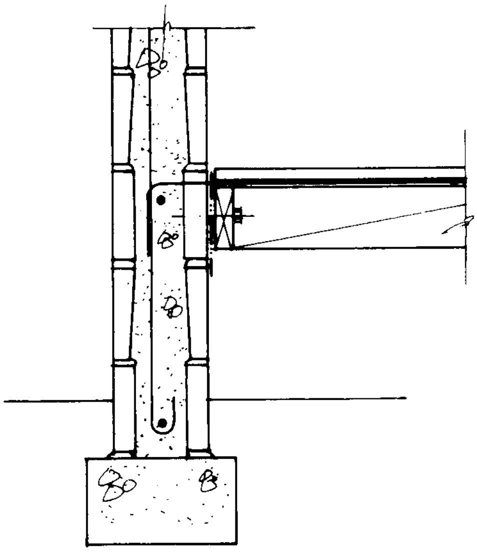
F5/A STRINGER SUPPORT

H = Bearer Height above ground:	150 mm minimum.
B = Height above ground of flooring:	450 mm minimum.
D = Foundation Depth:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Sub-footing Width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
F = Sub-footing Thickness:	150 mm or greater. See Section 6 NZS 4229.
T = Masonry Thickness:	140 for 150 series wall 190 for 20 series wall 240 for 25 series wall

F5



F5/B VENTILATOR (<1.8 M)



F5/C BOUNDARY JOISTS

F5.1 Principal units used in this detail:

- Footing - Type 16 inverted
- Partially filled construction - Type 01; 05; 14
- Solid filled construction - Type 16; 05

F5.2 Minimum sub-floor clearances are shown.

In addition, a crawl space of 450 min. height shall be provided to permit visual inspection. The 150 minimum height corresponds to the minimum height of masonry piles. Timber piles must extend 300 mm above ground level.

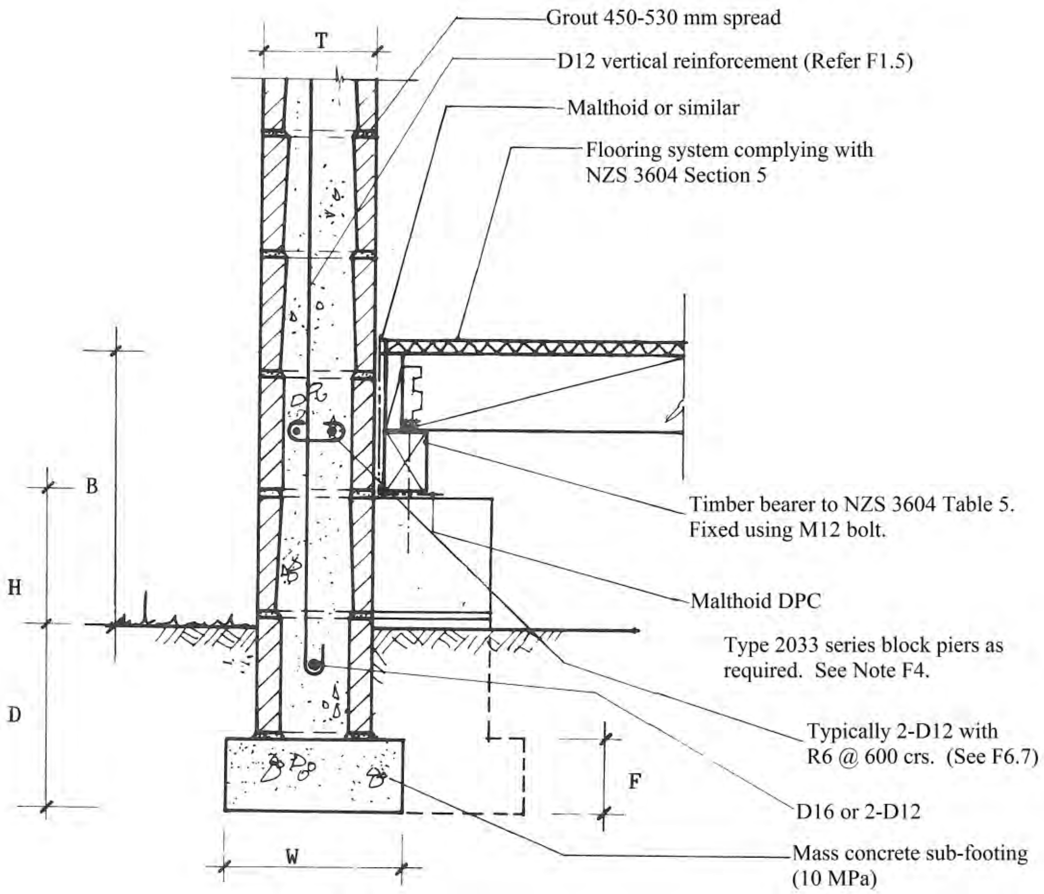
F5.3 Foundation plinths formed by 2033 'L' shaped units can be used as an alternative to bolting bearer to wall.

F5.4 In partially filled construction weepholes, in vertical joints between filled cells, are to be provided above bond beams.

F5.5 For diaphragm limitations refer, to Detail I2.

F6 FOUNDATION

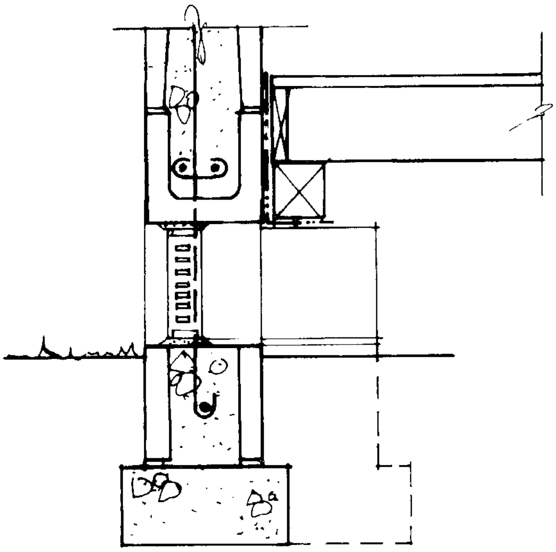
CONCRETE MASONRY WALL
(with BOND BEAM)
TIMBER FLOOR



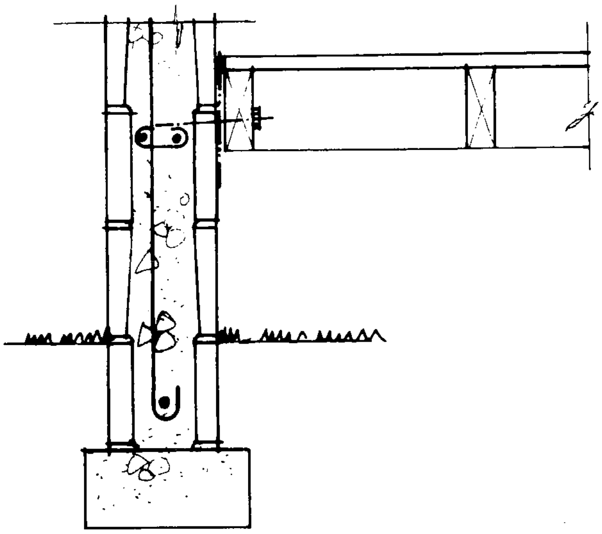
F6/A BEARERS ON MASONRY PLINTHS

H = Bearer Height above ground:	150 mm minimum.
B = Height above ground of flooring:	450 mm minimum.
D = Foundation Depth:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Sub-footing Width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
F = Sub-footing Thickness:	150 mm minimum. See Section 6 NZS 4229.
T = Masonry Thickness:	140 for 150 series wall 190 for 20 series wall 240 for 25 series wall

F6



F6/B VENTILATOR



F6/C BOUNDARY JOIST

F6.1 Principal units used in this detail:

- Footing - Type 16 inverted
- Partially filled construction - Type 01; 05; 14
- Solid filled construction - Type 16; 05

F6.2 Bearers, blocking and boundary joists separated from blockwork by malthoid.

F6.3 Foundation plinths as required for bearers.

F6.4 Minimum sub-floor clearances are shown. In addition, a crawl space of 450 minimum height shall be provided to permit visual inspection. The 150 minimum height corresponds to the minimum height of masonry piles. Timber piles must extend 300 mm above ground level.

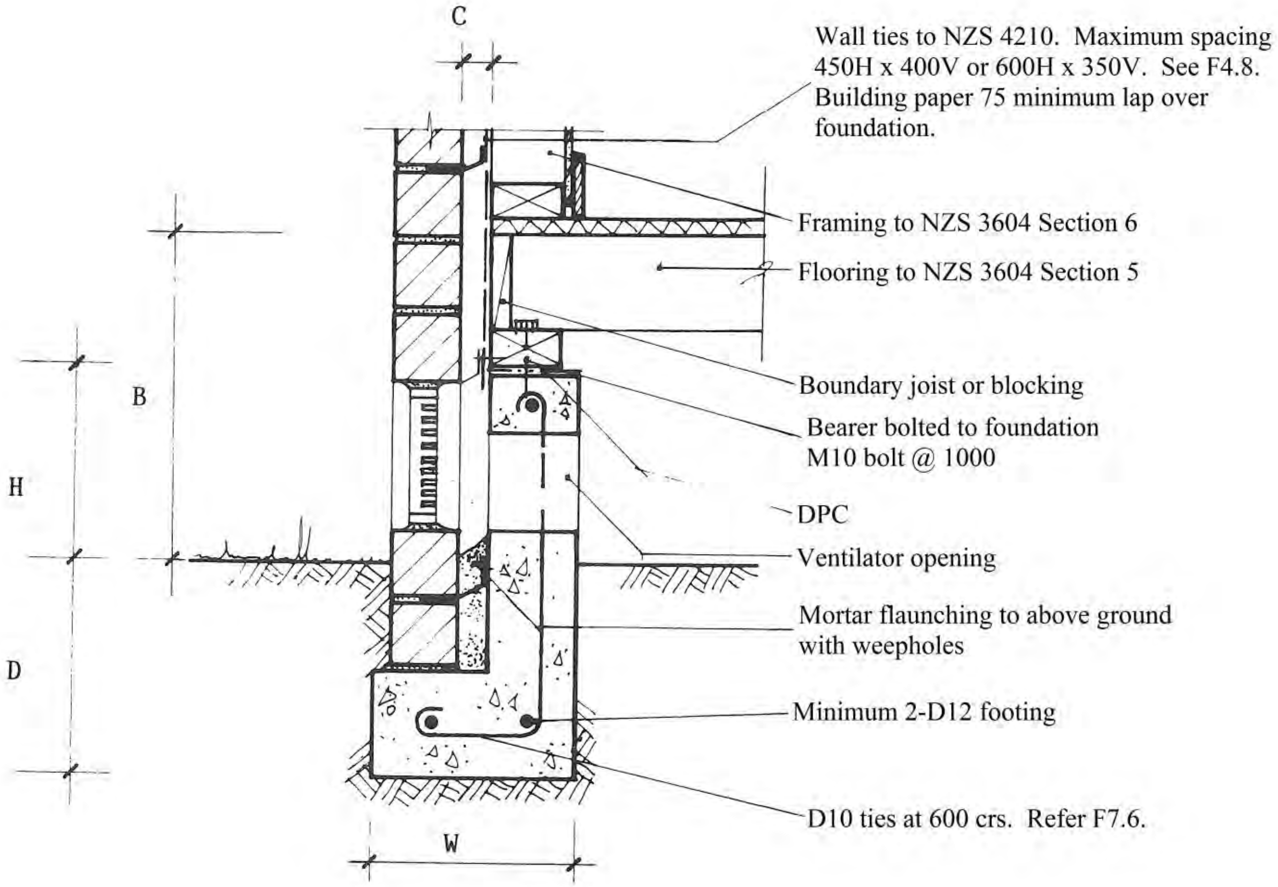
F6.5 Bearers may be bolted onto wall instead of 2033 plinths. Bond beam restraints as required by NZS 4229 Section 10.

F6.6 In partially filled construction, weepholes in vertical joints between filled cells are to be provided above bond beams.

F6.7 Bond beam reinforcement determined from NZS 4229 Table 10.1.

F7 FOUNDATION

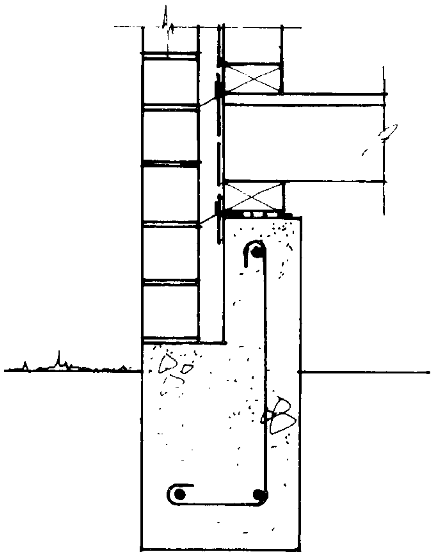
MASONRY VENEER
TIMBER FRAMING
TIMBER FLOOR
CONCRETE FOOTING



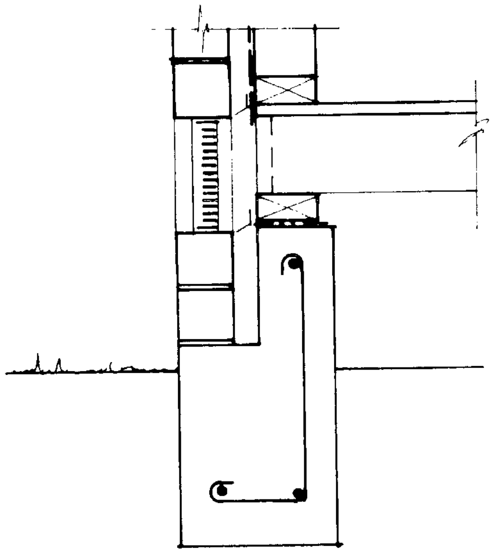
F7/A MASONRY BELOW GROUND LEVEL

H = Bearer Height above ground:	150 mm minimum.
B = Height above ground of flooring:	450 mm minimum.
D = Foundation Depth:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Sub-footing Width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
C = Cavity Width:	40-75 mm, NZS 4210

F7



F7/B MASONRY VENEER ABOVE GROUND



F7/C VENTILATOR (< 1.8 M)

—·—·— D.P.M. – – – Building Paper —··— Malthoid D.P.C. XXXXXX Bitumen Coating

F7.1 Ventilation of cavity to exterior at top and bottom required.

F7.2 Grilled sub-floor ventilators required at 1.800 m crs. maximum.

F7.3 Where veneer extends below finished ground level, raise flaunching in cavity above ground and provide external slope.

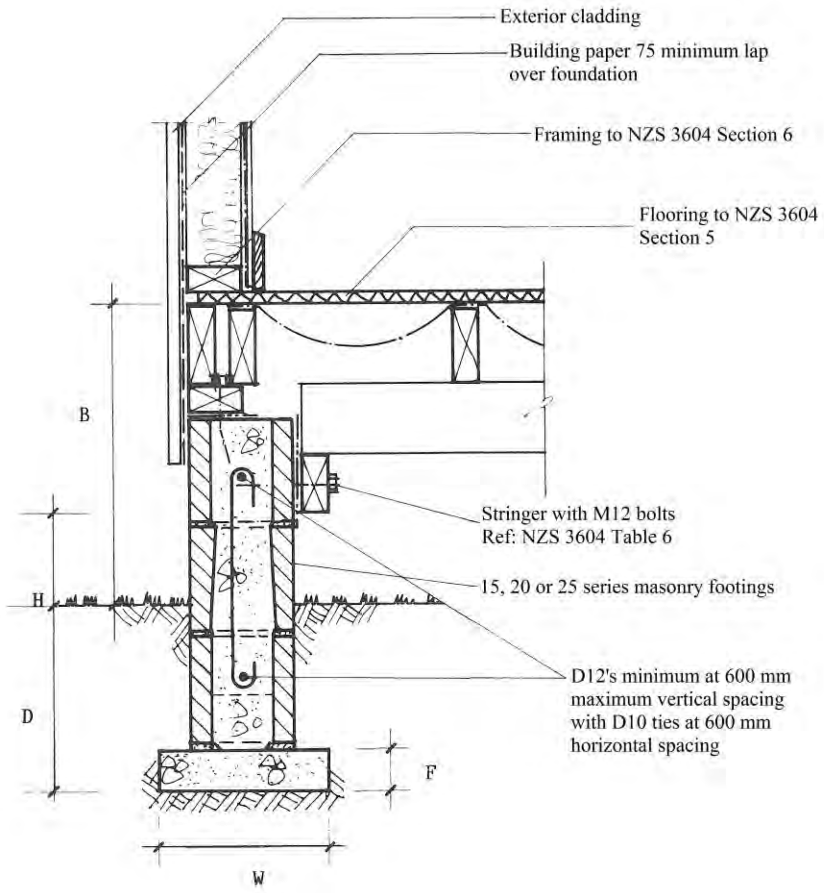
F7.4 Provide weepholes in vertical joints above flaunching or seating at 800 mm maximum crs.

F7.5 Extend building paper a minimum of 75 mm below bearer.

F7.6 Horizontal steel typically D10's @ 600 crs.

F8 FOUNDATION

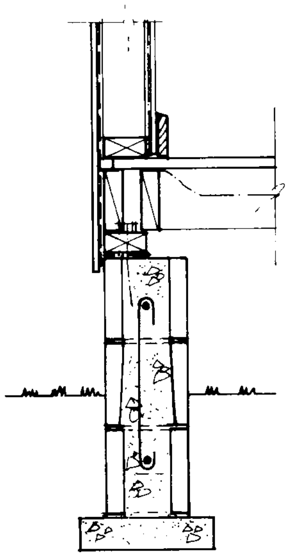
TIMBER FRAME WALL
TIMBER FLOOR
CONCRETE MASONRY FOOTING



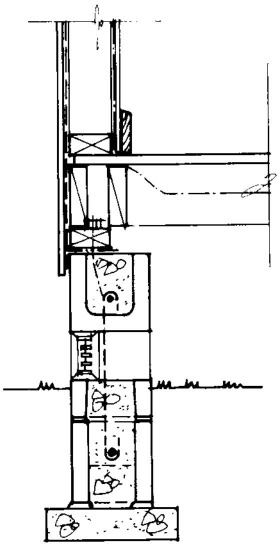
F8/A STRINGER SUPPORT

H = Bearer Height above ground:	150 mm minimum.
B = Height above ground of flooring:	450 mm minimum.
D = Foundation Depth:	Typically 300 mm depending on soil type. See Section 3 NZS 4229.
W = Sub-footing Width:	Minimum 300 mm depending on wall loading. See Section 6 NZS 4229.
F = Sub-footing:	60 mm minimum. 17.5 MPa concrete.

F8



F8/B FRAMING PLATE



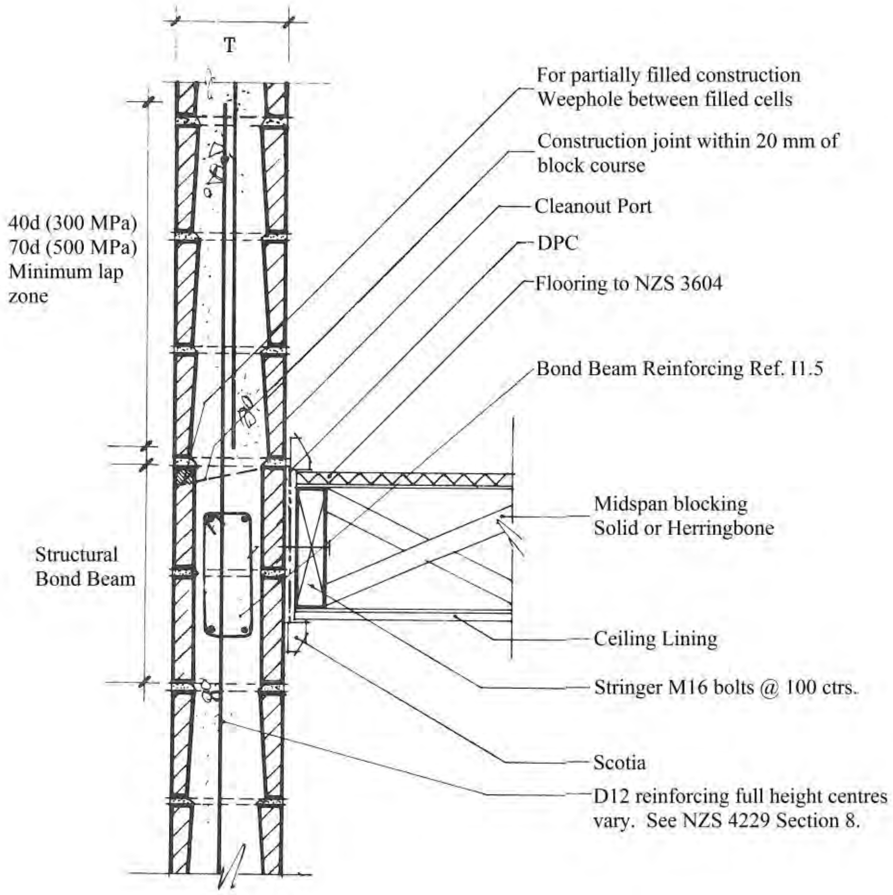
F8/C VENTILATOR

F8.1 Principal masonry units used are Type 16 units.

F8.2 Where flooring diaphragm action is required, see NZS 3604 for fixing requirements.

I1 INTERMEDIATE FLOOR

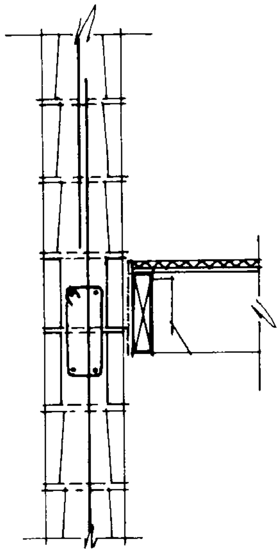
CONTINUING CONCRETE MASONRY WALL
STRUCTURAL BOND BEAM



I1/A FLOOR CONNECTIONS (PARALLEL TO JOISTS)

T = Wall Thickness:	140 mm for 15 series 190 mm for 20 series 240 mm for 25 series

F7



I1/B STRINGER SUPPORTING JOISTS

I1.1 Provide lower wall construction joint within 20 mm of top of block course. Strike with outward fall where practical.

I1.2 In solid filled construction inverted Type 16 blocks for lower course of upper wall.

I1.3 In partially filled construction provide weepholes and cleanout ports midway between filled cells and cleanouts at vertical reinforcement.

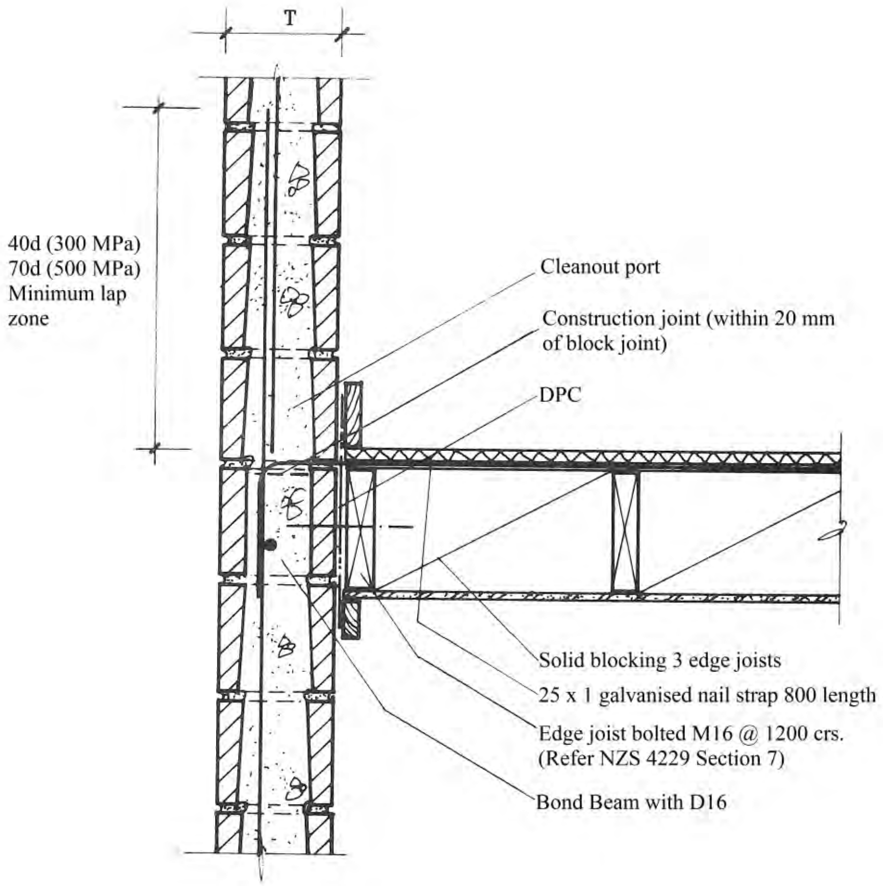
I1.4 Separate all timber from masonry with DPC.

I1.5 Refer to NZS 4229 Table 10.1 for reinforcement and permissible spans of bond beam.

I1.6 Wall thickness may change at junction from thicker lower to thinner upper wall.

I2 INTERMEDIATE FLOOR

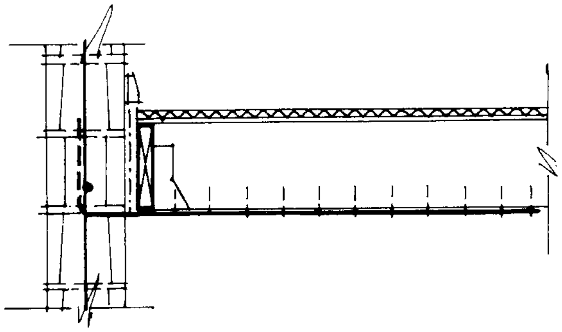
CONTINUING CONCRETE MASONRY WALL
TIMBER DIAPHRAGM FLOORING



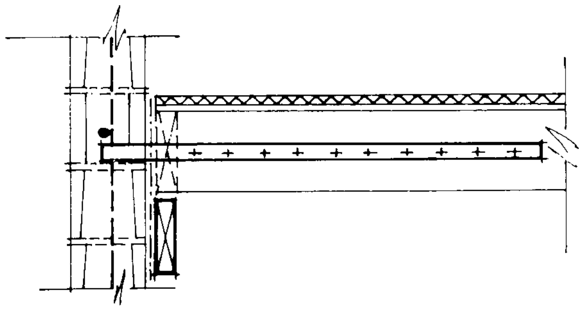
I2/A BOUNDARY JOIST SUPPORT

T = Masonry Thickness

I2



I2/B STRINGER SUPPORTING JOISTS



I2/C STRINTER BELOW INTERMITTENT BOCKING

I2.2 Structural diaphragm must comply with the following:

(a) length between supports shall be less than twice depth;

(b) be of running bond pattern sheet material(c)sheets shall be greater than 1,800 x 900;

(c) sheets shall be fastened along each edge to boundary members at centres specified (typically 3.15 mm dia. nails at 150 crs.) and every intermediate member at 300 crs;

(e) all sheets to lap over a 180 x 50 member;

(f) fasteners shall not be less than 10 mm from edge. See also NZS 3604 requirements.

I2.3 Floor diaphragms shall not span more than16 mm between supports.

I2.4 25 mm x 1 mm galvanised nail strap to be twisted around vertical steel and cast with lower wall grout lift.

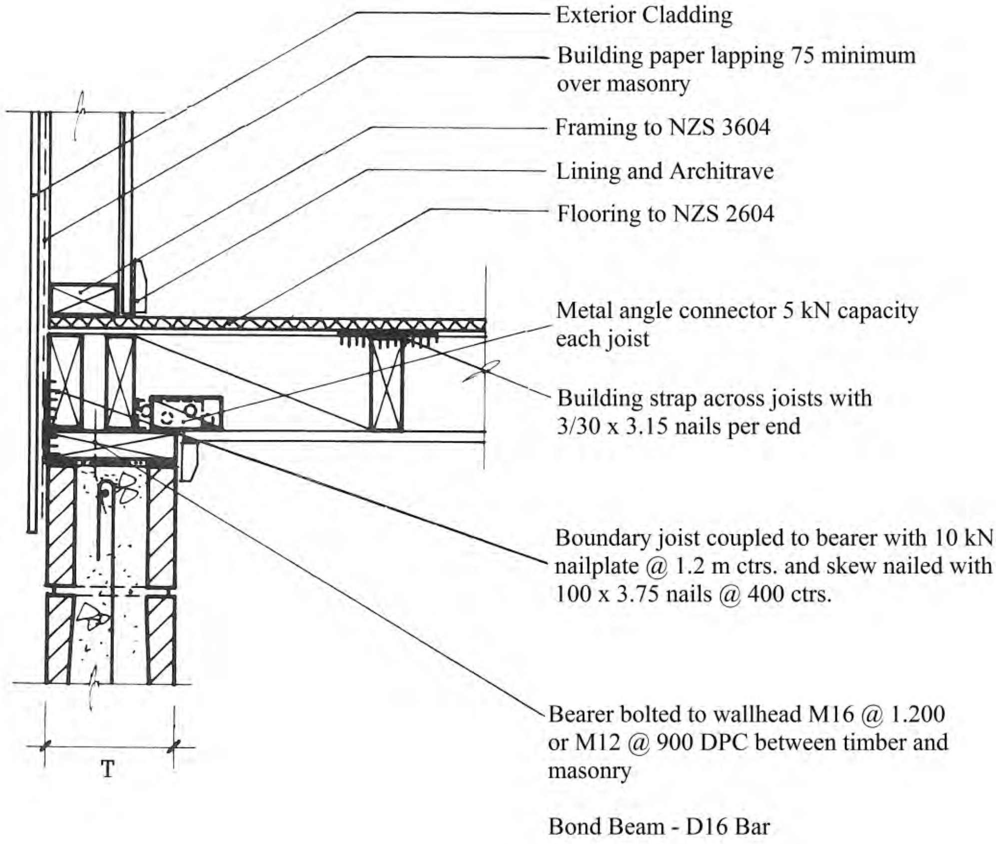
I2.5 25 x 1 galvanised nail strap to be nailed to joist or blocking for minimum of 800 mm with no less than ten 50 x 25 flathead nails.

I2.6 25 x 1 galvanised nail strap may extend along upper or lower face of the joists.

I2.7 Wall thickness may change at junction from thicker lower to thinner upper wall.

I3 INTERMEDIATE FLOOR

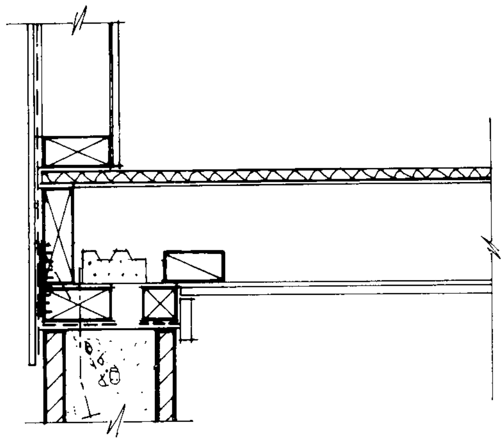
TIMBER FRAMED WALL ABOVE
TIMBER DIAPHRAGM
CONCRETE MASONRY WALL BELOW



I3/A PARALLEL WITH JOISTS

T = Masonry Thickness

I3



I3/B SUPPORTING FLOOR JOISTS

I3.1 Continuous plate required to be bolted to top of the masonry wall with Ml6 bolts at 1200 mm crs. Or M12 bolts at 900 mm crs.

I3.2 Plates shall be continuous over minor openings and be fastened to the wall within 200 mm of the opening extent.

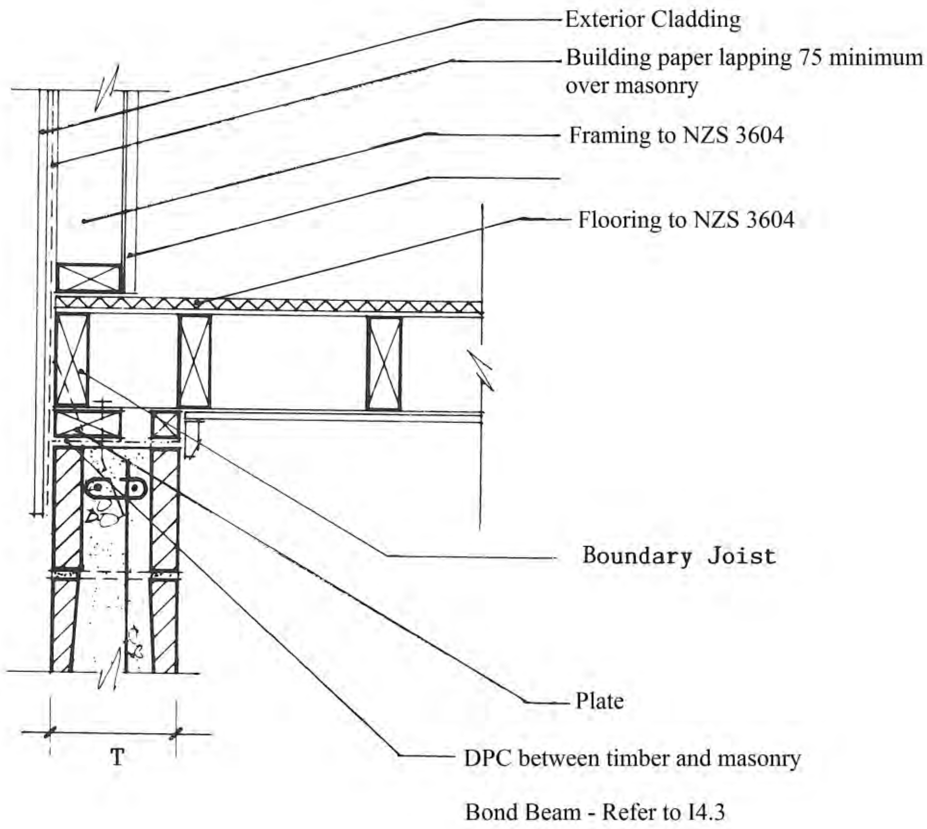
I3.3 The boundary joist or blocking is to be fastened to the plate by 100 x 3.75 skew nails at 400 crs. and by nail plates (10 kw capacity) at 1.2 m crs.

I3.4 Refer 12.2 and 12.3 for details of diaphragm dimensions and fixings.

I3.5 For details of the bond beam requirements for a diaphragm system refer, NZS 4229 Section 10.

I4 INTERMEDIATE FLOOR

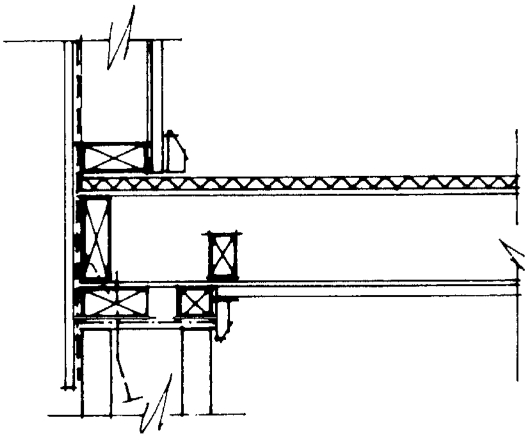
TIMBER FRAMED WALL ABOVE
STRUCTURAL BOND BEAM
CONCRETE MASONRY WALL BELOW



I4/A PARALLEL WITH JOISTS

T = Masonry Thickness

I4



I4/B SUPPORTING FLOOR JOISTS

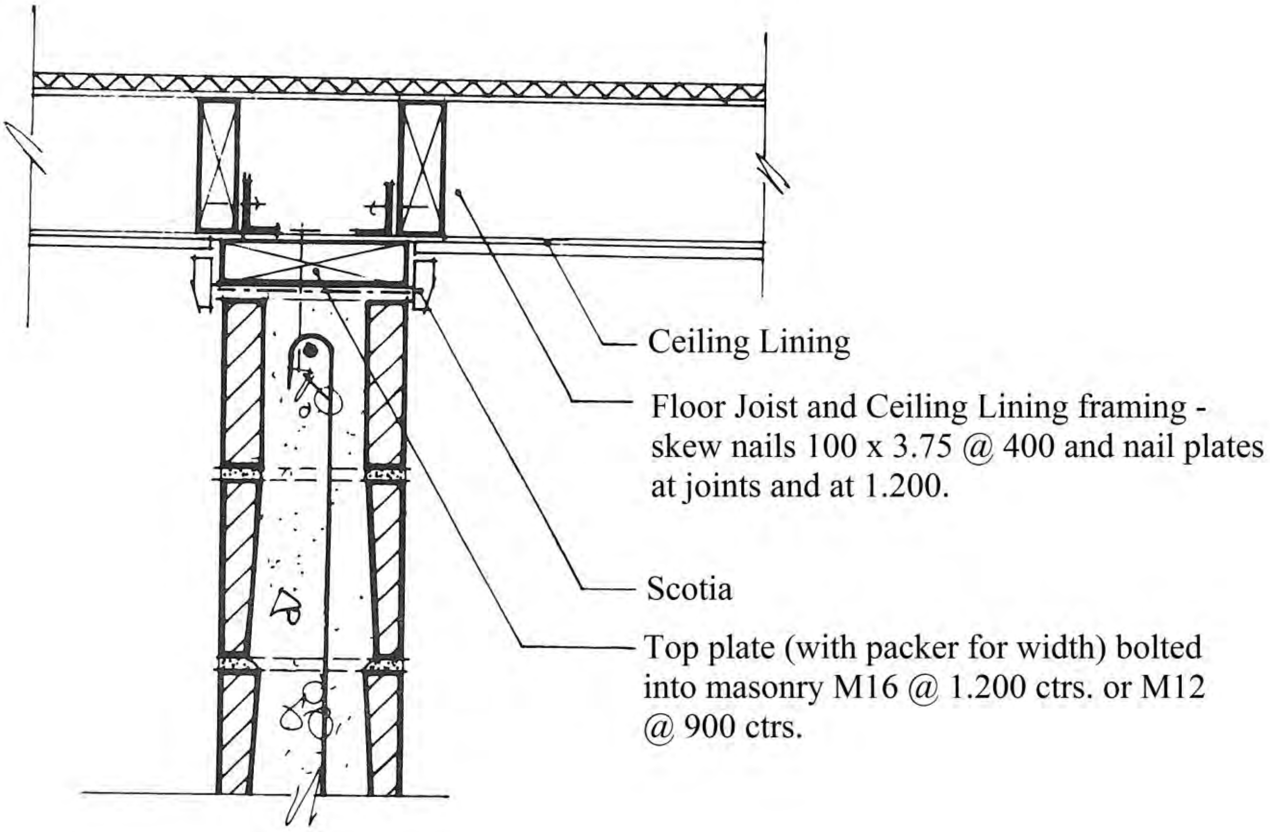
I4.1 Refer to Notes on I1for masonry details.

I4.2 Continuous plate required to be bolted to top of the masonry wall with M10 bolts or dowels at 1.4 maximum crs.

I4.3 Refer to Table 10, NZS 4229 for top bond beam reinforcing.

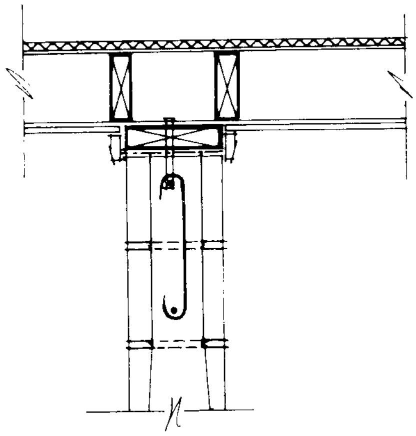
I5 INTERMEDIATE FLOOR

LOWER CONCRETE WALL
(with DIAPHRAGM TO FLOOR)
TIMBER FLOOR



I5/A PARALLEL WITH FLOOR JOISTS

I5



I5/B PERPENDICULAR TO JOISTS

I5.1 Continuous plate required to be bolted to top of the masonry wall with M16 bolts at 1200 mm crs or M12 bolts at 900 mm crs.

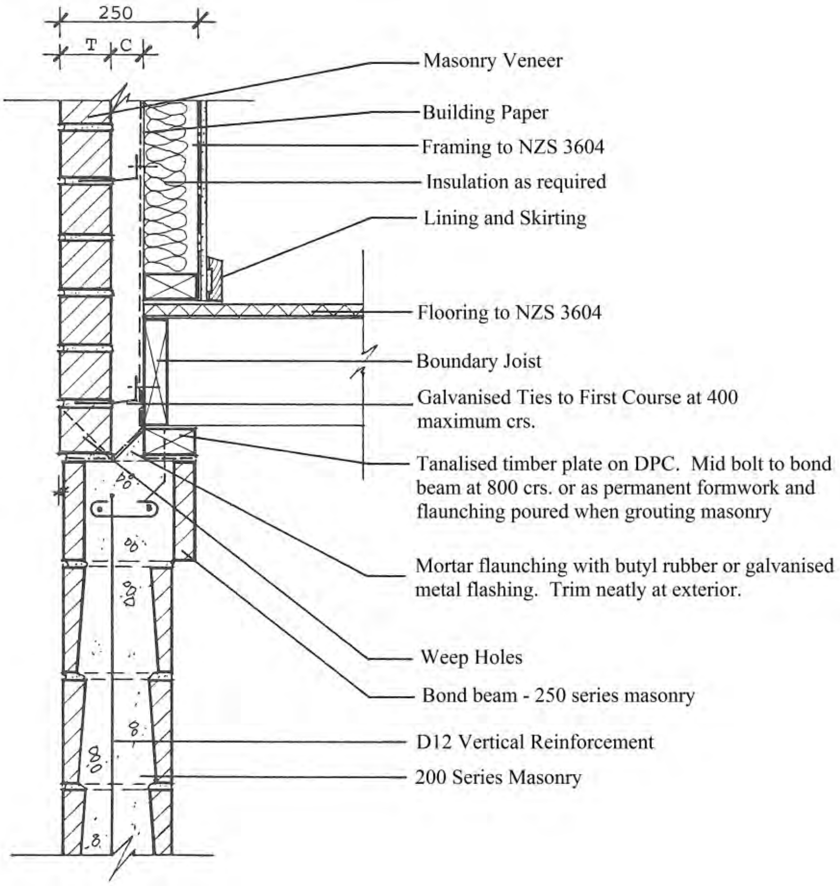
I5.2 Plates shall be continuous over minor openings and be fastened to the wall within 200 mm of the opening extent.

I5.3 The boundary joist or blocking is to be fastened to the plate by 100 x 3.75 skew nails at 400 crs. and by nail plates (10 kN capacity) at 1.2 m crs.

I5.4 Refer 12.2 and 12.3 for details of diaphragm restraints.

I6 INTERMEDIATE FLOOR

MASONRY VENEER
TIMBER FRAMING
TIMBER FLOOR
CONCRETE MASONRY WALL



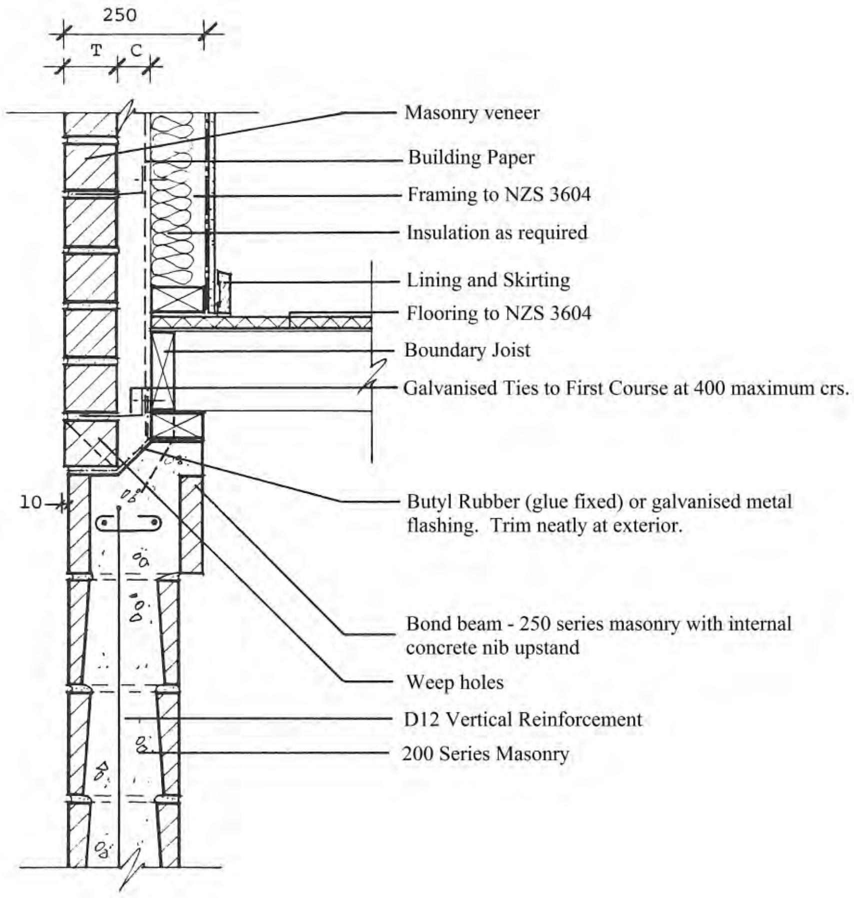
I6/A

T = Veneer Thickness: 70 mm minimum.

C = Cavity: 40-75 mm.

I6

ALTERNATIVE CONCRETE UPSTAND



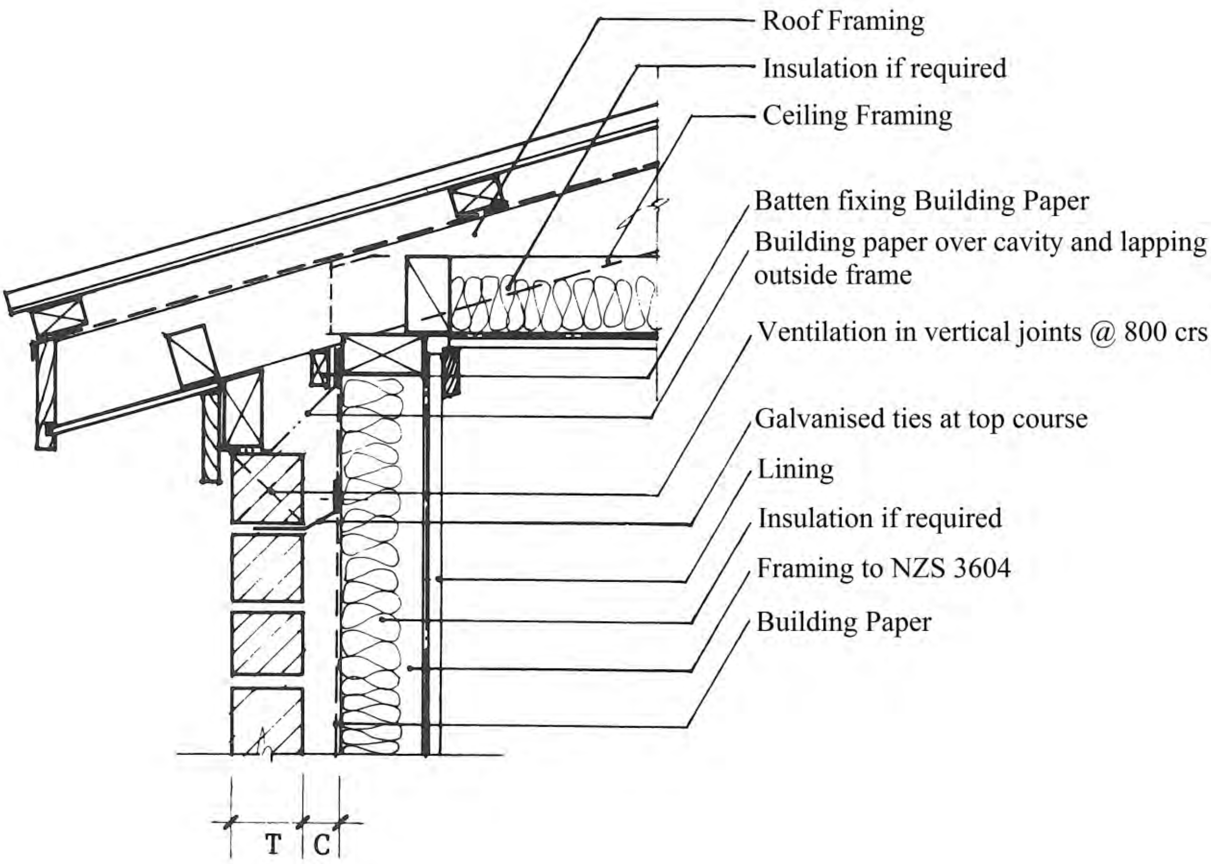
I6/B

T = Veneer Thickness: 70 mm minimum.

C = Cavity: 40-75 mm.

R1 ROOF

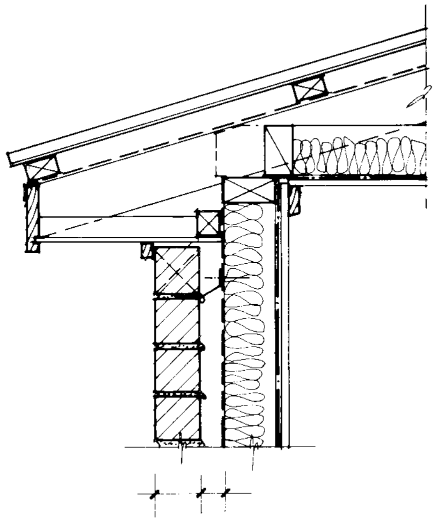
MASONRY VENEER CLADDING
TIMBER FRAMING
HORIZONTAL CEILING



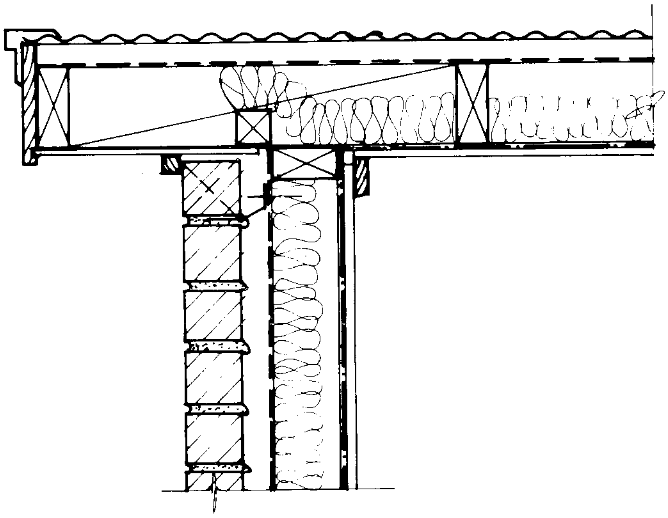
R1/A SLOPING SOFFIT

T = Veneer Thickness:	70 mm minimum.
C = Cavity	Not less than 40 mm, NZS 4210. Not more than 75 mm.

R1



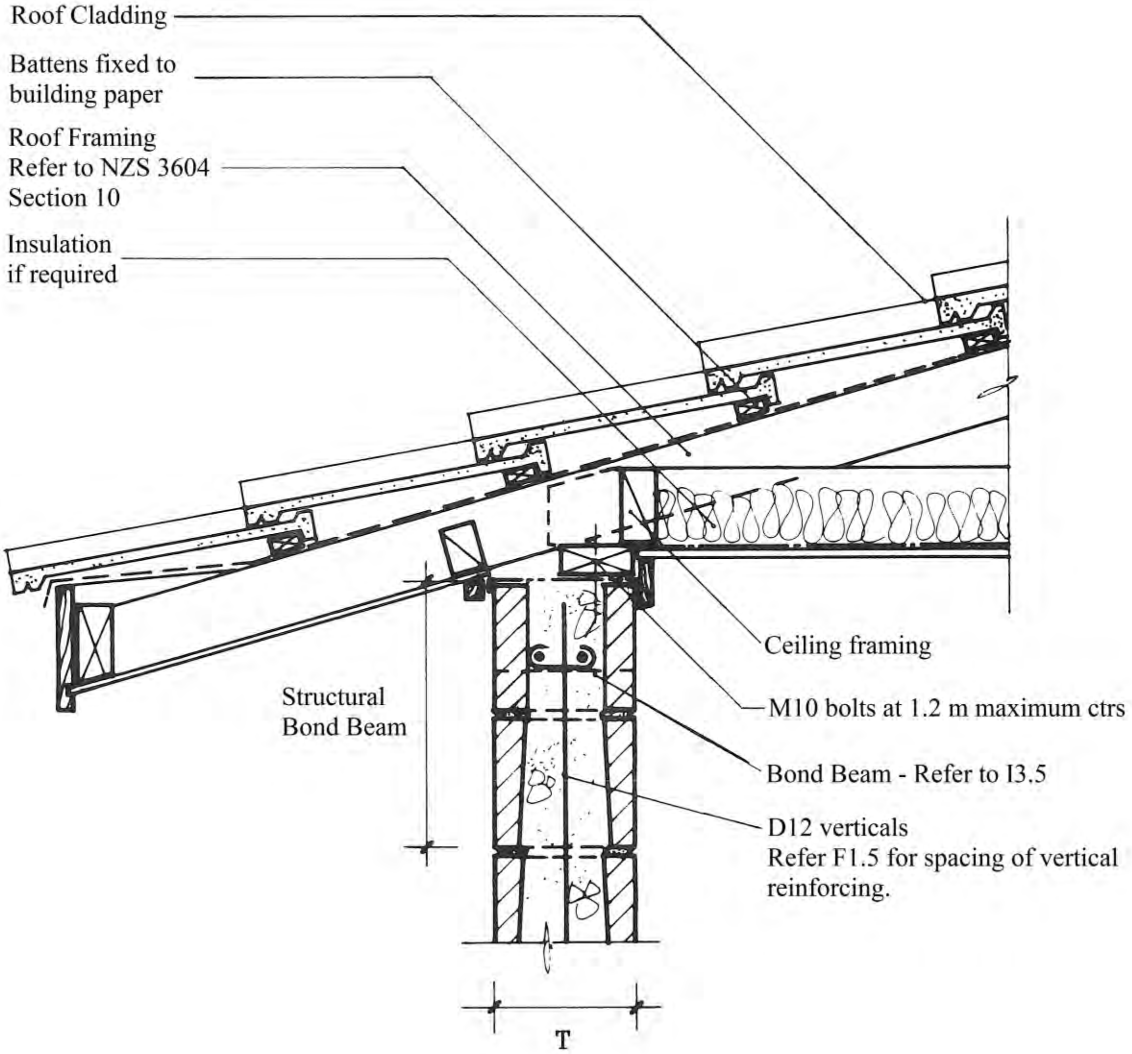
R1/B HORIZONTAL SOFFIT



R1/C GABLE WALL

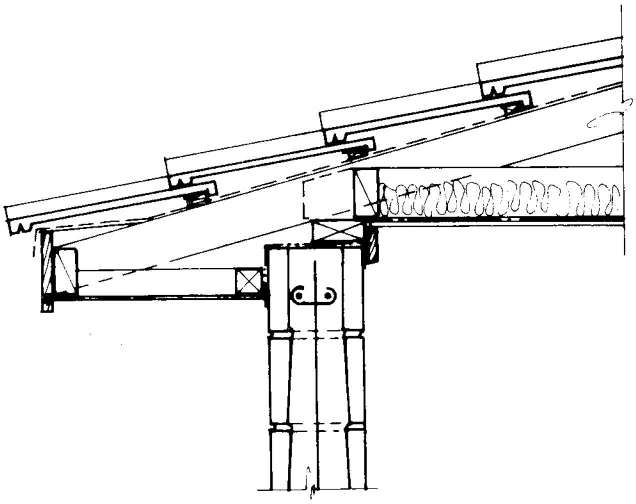
R2 ROOF

CONCRETE MASONRY WALL
WITH TOP BOND BEAM
HORIZONTAL CEILING

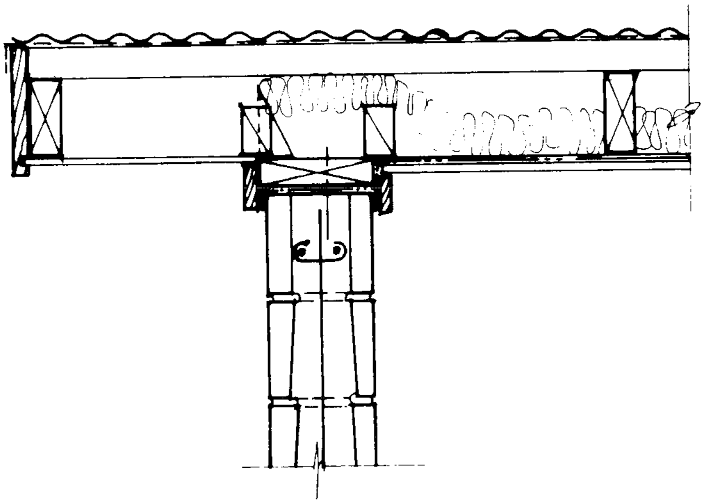


R2/A SLOPING SOFFIT

R2



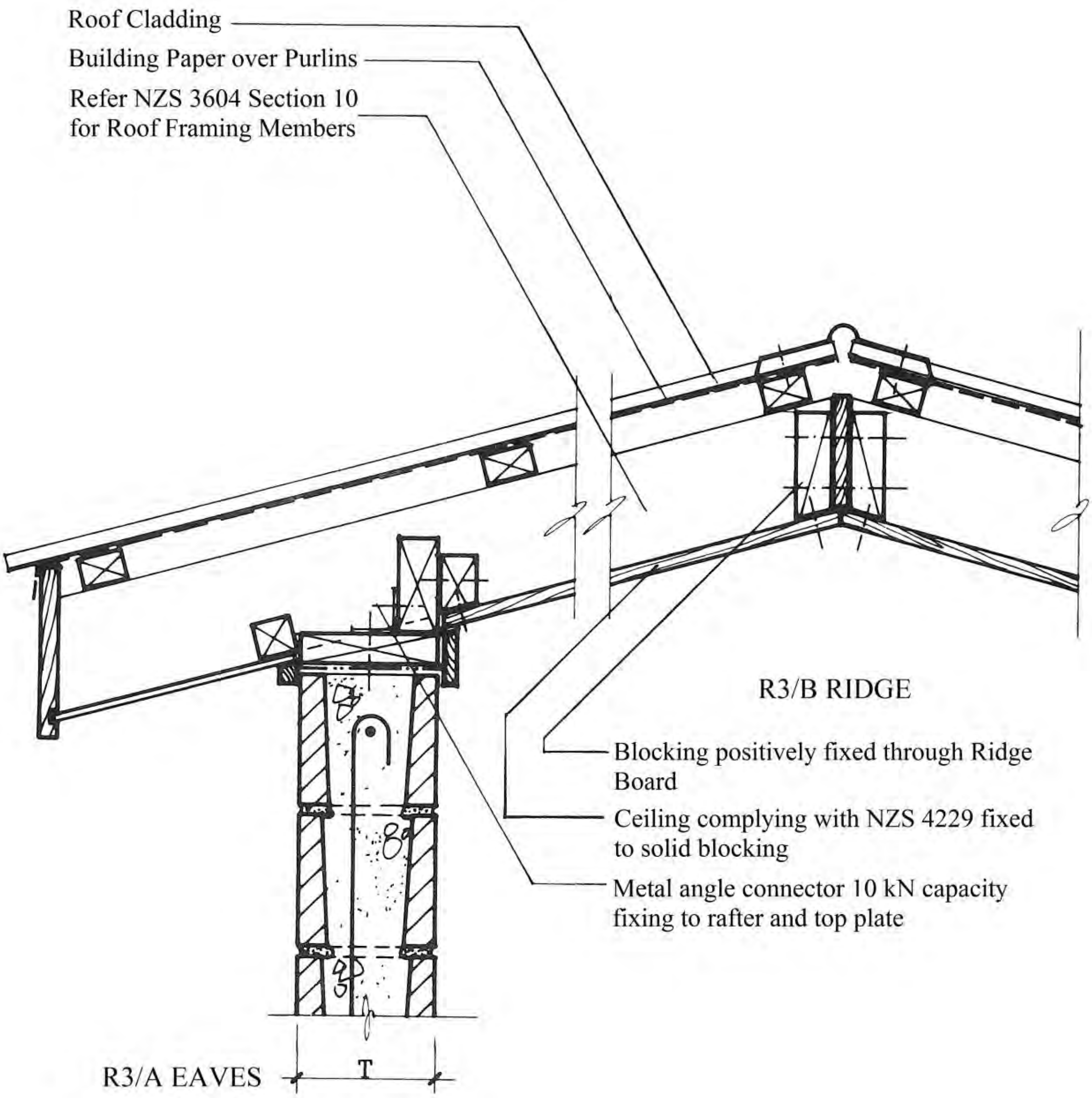
R2/B HORIZONTAL SOFFIT



R2/C GABLE WALL

R3 ROOF

CONCRETE MASONRY WALL
SLOPING DIAPHRAGM CEILING TO
UNDERSIDE OF RAFTERS

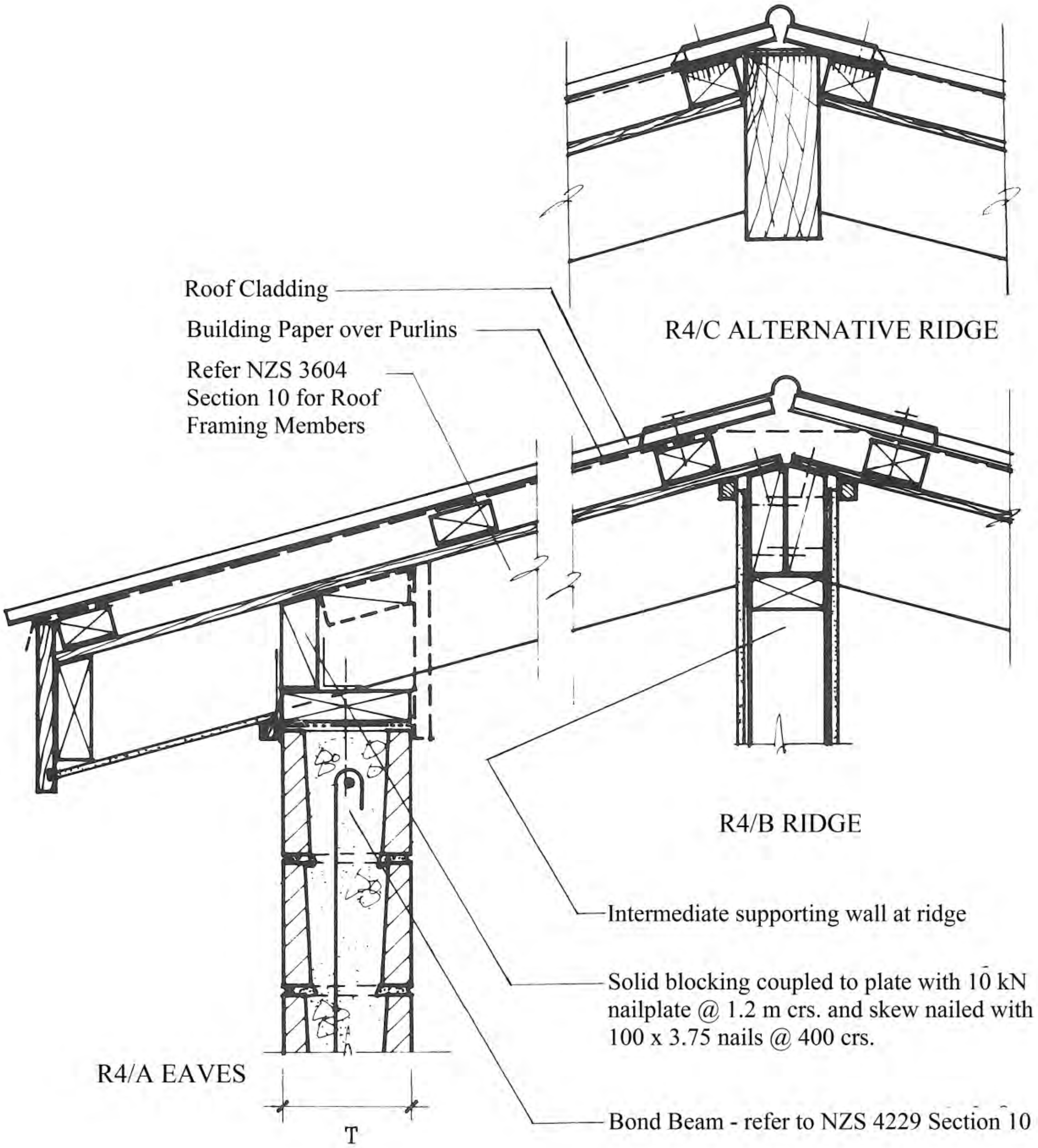


R3 DIAPHRAGM CEILING UNDERSIDE OF RAFTERS

T = Masonry Thickness

R4 ROOF

CONCRETE MASONRY WALL
SLOPING DIAPHRAGM CEILING
EXPOSED RAFTERS



R4 DIAPHRAGM CEILING UP TO TOP OF RAFTERS

T = Masonry Thickness

3.4 Compressive Strengths of Masonry Prisms

Introduction

The current situation in relation to the permitted ultimate compressive strength values f'_m in *NZS 4230 Design of reinforced concrete masonry structures* is summarised as follows:

Class A No limit, but values beyond 12 MPa must be confirmed via prism testing. (Supervised construction).

Class B 12 MPa. (Construction observation)

Class C 4 MPa. (No construction observation)

The concept of prism testing (3 units high) was not considered to be a viable control method, but in order to provide designers with some information as to the ultimate compressive strength of masonry, a series of prism tests were carried out (*NZCRA Report SLR 24*).

The New Zealand Concrete Research Association was particularly conscious NOT to have artificially raised the level of the recorded strength through specific laboratory procedures not representative of practical blocklaying and grouting. The results therefore clearly show the previous conservative nature of the Standards document prior to *NZS 4320*.

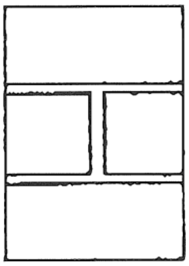
While the results in themselves are probably insufficient in number to form a basis upon which to propose a revised strength appraisal method, they have been passed to the University of Canterbury for inclusion within a new strength evaluation method which is now contained in Appendix B of *NZS 4230*.

It is considered that the results obtained should provide designers with a broad independent assessment of the actual compressive strengths likely to derive from grout filled concrete masonry in the Auckland, Wellington and Christchurch areas.

Summary of Results

Result 1:

Compression tests were carried out on 3 unit high prisms laid up as shown using results 2016 type units.



Result 2:

Three different sources of masonry units were used from Auckland, Wellington and Christchurch.

The mean compressive strengths of the individual masonry blocks from each consignment were measured in accordance with NZS 3102P:1974.

The values were:

Auckland	17.1 MPa
Wellington	26.5 MPa
Christchurch	25.6 MPa

Result 3:

Two types of mortar were used - X and Y:

Mix Design	X	Y
Sand/cement (by weight)	3.29	4.92
Water/cement (by weight)	0.83	0.97
Admixture (ml/kg)cement	2.80	2.80
Flow after suction* (% of initial flow)	69.0	51.0
28 day compressive strength**	19.5	16.0
Admixture (ml/kg)cement	2.80	2.80

* ***As per Clause 23 of ASTM C91***

** ***As per NZS 3112:1980 Part 2***

Result 4:

Two types of grouts were used - 17.5 MPa (Type A) and 20 MPa (Type B). Both grouts were proportioned to give spread values of between 490 mm and 510 mm. (NZS 4210P spread requirements 450 mm to 530 mm).

Result 5:

108 three high grouted prisms were tested to cover the various combinations of manufacture source, mortars and grout strengths.

The 7 day prism strengths can be summarised as:

Designation	Individual Prism Strengths (MPa)	Mean Prism Strength (MPa)	7 Day Grout Strength (MPa)	28 Day Grout Strength (MPa)
AXA1	12.3 13.5 13.7	13.2	11.0	18.0
AXA2	11.9 14.0 13.2	13.0	10.5	19.0
AXA3	13.6 12.3 11.8	12.6	11.5	19.5
AXB1	13.7 14.2 12.9	13.6	12.0	20.0
AXB2	14.1 14.3 13.6	14.0	13.0	22.5
AXB3	15.1 13.8 14.4	14.4	12.5	21.0
AYA1	13.2 12.7 13.4	13.1	10.5	16.5
AYA2	13.7 10.8 13.1	12.5	11.0	17.5
AYA3	12.6 12.9 13.3	12.9	9.5	18.0
AYB1	13.0 13.5 14.2	13.6	12.0	21.5
AYB2	13.9 14.5 15.1	14.5	11.5	20.5
AYB3	13.8 15.0 14.7	14.5	13.0	22.5
WXA1	13.2 13.0 13.7	13.3	10.5	18.5
WXA2	13.9 12.7 14.0	13.5	10.0	19.0
WXA3	14.0 14.2 16.7	15.0	11.5	19.5
WXB1	13.2 12.9 13.6	13.2	10.5	19.0
WXB2	11.7 14.3 14.8	13.6	11.0	20.
WXB3	17.4 16.5 16.5	16.8	12.5	21.5

Designation	Individual Prism Strengths (MPa)	Mean Prism Strength (MPa)	7 Day Grout Strength (MPa)	28 Day Grout Strength (MPa)
WYA1	12.9 13.3 12.9	13.0	9.5	17.0
WYA2	13.8 12.8 13.4	13.3	10.5	19.0
WYA3	14.2 13.8 13.6	13.9	10.5	18.0
WYB1	14.6 15.3 13.9	14.6	12.5	22.5
WYB2	14.9 14.9 15.8	15.2	12.0	19.5
WYB3	13.8 14.6 13.3	13.9	11.0	19.5
CXA1	12.7 11.8 13.0	12.5	9.5	17.0
CXA2	16.2 15.7 14.6	15.5	11.5	18.5
CXA3	13.1 13.0 13.9	13.3	11.0	18.5
CXB1	14.2 13.6 13.7	13.8	12.0	20.5
CXB2	15.6 16.2 14.9	15.6	12.5	21.0
CXB3	14.8 13.2 15.1	14.4	12.5	21.5
CYA1	12.9 13.2 13.5	13.2	8.5	15.0
CYA2	13.7 14.3 12.9	13.6	10.0	17.5
CYA3	14.1 13.2 13.7	13.7	11.0	18.5
CYB1	15.6 14.9 15.4	15.3	12.0	20.5
CYB2	14.7 16.6 13.2	14.8	13.0	22.5
CYB3	17.0 16.5 16.7	16.7	13.0	21.0

Each set of prisms is identified as follows:

1. ***First code letter represents source of masonry units – A = Auckland, W = Wellington, C = Christchurch.***
2. ***Second code letter represents type of mortar – X or Y.***
3. ***Third code letter represents grout type – A or B.***
4. ***Final number is replicate set number.***

Note: NZS 3102P has been superceded by *AS/NZS 4455*, and *NZS 4230* was introduced in 2004 replacing DZ 4210 Part B. The prism test results of 1983 remain valid and were used to complete Appendix B of *NZS 4230:2004*.

4.1 Design of Reinforced Concrete Masonry Structures

Users Guide to NZS 4230:2004

Contents

Acknowledgement

This user guide was written by Jason Ingham and Kok Choon Voon, Department of Civil and Environmental Engineering, University of Auckland.

The authors wish to acknowledge the role of Standards New Zealand and of the committee members responsible for drafting NZS 4230:2004.

The authors wish to thank David Barnard and Mike Cathie for their assistance in formulating the design notes and in development of the design examples included in this guide.

Dr Peter Laursen and Dr Gavin Wight are thanked for their significant contributions pertaining to the design of unbonded post-tensioned masonry walls.

It is acknowledged that the contents of this user guide, and in particular the design examples, are derived or adapted from earlier versions, and the efforts of Emeritus Professor Nigel Priestley in formulating those design examples is recognised. It is acknowledged that the strut-and-tie model in section 3.8 is an adaption of that reported in Paulay and Priestley (1992).

The user guide was reviewed and revised by Dr Jason Ingham in 2012.

1.0 Introduction

NZS 4230 is the materials standard specifying the design and detailing requirements for masonry structures. The current version of this document has the full title '**NZS 4230:2004 Design of Reinforced Concrete Masonry Structures**'. The purpose of this user guide is to provide additional information explaining the rationale for new or altered clauses within this version of the Standard with respect to its predecessor versions, and to demonstrate the procedure in which it is intended that NZS 4230:2004 be used.

1.1 Background

The New Zealand masonry design standard was first introduced in 1985 as a provisional Standard NZS 4230P:1985. This document superseded NZS 1900 Chapter 9.2, and closely followed the format of NZS 3101 'Code of practice for the design of concrete structures'. The document was formally introduced in 1990 as NZS 4230:1990.

Since 1985 NZS 4230 was subject to significant amendment, firstly as a result of the publication of the revised loadings standard, NZS 4203:1992. This latter document contained major revisions to the formatting of seismic loadings, which typically are the structural design actions that dominate the design of most New Zealand concrete masonry structures. NZS 4203:1992 was itself revised with the introduction of the joint loadings standard AS/NZS 1170, with the seismic design criteria for New Zealand presented in part 5 or NZS 1170.5.

1.2 Related Standards

Whilst a variety of Standards are referred to within NZS 4230:2004, several documents merit special attention:

- As noted above, NZS 4230:2004 is the material design standard for reinforced concrete masonry, and is to be used in conjunction with the appropriate loadings standard defining the magnitude of design actions and loading combinations to be used in design. Unfortunately, release of NZS 1170.5 encountered significant delay, such that NZS 4230:2004 was released before NZS 1170.5 was available. The timing of these release dates led to Amendment No. 1 to NZS 4230:2004 being issued in December 2006 to ensure consistency with AS/NZS 1170 and NZS 1170.5.

- NZS 4230:2004 is to be used in the design of concrete masonry structures. The relevant document stipulating appropriate masonry materials and construction practice is NZS 4210:2001 'Masonry construction: Materials and workmanship'.

- NZS 4230:2004 is a specific design standard. Where the structural form falls within the scope of NZS 4229:1999 'Concrete Masonry Buildings Not Requiring Specific Engineering Design', this latter document may be used as a substitute for NZS 4230:2004.

- NZS 4230:2004 is to be used in the design of concrete masonry structures. Its general form is intended to facilitate consultation with NZS 3101 'The design of concrete structures' standard, particularly for situations that are not satisfactorily considered in NZS 4230, but where engineering judgement may permit the content of NZS 3101 to indicate an appropriate solution.

2.0 Design Notes

The purpose of this chapter is to record and detail aspects of the Standard that differ from the previous version, NZS 4230:1990. While it is expected that the notes provided here will not address all potential queries, it is hoped that they may provide significant benefit in explaining the most significant changes presented in the latest release of the document.

2.1 Change of Title and Scope

The previous version of this document was titled "**NZS 4230:1990 Code of Practice for the Design of Masonry Structures**". The new document has three separate changes within the title:

- The word **Code** has ceased to be used in conjunction with Standards documents to more clearly delineate the distinction between the New Zealand Building Code (NZBC), and the Standards that are cited within the Code. NZS 4230:2004 is intended for citation in Verification Method B1/VM1 of the Approved Documents for NZBC Clause B1 "Structure".

- The previous document was effectively intended to be used primarily for the design of reinforced **concrete** masonry structures, but did not preclude its use in the design of other masonry materials, such as clay or stone. As the majority of structural masonry constructed in New Zealand uses hollow concrete masonry units, and because the research used to underpin the details within the Standard almost exclusively pertain to the use of concrete masonry, the title was altered to reflect this.

- Use of the word **reinforced** is intentional. Primarily because the majority of structural concrete masonry in New Zealand is critically designed to support seismic loads, the use of unreinforced concrete masonry is excluded by the Standard. The only permitted use of unreinforced masonry in New Zealand is as a veneer tied to a structural element. Design of masonry veneers is addressed in Appendix F of NZS 4230:2004, in NZS 4210:2001, in NZS 4229:1999 and also in NZS 3604:2011 'Timber Framed Structures'. Veneer design outside the scope of these standards is the subject of special design, though some assistance may be provided by referring to AS 3700 'Masonry Structures'.

2.2 Nature of Commentary

Much of the information in NZS 4230:1990 was a significant departure from that contained in both previous New Zealand masonry standards, and in the masonry codes and standards of other countries at that time. This was primarily due to the adoption of a limit state design approach, rather than the previous "allowable stress" method, and because the principle of capacity design had only recently been fully developed. Consequently, NZS 4230:1990: Part 2 contained comprehensive details on many aspects of structural seismic design that were equally applicable for construction using other structural materials.

Since release of NZS 4230:1990, much of the commentary details have been assembled within a text by Paulay and Priestley[1]. For NZS 4230:2004 it was decided to produce an abbreviated commentary that primarily addressed aspects of performance specific to concrete masonry.

This abbreviation permitted the Standard and the commentary to be produced as a single document, which was perceived to be preferable to providing the document in two parts. Consequently, designers may wish to consult the aforementioned text, or NZS 4230:1990:Part 2, if they wish to refresh themselves on aspects of general structural seismic design, such as the influence of structural form and geometry on seismic response, or the treatment of dynamic magnification to account for higher mode effects. In addition, care has been taken to avoid unnecessarily replicating information contained within NZS 3101, such that that Standard is in several places referred to in NZS 4230:2004.

2.3 Material Strengths

In the interval between release of NZS 4230:1990 and NZS 4230:2004 a significant volume of data has been collected pertaining to the material characteristics of concrete masonry. The availability of this new data has prompted the changes detailed below.

2.3.1 Compression Strength f'_m

The most significant change in material properties is that the previously recommended compressive strength value for Observation Type B masonry was found to be unduly conservative.

As identified in NZS 4210, the production of both concrete masonry units and of block-fill grout is governed by material standards. Accounting for the statistical relationship between the mean strength and the lower 5% characteristic strength for these constituent materials, it follows that a default value of f'_m = 12 MPa is appropriate for Observation Type B. This is supported by a large volume of masonry prism test results, and an example of the calculation conducted to establish this value is presented here in section 3.1.

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1 Paulay, T., and Priestley, M. J. N. (1992) "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, New York, 768 pp.

2.3.2 Modulus of Elasticity of Masonry, E_m

As detailed in section 3.4.2 of NZS 4230:2004, the modulus of elasticity of masonry is to be taken as E_m = 15 GPa. This value is only 60% of the value adopted previously.

Discussion with committee members responsible for development of NZS 4230P:1985 has indicated that the previously prescribed value of E_m = 25 GPa was adopted so that it would result in conservatively large stiffness, resulting in reduced periods and therefore larger and more conservative seismic loads. However, the value of E_m = 25 GPa is inconsistent with both measured behaviour and with a widely recommended relationship for concrete masonry of $E_m \approx 1000 f'_m$, representing a secant stiffness passing through the point (f'_m, $\varepsilon_m = 0.001$) on the stress strain curve.

Note also that application of this equation to 3.4.2 captures the notion that f'_m (12 MPa) is the lower 5% characteristic strength but that E_m (15 GPa) is the mean modulus of elasticity. This relationship is quantitatively demonstrated here in section 3.1.

It is argued that whilst period calculation may warrant a conservatively high value of E_m, serviceability design for deformations merits a correspondingly low value of E_m to be adopted. Consequently, the value of E_m = 15 GPa is specified as a mean value, rather than as an upper or a lower characteristic value.

2.3.3 Ultimate Compression Strain, ε_u

NZS 4230:1990 specified an ultimate compression strain for unconfined concrete masonry of ε_u = 0.0025. This value was adopted somewhat arbitrarily in order to be conservatively less than the comparable value of ε_u = 0.003 which is specified in NZS 3101 for the design of concrete structures.

In the period since development of NZS 4230:1990 it has become accepted internationally, based upon a wealth of physical test results, that there is no evidence to support a value other than that adopted for concrete. Consequently, when using NZS 4230:2004 the ultimate compression strain of unconfined concrete masonry shall be taken as ε_u = 0.003.

2.3.4 Strength Reduction Factors

Selection of strength reduction factors should be based on comprehensive studies on the measured structural performance of elements when correlated against their predicted strength, in order to determine the effect of materials and of construction quality.

The strategy adopted in NZS 4230:1990 was to consider the values used in NZS 3101, but to then add additional conservatism based on the perception that masonry material strength characteristics and construction practices were less consistent than their reinforced concrete equivalent.

In NZS 4230:2004 the strength reduction factors have been altered with respect to their predecessors because:

1. The manufacture of masonry constituent materials and the construction of masonry structures are governed by the same regulatory regimes as those of reinforced concrete.

2. There is no measured data to form a basis for the adoption of values of the strength reduction factors other than those employed in NZS 3101 for concrete structures, and the adoption of corresponding values will facilitate designers interchanging between NZS 4230 and NZS 3101.

3. The values adopted in NZS 4230:2004 are more conservative than those prepared by the Masonry Standards Joint Committee[2] (comprised of representatives from The Masonry Society, the American Concrete Institute, and the American Society of Civil Engineers), were Φ = 0.9 is specified for reinforced masonry in flexure and Φ = 0.8 is specified for reinforced masonry in shear.

[2] Masonry Standards Joint Committee (2011) "Building Code Requirements for Masonry Structures" and "Specification for Masonry Structures", TMS 402-11/ACI 530-11/ASCE 5-11, USA.

2.4 Design Philosophies

Table 3-2 of NZS 4230:2004 presents four permitted design philosophies, primarily based upon the permitted structural ductility factor, μ.

Whilst all design philosophies are equally valid, general discussion amongst designers of concrete masonry structures tends to suggest that nominally ductile and limited ductile response is most regularly favoured.

Taking due account for overall structural behaviour in order to avoid brittle failure mechanisms, nominally ductile design has the advantage over elastic design of producing reduced seismic design actions without requiring any special seismic detailing.

2.4.1 Limited Ductile Design

As outlined in section 3.7.3 of NZS 4230:2004, when conducting limited ductile design it is permitted to either adopt capacity design principles, or to use a simplified approach (3.7.3.3). In the simplified approach, where limits are placed on building height, the influence of material overstrength and dynamic magnification are accounted for by amplifying the seismic moments outside potential plastic hinge regions by an additional 50% (Eqn. 3-3) and by applying the seismic shear forces throughout the structure by an additional 100% (Eqn. 3-4). Consequently, the load combinations become:

$$\phi M_n \geq M^*_G + M^*_{Qu} + 1.5M^*_E \quad \text{and} \quad \phi V_n \geq V^*_G + V^*_{Qu} + 2V^*_E .$$

2.5 Component Design

An important modification to NZS 4230:2004 with respect to its predecessors is the use of a document format that collects the majority of criteria associated with specific components into separate sections.

This format is a departure from earlier versions which were formatted based upon design actions. The change was adopted because the new format was believed to be more helpful for users of the document.

The change also anticipated the release of NZS 3101:2006 to adopt a similar format, and is somewhat more consistent with equivalent Standards from other countries, particular AS 3700.

2.5.1 Definition of Column

Having determined that the design of walls, beams, and columns would be dealt with in separate sections, it was deemed important to clearly establish the distinction between a wall and a column.

In Section 2 of the standard it is stated that a column is an element having a length not greater than 790 mm and a width not less that 240 mm, subject primarily to compressive axial load. However, the intent of Section 7.3.1.5 was that a wall having a length less than 790 mm and having a compressive axial load less than $0.1f'_mA_g$ may be designed as either a wall or as a column depending on the intended function of the component within the design strategy, recognising that the design criteria for columns are more stringent than those for walls.

2.5.2 Moment Capacity of Walls

Moment capacity may be calculated from first principles using a linear distribution of strain across the section, the appropriate magnitude of ultimate compression strain, and the appropriate rectangular stress block. Alternatively, for **Rectangular**-section masonry components with **uniformly** distributed flexural reinforcement, Tables 2 to 5 over the page may be used.

These tables list in non-dimensional form the nominal capacity of unconfined and confined concrete masonry walls with either Grade 300 or Grade 500 flexural reinforcement, for different values of the two salient parameters, namely the axial load ratio N_n/f'_mL_wt or N_n/Kf'_mL_wt, and the strength-adjusted reinforcement ratio pf_y/f'_m or pf_y/Kf'_m.

Charts, produced from Tables 2 to 5, are also plotted which enable the user to quickly obtain a value for pf_y/f'_m or pf_y/Kf'_m given the axial load ratio N_n/f'_mL_wt or N_n/Kf'_mL_wt and the moment ratio $M_n/f'_mL_w^2t$ or $M_n/Kf'_mL_w^2t$. These charts are shown as Figures 1 to 4.

On the charts, each curve represents a different value for pf_y/f'_m or pf_y/Kf'_m. For points which fall between the curves, values can be established using linear interpolation.

2.6 Maximum Bar Diameters

Whilst not changed from the values given in NZS 4230:1990, it is emphasised here that there are limits to the permitted bar diameter that may be used for different component types, as specified in 7.3.4.5, 8.3.6.1 and 9.3.5.1.

Furthermore, as detailed in C7.3.4.5 there are limits to the size of bar that may be lapped, which makes a more restrictive requirement when using grade 500 MPa reinforcement.

Consequently, the resulting maximum bar sizes are presented below.

Table 1: **Maximum bar diameter for different block sizes**

Block size (mm)	Walls and beams		Columns	
	f_y = 300 MPa	f_y = 500 MPa	f_y = 300 MPa	f_y = 500 MPa
140	D16	DH12	5-D10	3-DH10
190	D20	DH16	3-D16	DH16
240	D25	DH20	2-D20	DH20
390	---	---	D32	DH32

2.7 Ductility Considerations

The Standard notes in section 7.4.6 that unless confirmed by a special study, adequate ductility may be assumed when the neutral axis depth of a component is less than an appropriate fraction of the section depth. Section 2.7.1 below lists the ratios c/L_w for masonry walls while justification for the relationship limiting the neutral axis depth is presented in sections 2.7.2 and 3.4.

An outline of the procedure for conducting a special study to determine the available ductility of cantilevered concrete masonry walls is presented in section 2.7.3.

2.7.1 Neutral Axis Depth

Neutral axis depth may be calculated from first principles, using a linear distribution of strain across the section, the appropriate level of ultimate compression strain and the appropriate rectangular stress block.

Alternatively, for **Rectangular** section structural walls, Tables 6 and 7 may be used.

These tables list in non-dimensional form the neutral axis depth of unconfined and confined walls with either Grade 300 or Grade 500 flexural reinforcement, for different values of axial load ratio N_n/f'_mL_wt or N_n/Kf'_mL_wt and reinforcement ratio pf_y/f'_m or pf_y/Kf'_m, where p is the ratio of uniformly distributed vertical reinforcement.

Charts, produced from Tables 6 and 7, are also plotted which enable the user to quickly obtain a value for c/L_w given the axial load ratio N_n/f'_mL_wt or N_n/Kf'_mL_wt and different value of pf_y/f'_m or pf_y/Kf'_m. These charts are shown as Figures 5 and 6.

Table 2: $\frac{M_n}{f'_m L_w^2 t}$ **for unconfined wall with f_y = 300 MPa**

$\frac{pf_y}{f'_m}$	Axial Load Ratio $\frac{N_n}{f'_m L_w t}$								
	0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
0.00	0.000	0.0235	0.0441	0.0618	0.0765	0.0882	0.0971	0.1029	0.1059
0.01	0.0049	0.0279	0.0480	0.0652	0.0795	0.0909	0.0995	0.1052	0.1079
0.02	0.0097	0.0322	0.0518	0.0686	0.0826	0.0937	0.1020	0.1075	0.1102
0.04	0.0190	0.0406	0.0593	0.0753	0.0886	0.0992	0.1070	0.1122	0.1146
0.06	0.0280	0.0487	0.0665	0.0818	0.0945	0.1045	0.1120	0.1168	0.1190
0.08	0.0367	0.0566	0.0735	0.0881	0.1002	0.1099	0.1169	0.1215	0.1235
0.10	0.0451	0.0641	0.0804	0.0944	0.1059	0.1152	0.1218	0.1261	0.1279
0.12	0.0534	0.0713	0.0871	0.1005	0.1116	0.1204	0.1267	0.1307	0.1324
0.14	0.0613	0.0783	0.0936	0.1064	0.1171	0.1255	0.1315	0.1353	0.1369
0.16	0.0690	0.0853	0.0999	0.1123	0.1225	0.1306	0.1363	0.1399	0.1414
0.18	0.0762	0.0922	0.1062	0.1181	0.1279	0.1357	0.1411	0.1445	0.1459
0.20	0.0832	0.0989	0.1124	0.1238	0.1332	0.1406	0.1459	0.1491	0.1503

Table 3: $\frac{M_n}{f'_m L_w^2 t}$ **for unconfined wall with f_y = 500 MPa**

$\frac{pf_y}{f'_m}$	Axial Load Ratio $\frac{N_n}{f'_m L_w t}$								
	0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
0.00	0.000	0.0235	0.0441	0.0618	0.0765	0.0882	0.0971	0.1029	0.1059
0.01	0.0049	0.0279	0.0480	0.0652	0.0794	0.0908	0.0993	0.1049	0.1076
0.02	0.0097	0.0322	0.0517	0.0685	0.0824	0.0934	0.1015	0.1068	0.1093
0.04	0.0190	0.0405	0.0591	0.0750	0.0881	0.0984	0.1059	0.1107	0.1128
0.06	0.0280	0.0484	0.0662	0.0813	0.0937	0.1033	0.1103	0.1147	0.1163
0.08	0.0365	0.0561	0.0731	0.0874	0.0992	0.1081	0.1147	0.1186	0.1199
0.10	0.0448	0.0635	0.0797	0.0934	0.1043	0.1129	0.1190	0.1225	0.1234
0.12	0.0528	0.0707	0.0862	0.0992	0.1096	0.1176	0.1233	0.1264	0.1271
0.14	0.0605	0.0777	0.0925	0.1047	0.1147	0.1223	0.1275	0.1303	0.1307
0.16	0.0680	0.0844	0.0986	0.1103	0.1198	0.1269	0.1318	0.1342	0.1344
0.18	0.0752	0.0910	0.1045	0.1157	0.1247	0.1315	0.1359	0.1381	0.1380
0.20	0.0823	0.0974	0.1104	0.1211	0.1297	0.1359	0.1400	0.1420	0.1417

Table 4: $\frac{M_n}{Kf'_m L_w^2 t}$ **for confined wall with f_y = 300 MPa**

$\frac{pf_y}{Kf'_m}$	Axial Load Ratio $\frac{N_n}{Kf'_m L_w t}$								
	0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
0.00	0.000	0.0236	0.0444	0.0625	0.0778	0.0903	0.1000	0.1069	0.1111
0.01	0.0049	0.0280	0.0484	0.0661	0.0810	0.0933	0.1027	0.1095	0.1136
0.02	0.0098	0.0324	0.0523	0.0696	0.0842	0.0962	0.1055	0.1121	0.1161
0.04	0.0191	0.0409	0.0599	0.0766	0.0905	0.1020	0.1108	0.1173	0.1211
0.06	0.0281	0.0491	0.0673	0.0833	0.0967	0.1078	0.1163	0.1224	0.1261
0.08	0.0369	0.0569	0.0746	0.0899	0.1029	0.1135	0.1217	0.1275	0.1311
0.10	0.0454	0.0645	0.0818	0.0964	0.1089	0.1191	0.1271	0.1326	0.1360
0.12	0.0537	0.0720	0.0888	0.1027	0.1149	0.1246	0.1323	0.1377	0.1410
0.14	0.0616	0.0794	0.0956	0.1090	0.1209	0.1302	0.1376	0.1428	0.1459
0.16	0.0692	0.0867	0.1021	0.1152	0.1267	0.1357	0.1428	0.1479	0.1509
0.18	0.0767	0.0939	0.1085	0.1214	0.1324	0.1412	0.1480	0.1530	0.1558
0.20	0.0841	0.1009	0.1149	0.1275	0.1381	0.1466	0.1532	0.1581	0.1608

Table 5: $\frac{M_n}{Kf'_m L_w^2 t}$ **for confined wall with f_y = 500 MPa**

$\frac{pf_y}{Kf'_m}$	Axial Load Ratio $\frac{N_n}{Kf'_m L_w t}$								
	0	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
0.00	0.000	0.0236	0.0444	0.0625	0.0778	0.0903	0.1000	0.1069	0.1111
0.01	0.0049	0.0280	0.0484	0.0661	0.0809	0.0932	0.1027	0.1094	0.1135
0.02	0.0098	0.0324	0.0523	0.0696	0.0841	0.0961	0.1054	0.1120	0.1159
0.04	0.0191	0.0408	0.0599	0.0765	0.0904	0.1019	0.1107	0.1171	0.1208
0.06	0.0281	0.0489	0.0673	0.0832	0.0967	0.1076	0.1161	0.1221	0.1257
0.08	0.0369	0.0569	0.0746	0.0898	0.1027	0.1133	0.1214	0.1272	0.1306
0.10	0.0454	0.0646	0.0817	0.0962	0.1088	0.1188	0.1267	0.1322	0.1355
0.12	0.0534	0.0720	0.0887	0.1026	0.1146	0.1243	0.1320	0.1372	0.1403
0.14	0.0614	0.0794	0.0956	0.1089	0.1205	0.1298	0.1372	0.1422	0.1452
0.16	0.0692	0.0866	0.1018	0.1151	0.1262	0.1352	0.1424	0.1472	0.1500
0.18	0.0769	0.0938	0.1083	0.1212	0.1319	0.1406	0.1475	0.1522	0.1549
0.20	0.0843	0.1006	0.1148	0.1273	0.1377	0.1460	0.1527	0.1573	0.1598

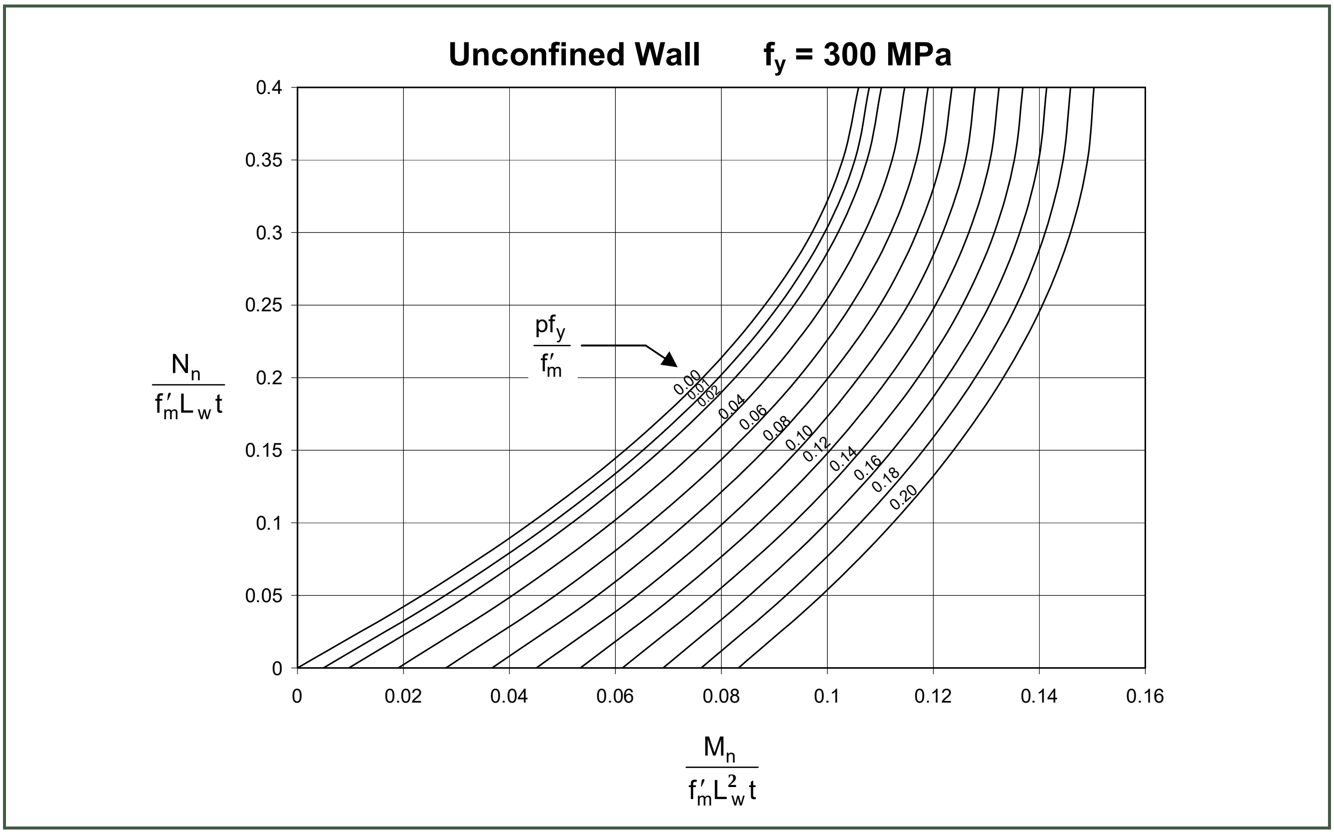


Figure 1: **Flexural Strength of Rectangular Masonry Walls with Uniformly Distributed Reinforcement, Unconfined Wall f_y = 300 MPa**

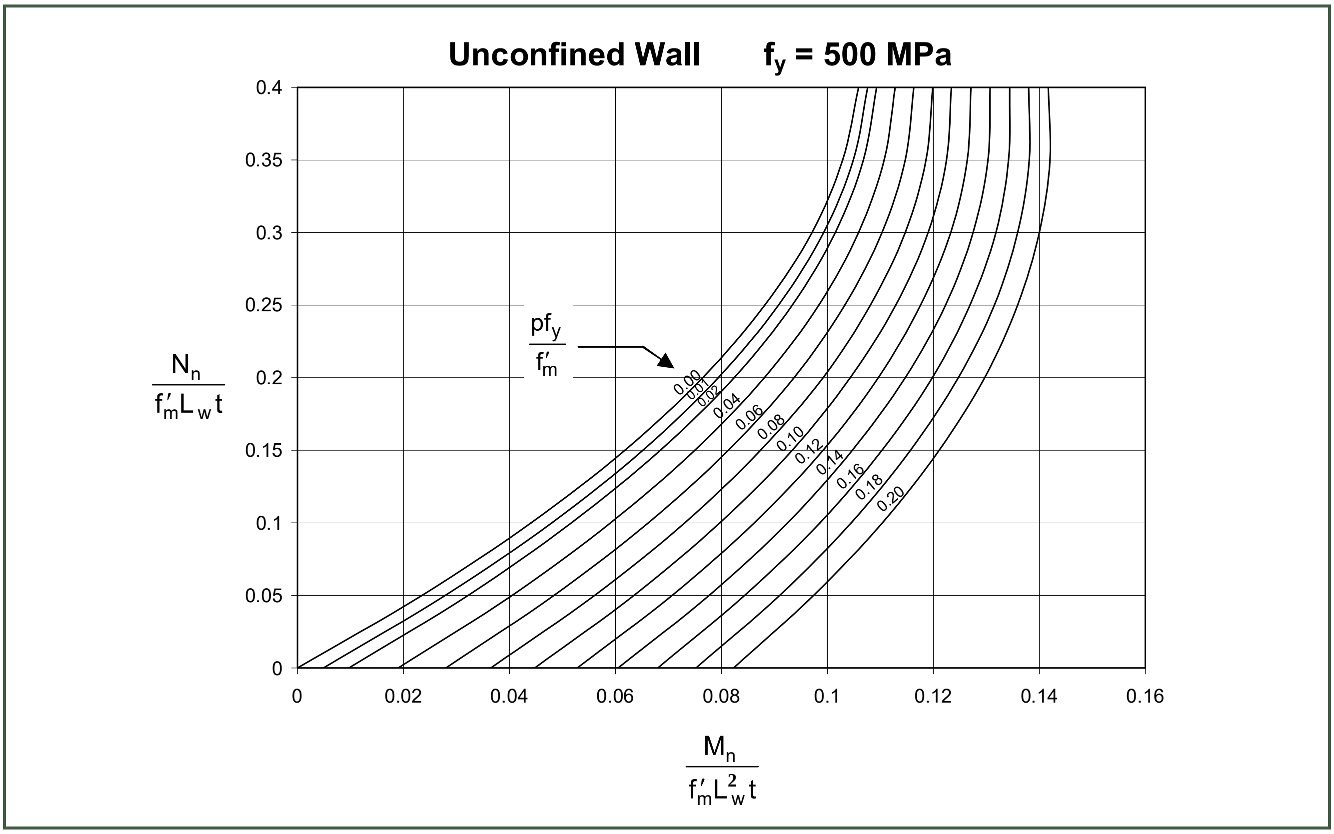


Figure 2: **Flexural Strength of Rectangular Masonry Walls with Uniformly Distributed Reinforcement, Unconfined Wall f_y = 500 MPa**

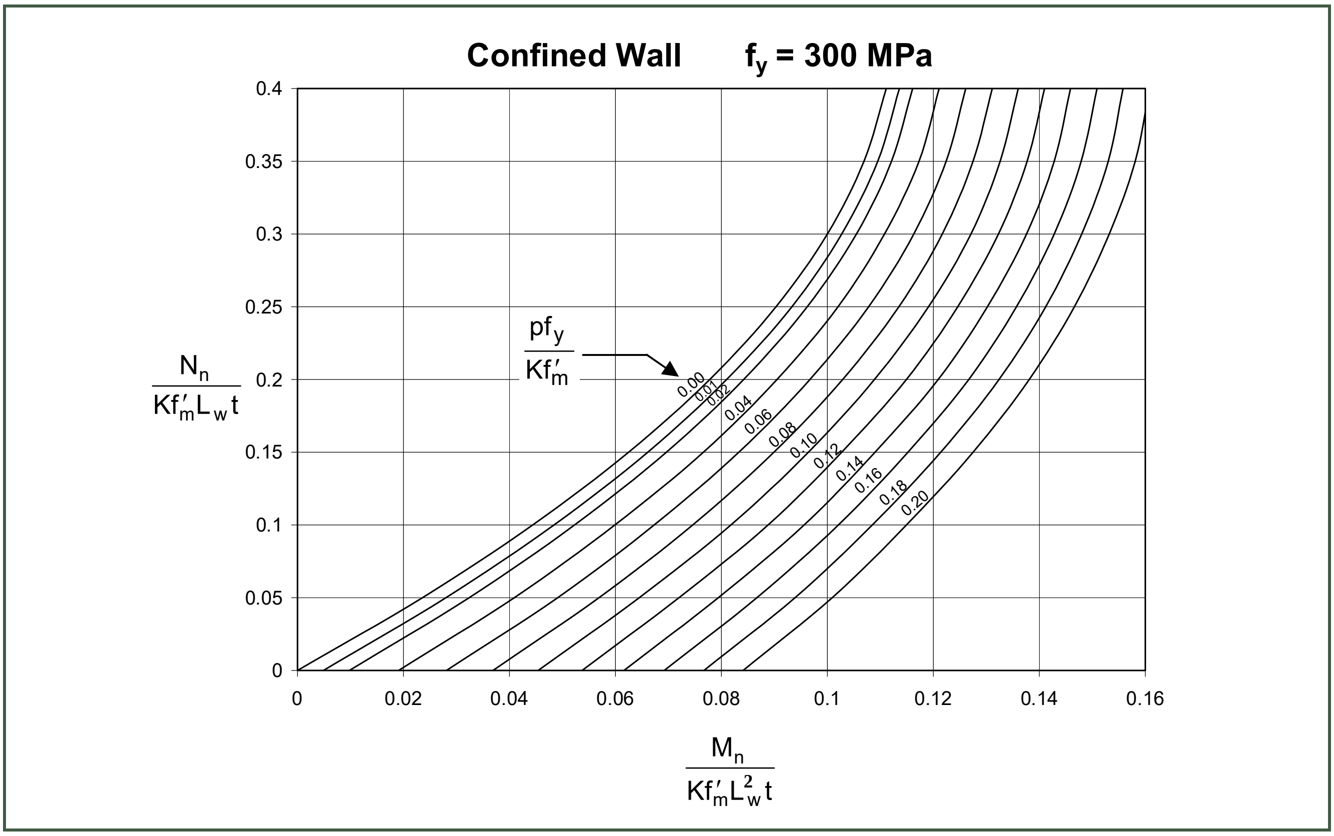


Figure 3: **Flexural Strength of Rectangular Masonry Walls with Uniformly Distributed Reinforcement, Confined Wall f_y = 300 MPa**

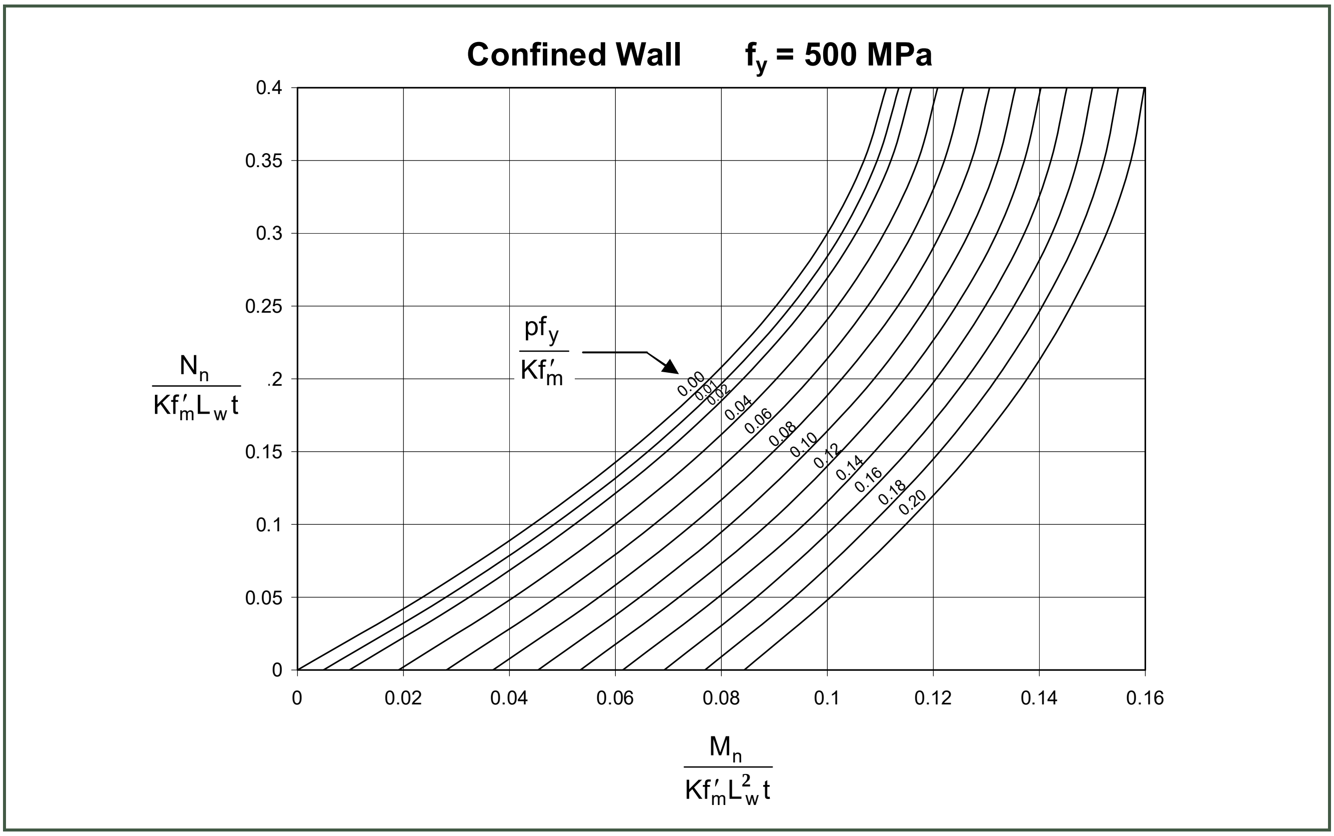


Figure 4: **Flexural Strength of Rectangular Masonry Walls with Uniformly Distributed Reinforcement, Confined Wall f_y = 500 MPa**

Table 6: **Neutral Axis Depth Ratio c/L_w (f_y = 300 MPa or 500 MPa): Unconfined Walls**

$\frac{pf_y}{f'_m}$	Axial Load Ratio $\frac{N_n}{f'_m L_w t}$								
	0	0.05	0.1	0.15	0.2	0.25	0.3	0.35	0.4
0	0.0000	0.0692	0.1384	0.2076	0.2768	0.3460	0.4152	0.4844	0.5536
0.01	0.0135	0.0808	0.1481	0.2155	0.2828	0.3502	0.4175	0.4848	0.5522
0.02	0.0262	0.0918	0.1574	0.2230	0.2885	0.3541	0.4197	0.4852	0.5508
0.04	0.0498	0.1121	0.1745	0.2368	0.2991	0.3614	0.4237	0.4860	0.5483
0.06	0.0712	0.1306	0.1899	0.2493	0.3086	0.3680	0.4273	0.4866	0.5460
0.08	0.0907	0.1473	0.2040	0.2606	0.3173	0.3739	0.4306	0.4873	0.5439
0.1	0.1084	0.1626	0.2168	0.2710	0.3252	0.3794	0.4336	0.4878	0.5420
0.12	0.1247	0.1766	0.2286	0.2805	0.3325	0.3844	0.4364	0.4883	0.5403
0.14	0.1397	0.1895	0.2394	0.2893	0.3392	0.3890	0.4389	0.4888	0.5387
0.16	0.1535	0.2014	0.2494	0.2974	0.3453	0.3933	0.4412	0.4892	0.5372
0.18	0.1663	0.2125	0.2587	0.3048	0.3510	0.3972	0.4434	0.4896	0.5358
0.2	0.1782	0.2227	0.2673	0.3118	0.3563	0.4009	0.4454	0.4900	0.5345

Table 7: **Neutral Axis Depth Ratio c/L_w (f_y = 300 MPa or 500 MPa): Confined Walls**

$\frac{pf_y}{Kf'_m}$	Axial Load Ratio $\frac{N_n}{Kf'_m L_w t}$								
	0	0.05	0.1	0.15	0.2	0.25	0.3	0.35	0.4
0	0.0000	0.0579	0.1157	0.1736	0.2315	0.2894	0.3472	0.4051	0.4630
0.01	0.0113	0.0679	0.1244	0.1810	0.2376	0.2941	0.3507	0.4072	0.4638
0.02	0.0221	0.0774	0.1327	0.1881	0.2434	0.2987	0.3540	0.4093	0.4646
0.04	0.0424	0.0953	0.1483	0.2013	0.2542	0.3072	0.3602	0.4131	0.4661
0.06	0.0610	0.1118	0.1626	0.2134	0.2642	0.3150	0.3659	0.4167	0.4675
0.08	0.0781	0.1270	0.1758	0.2246	0.2734	0.3223	0.3711	0.4199	0.4688
0.1	0.0940	0.1410	0.1880	0.2350	0.2820	0.3289	0.3759	0.4229	0.4699
0.12	0.1087	0.1540	0.1993	0.2446	0.2899	0.3351	0.3804	0.4257	0.4710
0.14	0.1224	0.1661	0.2098	0.2535	0.2972	0.3409	0.3846	0.4283	0.4720
0.16	0.1351	0.1774	0.2196	0.2618	0.3041	0.3463	0.3885	0.4307	0.4730
0.18	0.1471	0.1879	0.2288	0.2696	0.3105	0.3513	0.3922	0.4330	0.4739
0.2	0.1582	0.1978	0.2373	0.2769	0.3165	0.3560	0.3956	0.4351	0.4747

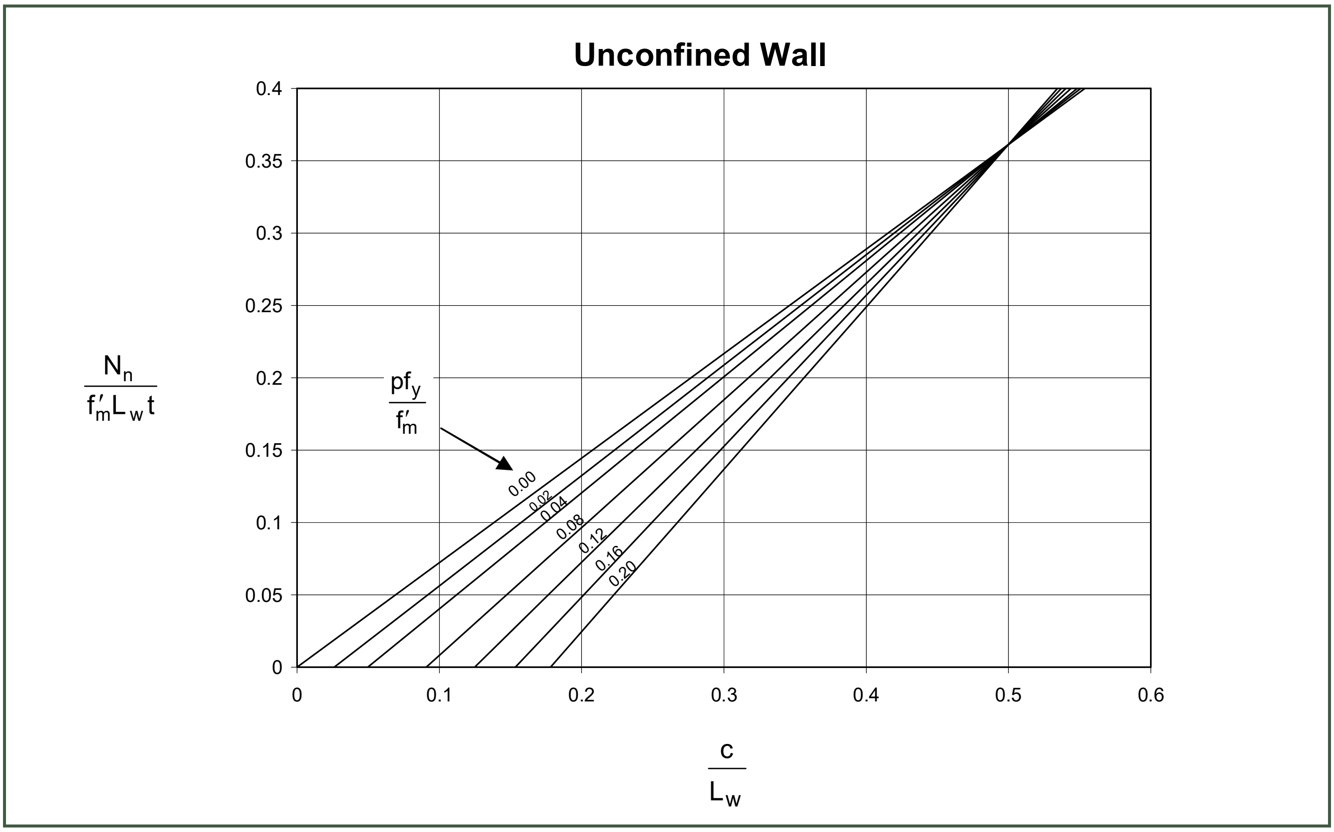


Figure 5: **Neutral Axis Depth of Unconfined Rectangular Masonry Walls with Uniformly Distributed Reinforcement, f_y = 300 MPa or 500 MPa**

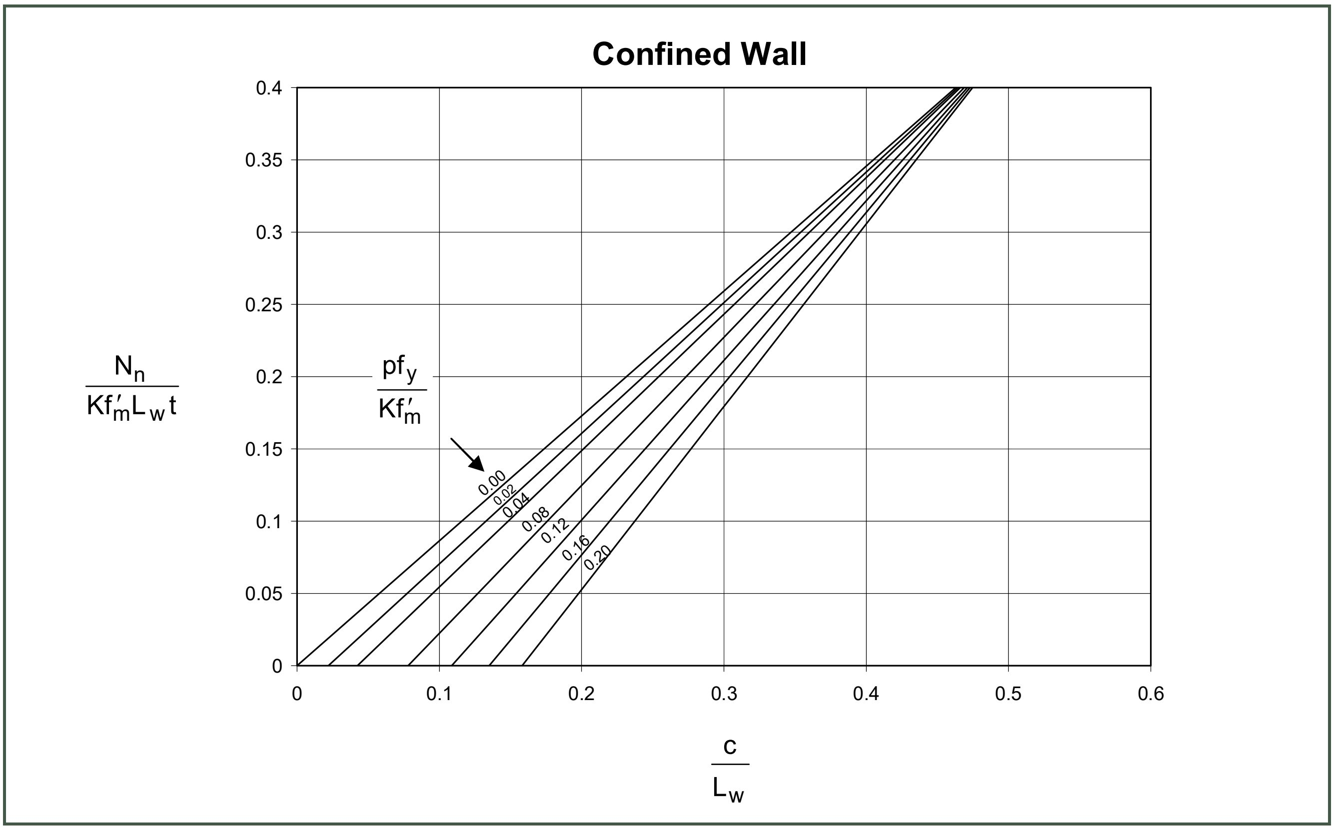


Figure 6: **Neutral Axis Depth of Confined Rectangular Masonry Walls with Uniformly Distributed Reinforcement, f_y = 300 MPa or 500 MPa**

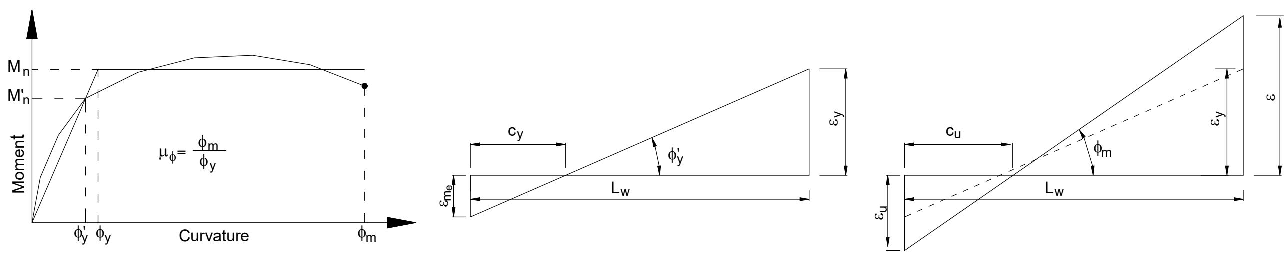
2.7.2 Curvature Ductility

To avoid failure of potential plastic hinge regions of unconfined masonry shear walls, the masonry standard limits the extreme fibre compression strain at the full design inelastic response displacement to the unconfined ultimate compression strain of $\varepsilon_u = 0.003$. The available ductility at this ultimate compression strain decreases with increasing depth of the compression zone, expressed as a fraction of the wall length. Section 7.4.6 of NZS 4230:2004 ensures that the available ductility will exceed the structural ductility factor, μ, for walls of aspect ratio less than 3. This section provides justification for the relationship limiting neutral axis depth.

The most common and desirable sources of inelastic structural deformations are rotations in potential plastic hinges. Therefore, it is useful to relate section rotations per unit length (i.e. curvature) to corresponding bending moments. As shown in Figure 7(a), the maximum curvature ductility is expressed as:

$$\mu_\phi = \frac{\phi_m}{\phi_y} \qquad [1]$$

where ϕ_m is the maximum curvature expected to be attained or relied on and ϕ_y is the yield curvature.



(a) Moment Curvature Relationship (b) First-yield Curvature (c) Ultimate Curvature

Figure 7: **Definition of curvature ductility**

Yield Curvature

For distributed flexural reinforcement, as would generally be the case for a masonry wall, the curvature associated with tension yielding of the most extreme reinforcing bar, ϕ'_y, will not reflect the effective yielding curvature of all tension reinforcement, identified as ϕ_y. Similarly, ϕ'_y may also result from nonlinear compression response at the extreme compression fibre.

$$\phi'_y = \frac{\varepsilon_y}{L_w - c_y} \quad \text{or} \quad \phi'_y = \frac{\varepsilon_y + \varepsilon_{me}}{L_w} \qquad [2]$$

where $\varepsilon_y = f_y / E_s$ and c_y is the corresponding neutral-axis depth. Extrapolating linearly to the nominal moment M_n, as shown in Figure 7(a), the yield curvature ϕ_y is given as:

$$\phi_y = \frac{M_n}{M'_n} \phi_y \qquad [3]$$

Maximum Curvature

The maximum attainable curvature of a section is normally controlled by the maximum compression strain ε_u at the extreme fibre. With reference to Figure 7(c), this curvature can be expressed as:

$$\phi_m = \frac{\varepsilon_u}{c_u} \qquad [4]$$

Displacement and Curvature Ductility

The displacement ductility for a cantilever concrete masonry wall can be expressed as:

$$\mu_\Delta = \frac{\Delta}{\Delta_y} \quad \text{or} \quad \mu_\Delta = \frac{\Delta_y + \Delta_p}{\Delta_y} \qquad [5]$$

consequently;

$$\mu_\Delta = 1 + \frac{\Delta_p}{\Delta_y}$$

Yield Displacement

The yield displacement for a cantilever wall of height h_w may be estimated as:

$$\Delta_y = \phi_y h_w^2 / 3 \qquad [6]$$

Plastic Displacement

The plastic rotation occurring in the equivalent plastic hinge length L_p is given by:

$$\theta_p = \phi_p L_p = (\phi_m - \phi_y) L_p \qquad [7]$$

Assuming the plastic rotation to be concentrated at mid-height of the plastic hinge, the plastic displacement at the top of the cantilever wall is:

$$\Delta_p = \theta_p (h_w - 0.5L_p) = (\phi_m - \phi_y) L_p (h_w - 0.5L_p) \qquad [8]$$

Substituting Eqns. 6 and 8 into Eqn. 5 gives:

$$\mu_\Delta = 1 + \frac{(\phi_m - \phi_y) L_p (h_w - 0.5L_p)}{\phi_y h_w^2 / 3}$$

$$= 1 + 3(\mu_\phi - 1) \frac{L_p}{h_w} \left(1 - L \frac{L_p}{2h_w}\right) \qquad [9]$$

Rearranging Eqn. 9:

$$\mu_\phi = 1 + \frac{\mu_\Delta - 1}{3(L_p/h_w)(1 - L_p/2h_w)} \qquad [10]$$

Paulay and Priestley (1992) indicated that typical values of the plastic hinge length is $0.3 < L_p/L_w < 0.8$. For simplicity, the plastic hinge length L_p may be taken as half the wall length L_w, and Eqn. 10 may be simplified to:

$$\mu_\phi = 1 + \frac{\mu_\Delta - 1}{\frac{3}{2}(L_w/h_w)(1 - L_w/4h_w)} \quad \text{or} \quad \mu_\phi = 1 + \frac{\mu_\Delta - 1}{\frac{3}{2A_r}\left(1 - \frac{1}{4A_r}\right)} \qquad [11]$$

where A_r is the wall aspect ratio h_w/L_w.

Reduced Ductility

The flexural overstrength factor $\phi_{o,w}$ is used to measure the extent of any over- or undersign:

$$\phi_{o,w} = \frac{\text{flexural overstrength}}{\text{moment resulting from loading Standard forces}} = \frac{M_{o,w}}{M_E^*} \qquad [12]$$

Whenever $\phi_{o,w}$ exceeds λ_o/ϕ, the wall possesses reserve strength as higher resistance will be offered by the structure than anticipated when design forces were established. The overstrength factors λ_o are taken as 1.25 and 1.40 for grade 300 and 500 reinforcement respectively, while the strength reduction factor ϕ shall be taken as 0.85. It is expected that a corresponding reduction in ductility demand in the design earthquake will result. Consequently, design criteria primarily affected by ductility capacity may be met for the reduced ductility demand ($\mu_{\Delta r}$) rather than the anticipated ductility (μ_Δ). Therefore:

$$\mu_{\Delta r} = \frac{\lambda_o/\phi}{\phi_{o,w}}\mu_\Delta \qquad [13]$$

2.7.3 Ductility Capacity of Cantilevered Concrete Masonry Walls

Section 7.4.6.1 of NZS 4230:2004 provides a simplified but conservative method to ensure that adequate ductility can be developed in masonry walls. The Standard allows the rational analysis developed by Priestley[3, 4] as an alternative to determine the available ductility of cantilevered concrete masonry walls.

Figure 8 includes dimensionless design charts for the ductility capacity, μ_3 of unconfined concrete masonry walls whose aspect ratio is $A_r = h_w/L_w = 3$. For walls of other aspect ratio, A_r, the ductility capacity can be found from the μ_3 value using Eqn. 14:

$$\mu_{A_r} = 1 + \frac{3.3\left(\mu_3 - 1\right)\left(1 - \frac{0.25}{A_r}\right)}{A_r} \qquad [14]$$

When the ductility capacity found from Figure 8 and Eqn. 14 is less than that required, redesign is necessary to increase ductility. The most convenient and effective way to increase ductility is to use a higher design value of f'_m for Type A masonry. This change will reduce the axial load ratio N_n/f'_mA_g (where $N_n = N^*/\phi$) and the adjusted reinforcement ratio $p^* = p12/f'_m$ proportionally. From Figure 8, the ductility will therefore increase.

Where the required increase in f'_m cannot be provided, a second alternative is to confine the masonry within critical regions of the wall. The substantial increase in ductility capacity resulting from confinement is presented in Figure 9. A third practical solution is to increase the thickness of the wall.

In Figures 8 and 9, the reinforcement ratio is expressed in the dimensionless form p*, where:

for unconfined walls: $$p^* = \frac{12p}{f'_m}$$

for confined walls: $$p^* = \frac{14.42p}{Kf'_m}$$

and $$K = 1 + p_s\frac{f_{yh}}{f'_m}$$

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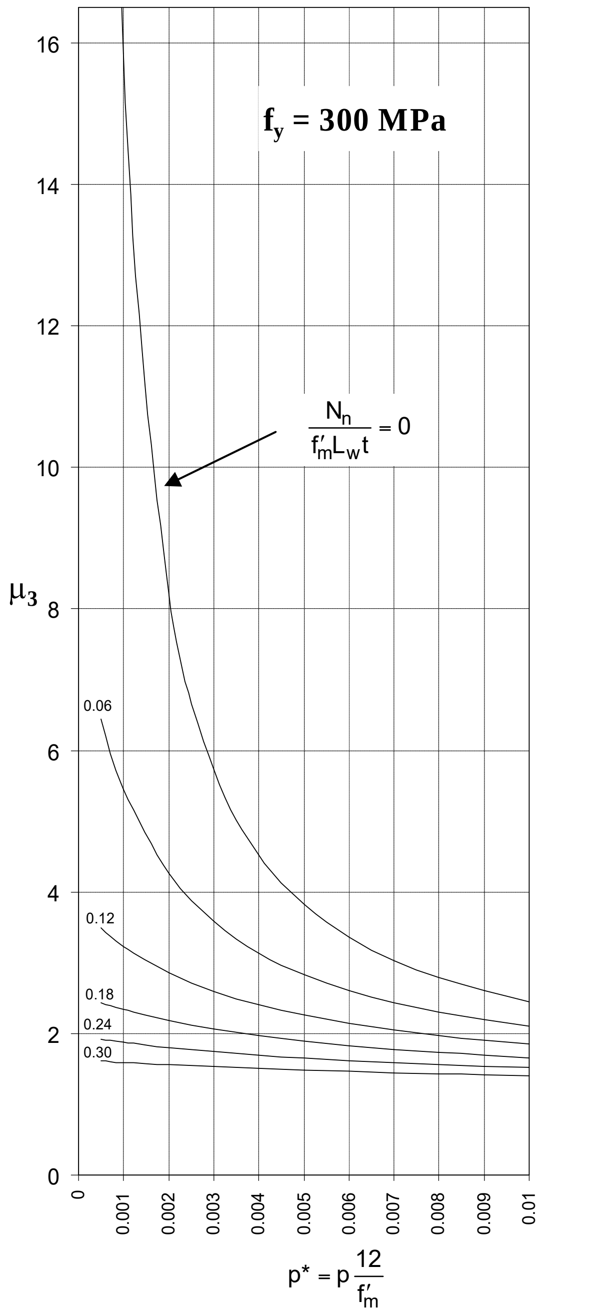
[3] Priestley, M. J. N. (1981). "Ductility of Unconfined Masonry Shear Walls", Bulletin NZNSEE, Vol. 14, No. 1, pp. 3-11.

[4] Priestley, M. J. N. (1982). "Ductility of Confined Masonry Shear Walls", Bulletin NZNSEE, Vol. 5, No. 1, pp. 22-26.



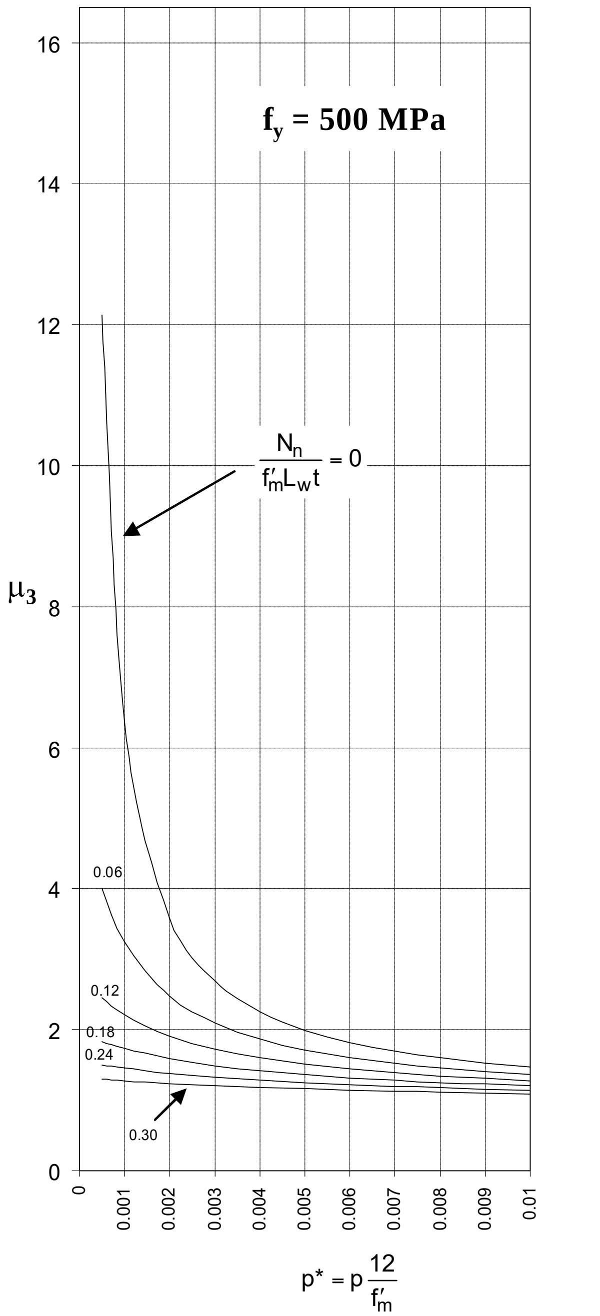


Figure 8: **Ductility of Unconfined Concrete Masonry Walls for Aspect Ratio A_r = 3**

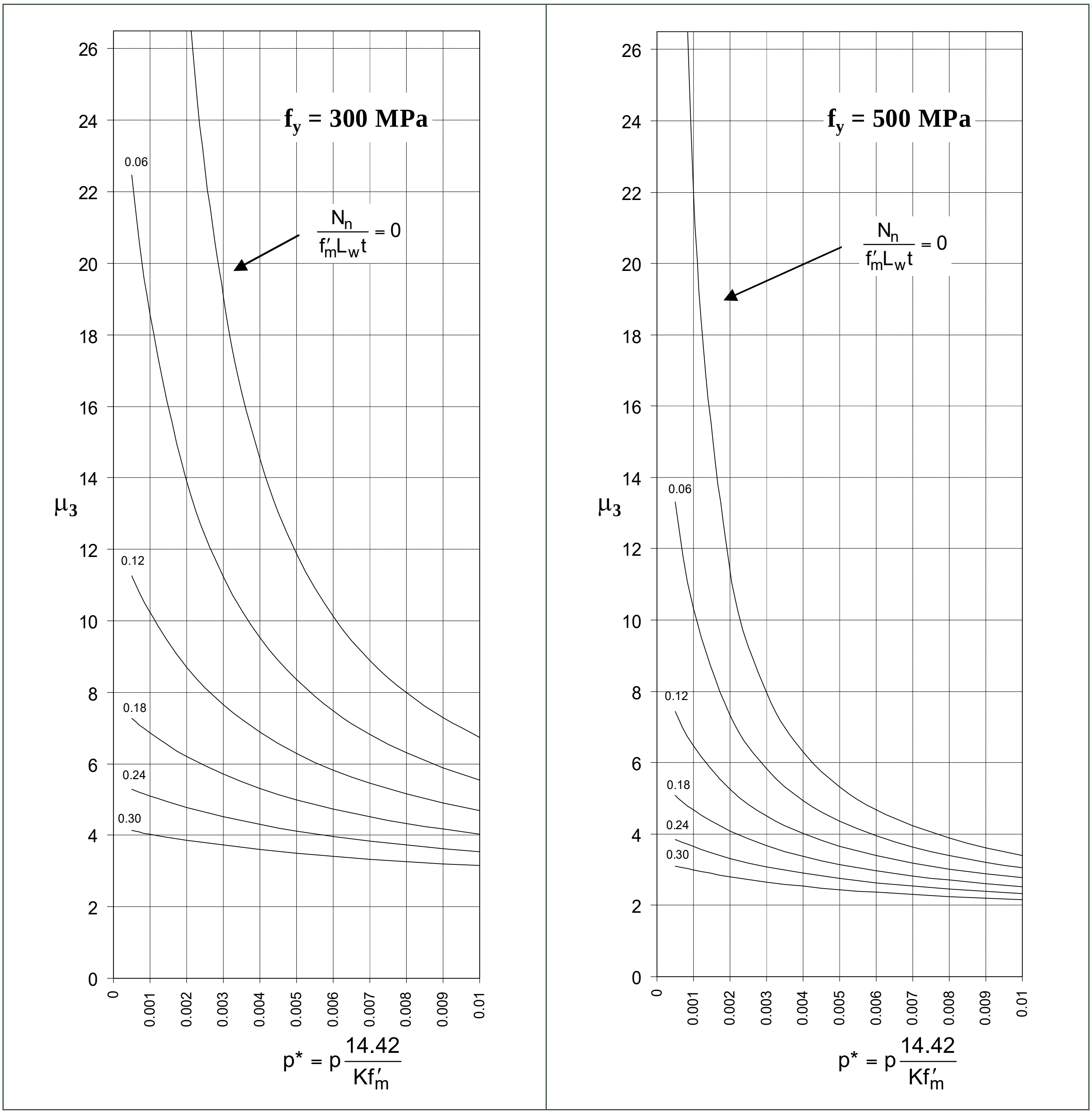


Figure 9: **Ductility of Confined Concrete Masonry Walls for Aspect Ratio A_r = 3**

2.7.4 Walls With Openings

Section 7.4.8.1 requires that for ductile cantilever walls with irregular openings, appropriate analyses such as based on strut-and-tie models shall be used to establish rational paths for the internal forces. Significant guidance on the procedure for conducting such an analysis is contained within NZS 3101, and an example is presented here in section 3.8.

2.8 Masonry In-plane Shear Strength

At the time NZS 4230:1990 was released, it was recognised that the shear strength provisions it contained were excessively conservative. However, the absence at that time of experimental data related to the shear strength of masonry walls when subjected to seismic forces prevented the preparation of more accurate criteria.

The shear resistance of reinforced concrete masonry components is the result of complex mechanisms, such as tension of shear reinforcement, dowel action of longitudinal reinforcement, as well as aggregate interlocking between the parts of the masonry components separated by diagonal cracks and the transmission of forces by diagonal struts forming parallel to shear cracks.

More recent experimental studies conducted in New Zealand and abroad have successfully shown the shear strength of reinforced masonry walls to be significantly in excess of that allowed by NZS 4230:1990. Consequently, new shear strength provisions are provided in section 10.3.2 of NZS 4230:2004. As outlined in clause 10.3.2.2 (Eqn. 10-5), masonry shear strength shall be evaluated as the sum of contributions from individual components, namely masonry (v_m), shear reinforcement (v_s) and applied axial compression load (v_p).

Masonry Component v_m

It has been successfully demonstrated through experimental studies that masonry shear strength, v_m increases with f'_m. However, the increase is not linear in all ranges of f'_m, but the rate becomes gradually lower as f'_m increases. Consequently, it is acceptable that v_m increases approximately in proportion to $\sqrt{f'_m}$. Eqn. 10-6 of NZS 4230:2004 is a shear expression developed by Voon and Ingham[5] for concrete masonry walls, taking into account the beneficial influence of the dowel action of tension longitudinal reinforcement and the detrimental influence of wall aspect ratio. These conditions are represented by the C_1 and C_2 terms included in Eqn. 10-6 of NZS 4230:2004. The v_{bm} specified in table 10.1 was established for a concrete masonry wall that has the worst case aspect ratio of $h_e/L_w \geq 1.0$ and reinforced longitudinally using grade 300 reinforcing steel with the minimum specified p_w of 0.07% (7.3.4.3).

For masonry walls that have aspect ratios of $0.25 \leq h_e/L_w \leq 1.0$ and/or p_w greater then 0.07%, the v_{bm} may be amplified by the C_1 and C_2 terms to give v_m. In order to guard against premature shear failure within the potential plastic hinge region of a component, the masonry standard assumes that little strength degradation occurs up to a component ductility ratio of 1.25, followed by a gradual decrease to higher ductility. This behaviour is represented by table 10.1 of NZS 4230:2004.

Axial Load Component v_p

Unlike NZS 4230:1990, the shear strength provided by axial load is evaluated independently of v_m in NZS 4230:2004. Section 10.3.2.7 of NZS 4230:2004 outlined the formulation, which considers the axial compression force to enhance the shear strength by arch action forming an inclined strut. Limitations of $v_p \leq 0.1f'_m$ and $N^* \leq 0.1f'_mA_g$ are included to prevent excessive dependence on v_p in a relatively squat masonry component and to avoid the possibility of brittle shear failure of a masonry component. In addition, the use of N* when calculating v_p is to ensure a more conservative design than would arise using N_n.

Shear Reinforcement Component v_s

The shear strength contributed by the shear reinforcement is evaluated using the method incorporated in NZS 3101, but is modified for the design of masonry walls to add conservatism based on the perception that bar anchorage effects result in reduced efficiency of shear reinforcement in masonry walls, when compared with the use of enclosed stirrups in beams and columns.

As the shear strength provisions of NZS 4230:2004 originated from experimental data of masonry walls and because the new shear strength provisions generated significantly reduce shear reinforcement requirements, sections 8.3.11 and 9.3.6, and Eqn. 10-9 of NZS 4230:2004, must be considered to establish the quantity and detailing of minimum shear reinforcement required in beams and columns.

2.9 Design of Slender Wall

Slender concrete masonry walls are often designed as free standing vertical cantilevers, in applications such as boundary walls and fire walls, and also as simply supported elements with low stress demands such as exterior walls of single storey factory buildings. In such circumstances these walls are typically subjected to low levels

[5] Voon, K. C., and Ingham, J. M. (2007). "Design Expression for the In-plane Shear Strength of Reinforced Concrete Masonry", ASCE Journal of Structural Engineering, Vol. 133, No. 5, May, pp. 706-713.

of axial and shear stress, and NZS 4230:1990 permitted relaxation of the criteria associated with maximum wall slenderness in such situations.

At the time of release of NZS 4230:2004 there was considerable debate within the New Zealand structural design fraternity regarding both an appropriate rational procedure for determining suitable wall slenderness criteria, and appropriate prescribed limits for maximum wall slenderness (alternatively expressed as a minimum wall thickness for a prescribed wall height). This debate was directed primarily at the design of slender precast reinforced concrete walls, but it was deemed appropriate that any adopted criteria for reinforced concrete walls be applied in a suitably adjusted manner to reinforced concrete masonry walls.

Recognising that at the time of release of NZS 4230:2004 there was considerable "engineering judgement" associated with the design of slender walls, the position taken by the committee tasked with authoring NZS 4230:2004 was to permit a minimum wall thickness of $0.05L_n$, where L_n is the smaller of the clear vertical height between effective line of horizontal support or the clear horizontal length between line of vertical support.

For free standing walls, an effective height of twice that of the actual cantilever height should be adopted. This $0.05L_n$ minimum wall thickness criteria, without permitting relaxation to $0.03L_n$ in special low-stress situations, is more stringent than the criteria provided previously in NZS 4230:1990, more stringent than that permitted in the US document TMS 402-11/ACI 530-11/ASCE 5-11, and more stringent than the criteria in NZS 3101:2006. Consequently, designers may elect to use "engineering judgement" to design outside the scope of NZS 4230:2004, at their discretion. The appropriate criteria from these other documents is reported in Table 8 below.

Table 8: **Wall slenderness limits in other design standards**

Standard	Limits
NZS 4230:1990	Minimum wall thickness of $0.03L_n$ if: (a) Part of single storey structure, and (b) Elastic design for all load combinations, and (c) Shear stress less than $0.5v_n$
TMS 402-11/ACI 530-11/ASCE 5-11	Minimum wall thickness of $0.0333L_n$ if: (a) Factored axial compression stress less than $0.05f'_m$ (see section 3.3.5.3)
NZS 3101:2006	Minimum wall thickness of $0.0333L_n$ if: (a) $N^* > 0.2 f'_c A_g$ (section 11.3.7) Otherwise, more slender walls permitted (see NZS 3101:Part 1:2006, section 11.3 for further details)

3.0 Design Examples

3.1 Determine f'_m From Strengths of Grout and Masonry Units

Calculate the characteristic masonry compressive strength, f'_m, given that the mean strengths of concrete masonry unit and grout are 17.5 MPa and 22.0 MPa, with standard deviations of 3.05 MPa and 2.75 MPa respectively.

For typical concrete masonry, the ratio of the net concrete block area to the gross area of masonry unit is to be taken as 0.45, i.e. $\alpha = 0.45$.

SOLUTION

The characteristic masonry compressive strength (5 percentile value) f'_m can be calculated from the strengths of the grout and the masonry unit using the equations presented in Appendix B of NZS 4230:2004.

Finding the mean masonry compressive strength, f_m

From Eqn. B-1 of NZS 4230:2004:

$$f_m = 0.59\,\alpha f_{cb} + 0.90(1-\alpha) f_g$$

$$= 0.59 \times 0.45 \times 17.5 + 0.90 \times (1-0.45) \times 22.0$$

$$= 15.54 \text{ MPa}$$

Finding the standard deviation of masonry strength, x_m

From Eqn. B-2 of NZS 4230:2004:

$$x_m = \sqrt{0.35\alpha^2 x_{cb}^2 + 0.81(1-\alpha)^2\, x_g^2}$$

$$= \sqrt{0.35 \times 0.45^2 \times 3.05^2 + 0.81 \times (1-0.45)^2 \times 2.75^2}$$

$$= 1.59 \text{ MPa}$$

Finding the characteristic masonry compressive strength, f'_m

From Eqn. B-3 of NZS 4230: 2004:

$$f'_m = f_m - 1.65 x_m$$

$$= 15.54 - 1.65 \times 1.59$$

$$= 12.9 \text{ MPa}$$

Note that the values for mean and standard deviation of strength used here for masonry units and for grout correspond to the lowest characteristic values permitted by NZS 4210, with a resultant f'_m in excess of that specified in table 3.1 of NZS 4230:2004 for observation types B and A. Note also that these calculations have established a mean strength of approximately 15 MPa, supporting the use of E_m = 15 GPa as discussed here in section 2.3.2.

3.2 In-plane Flexure

3.2.1 3.2(a) Establishing Flexural Strength of Masonry Beam

Calculate the nominal flexural strength of the concrete masonry beam shown in Figure 10. Assume the beam is unconfined, f'_m = 12 MPa and f_y = 300 MPa.

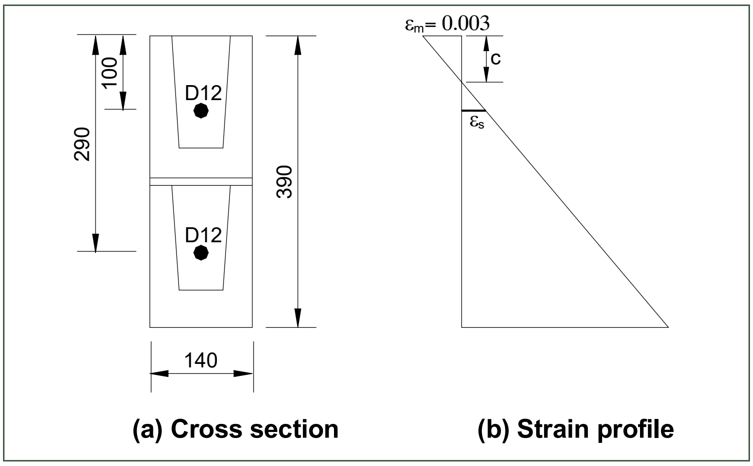


Figure 10: **Concrete Masonry Beam**

SOLUTION

Assume that both D12 bars yield in tension. Therefore tension force due to reinforcement is:

$$A_s = \pi \times 12^2/4 \qquad = 113.1 \text{ mm}^2$$

$$\Rightarrow \qquad \Sigma T_i = \Sigma A_{si} f_y = 2 \times 113.1 \times 300 = 67.85 \text{ kN}$$

Now consider Force Equilibrium:

$$C_m = \Sigma T_i$$

where $C_m = 0.85 f'_m ab$

$$\Rightarrow \qquad 0.85 f'_m ab = 67.85 \text{ kN}$$

$$a = \frac{67.85 \times 10^3}{0.85 f'_m \times 140} = 47.5 \text{ mm}$$

$$c = \frac{47.5}{0.85} = 55.9 \text{ mm}$$

Check to see if the upper reinforcing bar indeed yields:

$$\frac{\varepsilon_s}{100 - c} = \frac{\varepsilon_m}{c}$$

$$\Rightarrow \qquad \varepsilon_s = \frac{0.003}{55.9} \times 44.1 = 0.00237 > 0.0015 \qquad \text{therefore bar yielded}$$

Now taking moment about the neutral axis:

$$M_n = C_m \times (c - a/2) + T_i \times (d_i - c)$$

$$M_n = 67.85 \times (55.9 - 47.5/2) + 33.9 \times (100 - 55.9) + 33.9 \times (290 - 55.9)$$

$$= 11.6 \text{ kNm}$$

Alternatively, use Table 2 to establish flexural strength of the masonry beam:

$$p = \frac{A_s}{A_n} = \frac{226.2}{140 \times 390} = 0.0041$$

$$p\frac{f_y}{f'_m} = 0.0041 \times \frac{300}{12}$$

$$= 0.103$$

and $\dfrac{N_n}{f'_m A_n} = 0$

$$\Rightarrow \qquad \text{From Table 2,} \quad \frac{M_n}{f'_m h_b^2 t} \approx 0.0451$$

$$\Rightarrow \quad M_n = 0.0451 \times 12 \times 390^2 \times \frac{140}{1 \times 10^6}$$

$$M_n = 11.5 \text{ kNm}$$

3.2.2 3.2(b) Establishing Flexural Strength of Masonry Wall

Calculate the nominal flexural strength of the 140 mm wide concrete masonry wall shown in Figure 11. Assume the wall is unconfined, f'_m = 12 MPa, f_y = 300 MPa and N* = 115 kN.

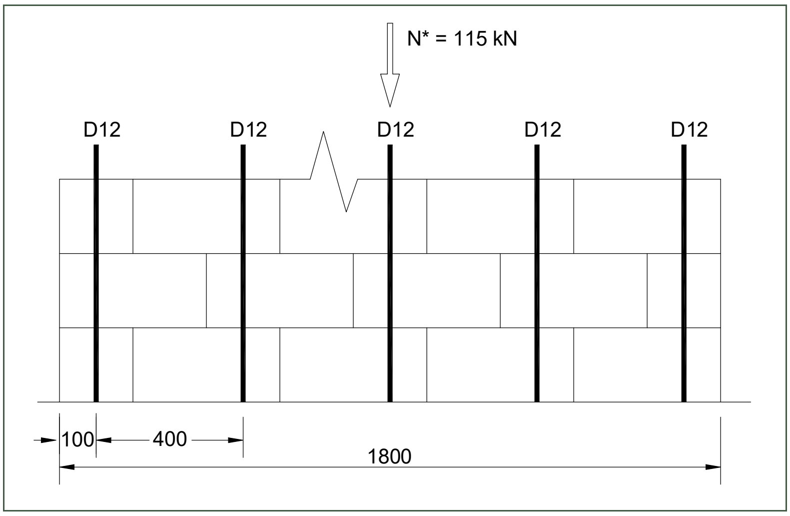
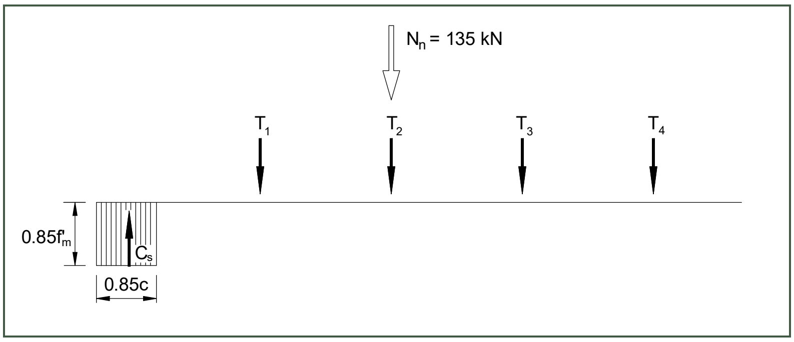


Figure 11: **Concrete Masonry Wall**

SOLUTION

Arial Load at Base

$$N_n = \frac{N^*}{\phi} = \frac{115}{0.85} = 135 \text{ kN}$$



Assume 4-D12 yield in tension and 1-D12 yields in compression:

$$\text{Area of 1-D12} = \pi \times \frac{12^2}{4} = 113.1 \text{ mm}^2$$

Therefore total tension force from longitudinal reinforcement:

$\Rightarrow$ T = 4 x 113.1 x 300 = 135.1 kN

and C_s = 113.1 x 300 = 33.9 kN

Now consider Force Equilibrium:

$C_m + C_s = T + N_n$

$C_m = T + N_n - C_s$ where $C_m = 0.85f'_m ab$

$\Rightarrow \quad 0.85f'_m ab = 135.1 + 135 - 33.9$

$\Rightarrow \quad 0.85f'_m ab = 236.8$ kN

$$a = \frac{236.8 \times 10^3}{0.85\, f'_m \times 140} = 165.8 \text{ mm}$$

$$c = \frac{165.8}{0.85} = 195.1 \text{mm}$$

The reinforcing bar in compression is located closest to the neutral axis. Check to see that this bar does indeed yield:

$$\frac{\varepsilon_s}{c - 100} = \frac{\varepsilon_m}{c}$$

$$\Rightarrow \quad \varepsilon_s = \frac{0.003}{195.1} \times 95.1 = 0.00146 \approx 0.0015 \quad \text{therefore OK}$$

Now taking moment about the neutral axis:

$$M_n = C_m \times \left(c - \frac{a}{2}\right) + T_i \times \left(d_i - c\right) + N_n \times \left(\frac{L_w}{2} - c\right)$$

$$M_n = 236.8 \text{ x } \left(195.1 - \frac{165.8}{2}\right) + 33.9 \text{ x } (195.1 - 100) + 33.9 \text{ x } (500 - 195.1) + 33.9 \text{ x } (900 - 195.1)$$

$$+ 33.9 \text{ x } (1300 - 195.1) + 33.9 \text{ x } (1700 - 195.1) + 135 \text{ x } \left(\frac{1800}{2} - 195.1\right)$$

$$= 247.7 \text{ kNm}$$

Alternatively, use Table 2 to establish flexural strength of the masonry wall:

$$p = \frac{A_s}{L_w t} = \frac{5 \times 113.1}{140 \text{ x } 1800} = 0.00224$$

$$p\frac{f_y}{f'_m} = 0.00224 \times \frac{300}{12} = 0.056$$

and $$\frac{N_n}{f'_m L_w t} = \frac{135 \times 10^3}{12 \times 1800 \times 140} = 0.045$$

$\Rightarrow$ From Table 2, $\dfrac{M_n}{f'_m L_w^2 t} \approx 0.04499$

$$M_n = 0.04499 \times 12 \times 1800^2 \times \frac{140}{1 \times 10^6}$$

$$= 245 \text{ kNm}$$

3.3 Out-of-Plane Flexure

A 190 mm thick fully grouted concrete masonry wall is subjected to N* = 21.3 kN/m and is required to resist an out-of-plane moment of M* = 17 kNm/m. Design the flexural reinforcement, using f'_m = 12 MPa and f_y = 300 MPa.

SOLUTION

Axial load: $$N_n = \frac{N^*}{\phi} = \frac{21.3}{0.85} \approx 25.0 \text{ kN/m}$$

Require $$M_n \geq \frac{M^*}{\phi}$$

$$\geq \frac{17}{0.85} = 20 \text{ kNm/m}$$

It is assumed that $M_n = M_p + M_s$, where M_p is moment capacity due to axial compression load N_n and M_s is moment capacity to be sustained by the flexural reinforcement.

As shown in Figure 12, moment due to N_n

$$a_1 = \frac{N_n}{0.85\, f'_m 1.0} = \frac{25 \times 10^3}{0.85 \times 12 \times 10^6}$$

$$= 2.45 \text{ mm}$$

Therefore $$M_p = N_n\left(\frac{t}{2} - \frac{a_1}{2}\right)$$

$$= 25 \times \left(\frac{190 - 2.45}{2}\right) = 2.34 \text{ kNm/m}$$

Now $$M_s = M_n - M_p$$

$$= 20 - 2.34$$

$$= 17.66 \text{ kNm/m}$$

Assuming $$a_2 \approx a_1 \frac{M_s}{M_p}$$

$$a_2 \approx 2.45 \times \frac{17.66}{2.34} \approx 18.5 \text{ mm}$$

$$M_s = A_s f_y\left(\frac{t}{2} - a_1 - \frac{a_2}{2}\right)$$

Therefore $$A_s \geq \frac{M_s}{f_y\left(\frac{t}{2} - a_1 - \frac{a_2}{2}\right)} = \frac{17.66 \text{ x } 10^3}{300 \text{ x}\left(95 - 2.45 - \frac{18.5}{2}\right) \text{x } 10^3}$$

$$= 707 \text{ mm}^2\text{/m}$$

Try D20 reinforcing bars spaced at 400 mm c/c, A_s = 785 mm²/m

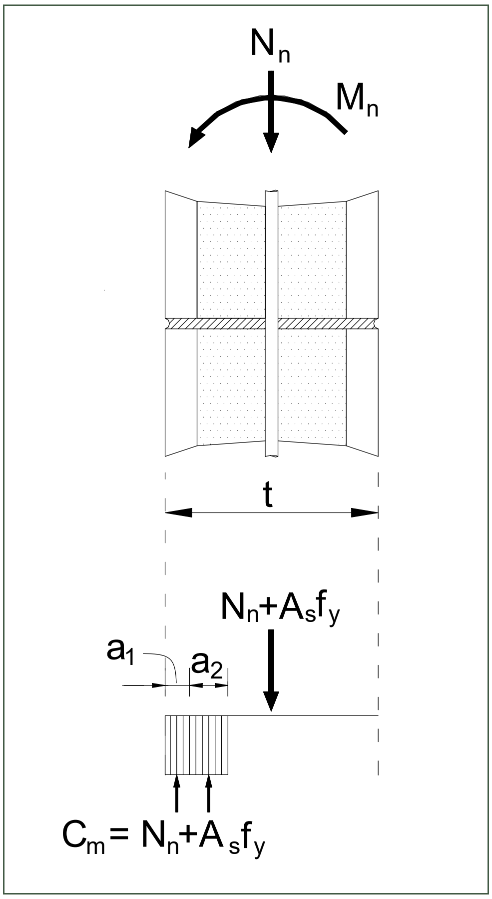


Figure 12: **Forces acting on wall**

Check

$$a = \frac{N_n + A_s f_y}{0.85\, f'_m\, 1.0} = \frac{25 \times 10^3 + 785 \times 300}{0.85 \text{ x } 12 \text{ x } 10^3} = 25.54 \text{ mm}$$

$$M_n = \left(N_n + A_s f_y\right) \times \left(\frac{t}{2} - \frac{a}{2}\right) = \left(25 \times 10^3 + 785 \times 300\right) \times \left(\frac{190 - 25.54}{2}\right) = 21.4 \text{ kNm/m} > \frac{M^*}{\phi}$$

3.4 Design of Shear Reinforcement

The single storey cantilevered concrete masonry wall of Figure 13 is to resist a shear force while responding elastically to the design earthquake. For a wall width of 140 mm, f'_m = 12 MPa and N* = 50 kN, design the required amount of shear reinforcement.

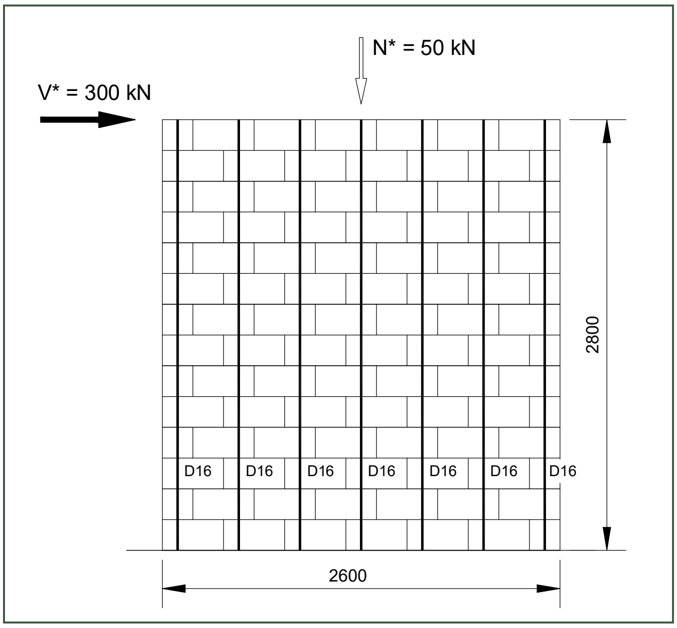


Figure 13: **Forces acting on masonry wall**

SOLUTION

N* = 50 kN

Therefore $N_n = \frac{N^*}{\phi} = \frac{50}{0.85} = 58.8$ kN

V* = 300 kN

Require $\phi V_n \geq V^*$

Therefore $V_n \geq \frac{V^*}{\phi}$

$$\geq \frac{300}{0.75}$$

$$\geq 400 \text{ kN}$$

Check maximum shear stress

$$V_n = \frac{V_n}{b_w d} \quad \text{note that } d = 0.8L_w \text{ for walls}$$

$$= \frac{400 \times 10^3}{140 \times 0.8 \times 2600}$$

$$= 1.37 \text{ MPa} < v_g$$

$$v_g = 1.50 \text{ MPa for } f'_m = 12 \text{ MPa}$$

Now $v_n = v_m + v_p + v_s$

Shear stress carried by v_m

$$v_m = (C_1 + C_2)v_{bm}$$

where $C_1 = 33p_w \dfrac{f_y}{300}$

and $p_w = \dfrac{7\text{bars} \times D16}{b_w d}$

$$= \frac{7 \times 201}{140 \times 0.8 \times 2600}$$

$$= 0.0048$$

$$\Rightarrow \quad C_1 = 33 \times 0.0048 \times \frac{300}{300}$$

$$= 0.16$$

$C_2 = 1.0$ since $h_e/L_w > 1.0$

Hence, $v_m = (0.16 + 1.0)v_{bm}$ where $v_{bm} = 0.70$ MPa for $\mu = 1$ and $f'_m = 12$ MPa

$$\Rightarrow \quad v_m = 1.16 \times 0.70 = 0.81 \text{ MPa}$$

Shear stress carried by v_p

$$v_p = 0.9\frac{N^*}{b_w d}\tan\alpha$$

where $N^* = 50$ kN

As illustrated in Figure 10.2 of NZS 4230:2004, it is necessary to calculate the compression depth **a** in order to establish tanα. The following illustrates the procedure of establishing compression depth **a** using Table 6:

$$p = \frac{7\text{bars} \times D16}{b_w \times L_w}$$

$$= \frac{7 \times 201}{140 \times 2600}$$

$$= 0.00387$$

$$p\frac{f_y}{f'_m} = 0.00387 \times \frac{300}{12} = 0.0967$$

and $$\frac{N_n}{f'_m L_w t} = \frac{58.8 \times 10^3}{12 \times 2600 \times 140} = 0.0135$$

From Table 6

$$\frac{c}{L_w} = 0.12$$

Therefore $c = 0.12 \times 2600$

$= 312$ mm

$\Rightarrow$ $a = \beta c$ (for unconfined concrete masonry, $\beta = 0.85$)

$= 0.85 \times 312$

$= 265.2$ mm

Therefore $$\tan\alpha = \frac{\frac{L_w}{2} - \frac{a}{2}}{h} = \frac{\frac{2600}{2} - \frac{265.2}{2}}{2800}$$

$= 0.417$

Hence, $$v_p = 0.9 \frac{50 \times 10^3}{140 \times 0.8 \times 2600} \times 0.417$$

$= 0.064$ MPa

Shear stress to be carried by v_s

$v_s = v_n - v_m - v_p = 1.37 - 0.81 - 0.064$

$= 0.50$ MPa

and $$v_s = C_3 \frac{A_v f_y}{b_w s}$$ where $C_3 = 0.8$ for a masonry walls

$\Rightarrow$ $$0.50 = 0.8 \frac{A_v \times 300}{140 \times 200}$$ Try $f_y = 300$ MPa and reinforcement spacing = 200 mm

$\Rightarrow A_v = 58.3\ \text{mm}^2$

Therefore, use **R10 @ 200 crs** = 78.5 mm^2 per 200 mm spacing.

It is essential that shear reinforcement be adequately anchored at both ends, to be fully effective on either side of any potentially inclined crack. This generally required a hook or bend at the end of the reinforcement. Although hooking the bar round the end vertical reinforcement in walls is the best solution for anchorage, it may induce excessive congestion at end flues and result in incomplete grouting of the flue. Consequently bending the shear reinforcement up or down into the flue is acceptable, particularly for walls of small width.

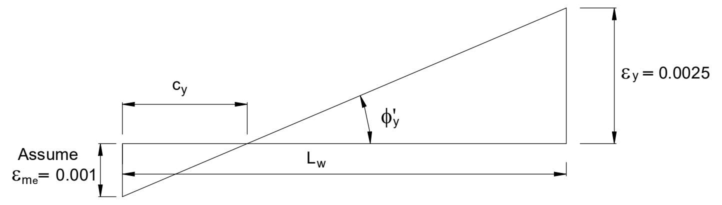
3.5 Concrete Masonry Wall Ductility Considerations

3.5.1 Neutral Axis of Limited Ductile Masonry Wall

Find the maximum allowable neutral axis depth for a limited ductile cantilever wall with aspect ratio of 3. The wall is reinforced with grade 500 reinforcement.

SOLUTION

$$\varepsilon_y = \frac{500}{200 \times 10^3} = 0.0025$$



For the purpose of an approximation that will generally overestimate the yield curvature, it may be assumed that $\varepsilon_{me} = 0.001$. This value would necessitate a rather large quantity of uniformly distributed vertical reinforcement in a rectangular wall, in excess of 1.5%. With this estimate the extrapolated yield curvature can be evaluated using Eqn. 2.

Using Eqn. 2: $\phi'_y = \dfrac{0.0025 + 0.001}{L_w} = \dfrac{0.0035}{L_w}$

Using Eqn. 3: $\phi_y = \dfrac{M_n}{M'_n}\phi'_y \Rightarrow \phi_y \approx \dfrac{4}{3}\phi'_y = \dfrac{4}{3} x \dfrac{0.0035}{L_w}$

Using Eqn. 11: $\mu_\phi = 3.18$ for $\mu = 2$ and $h_e / L_w = 3$

Consequently;

$$\phi_m = \frac{\varepsilon_u}{c_{max}} = \mu_\phi \phi_y$$

$$= \frac{0.003}{c_{max}} = 3.18 x \frac{4}{3} x \frac{0.0035}{L_w}$$

$$\Rightarrow c_{max} = 0.202 L_w \qquad [15]$$

3.5.2 Neutral Axis of Ductile Masonry Wall

Find the maximum allowable neutral axis depth for a ductile cantilever wall (Aspect ratio of 3) reinforced with grade 500 reinforcement.

SOLUTION

Repeating the above exercise using Eqn. 11 we obtain $\mu_\phi = 7.54$ for $\mu = 4$ and $h_e / L_w = 3$

Consequently; $\phi_m = \dfrac{\varepsilon_u}{c_{max}} = \mu_\phi \phi_y$

$$\phi_m = \frac{0.003}{c_{max}} = 7.54 \times \frac{4}{3} \times \frac{0.0035}{L_w}$$

$$\Rightarrow c_{max} = 0.085\ L_w$$

3.6 Ductile Cantilever Shear Wall

The 6 storey concrete masonry shear wall of Figure 14 is to be designed for the seismic lateral loads shown, which have been based on a ductility factor of μ = 4.0. Design gravity loads of 150 kN, including self weight, act at each floor and at roof level, and the weight of the ground floor and footing are sufficient to provide stability at the foundation level under the overturning moments. Wall width should be 190 mm. Design flexural and shear reinforcement for the wall.

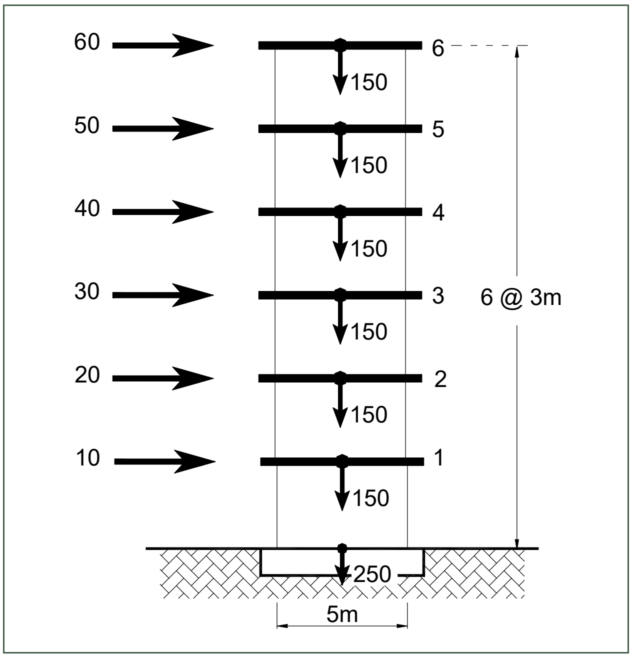


Figure 14: **Ductile Cantilever Shear Wall**

SOLUTION

Initially f'_m = 12 MPa will be assumed. From the lateral loads of Figure 14, the wall **base moment** is

$$M^* = 3 \times (60 \times 6 + 50 \times 5 + 40 \times 4 + 30 \times 3 + 20 \times 2 + 10)$$

$$= 2730 \text{ kNm}$$

Require $\phi M_n \geq M^*$

Therefore $M_n \geq \dfrac{M^*}{\phi}$

$$M_n \geq \frac{2730}{0.85}$$

$$\geq 3211 \text{ kNm}$$

Axial Load at Base

$$N^* = 6 \times 150 = 900 \text{ kN}$$

$$N_n = \frac{N^*}{\phi} = \frac{900}{0.85} = 1058.8 \text{ kN}$$

Check Dimensional Limitations

Assuming a 200 mm floor slab, the unsupported interstorey height = 2.8 m.

$$\frac{b_w}{L_n} = \frac{190}{2800} = 0.068 \qquad < 0.075$$

This is less than the general seismic requirement cited by the standard (clause 7.4.4.1). However, the axial load ratio is:

$$\frac{N_n}{f'_m L_w t} = \frac{1058.8 \times 10^3}{12 \times 5000 \times 190} = 0.093 \approx 0.1$$

And so from Table 6 we have

$$c < 0.3 L_n$$

Hence the less stringent demand of

$$\frac{b_w}{L_n} \geq 0.05$$

applies here (clause 7.3.3) and this is satisfied by the geometry of the wall.

Flexure and Shear Design

Dimensionless Design Parameters

$$\frac{M_n}{f'_m L_w^2 t} = \frac{3211.8 \times 10^6}{12 \times 5000^2 \times 190} = 0.0563$$

and $$\frac{N_n}{f'_m L_w t} = \frac{1058.8 \times 10^3}{12 \times 5000 \times 190} = 0.0929$$

From Figure 1 and assuming $f_y = 300$ MPa for flexural reinforcement

$$p\frac{f_y}{f'_m} = 0.04$$

Therefore $p = 0.0016$

Check Ductility Capacity

Check this using the ductility chart, Figure 8:

$$p\frac{12}{f'_m} = 0.0016 \quad \text{and} \quad \frac{N_n}{f'_m A_g} = 0.0929$$

Figure 6 gives $\mu_3 = 3.3$

Actual aspect ratio: $A_r = \frac{3 \times 6}{5} = 3.6$

Therefore from Eqn.14

$$\mu_{3.6} = 1 + \frac{3.3 \times (3.3-1) \times \left(1 - \frac{0.25}{3.6}\right)}{3.6} = 3.0 < \mu = 4 \text{ assumed}$$

Thus ductility is inadequate and redesign is necessary

Redesign for f’$_m$ =16 MPa. (Note that this will require verification of strength using the procedures reported in Appendix B of NZS 4230:2004).

Now New Dimensionless Design Parameters

$$\frac{N_n}{f'_m A_g} = \frac{1058.8 \times 10^3}{16 \times 5000 \times 190} = 0.0697$$

and $$\frac{M_n}{f'_m L_w^2 t} = \frac{3211.8 \times 10^6}{16 \times 5000^2 \times 190} = 0.0423$$

From Figure 1 and for f_y = 300 MPa for flexural reinforcement

$$p \frac{f_y}{f'_m} = 0.028$$

Therefore $p = \frac{0.028 \times 16}{300} = 0.0015$

Check Ductility Capacity

Using Figure 8, check the available ductility

$$p^* = p \frac{12}{f'_m} = 0.0015 \times \frac{12}{16} = 0.0011$$

$$\frac{N_n}{f'_m A_g} = 0.0697$$

From Figure 8, $\mu_3 \approx 4.5$

From Eqn. 14,

$$\mu_{3.6} = 1 + \frac{3.3 \times (4.5-1) \times \left(1 - \frac{0.25}{3.6}\right)}{3.6} = 3.98 \approx 4.0$$

Hence ductility OK

Flexural Reinforcement

For p = 0.0015 reinforcement per 400 mm will be

$$A_s = 0.0015 \times 400 \times 190 = \frac{114\ \text{mm}^2}{400\ \text{mm}}$$

Therefore use **D12 @ 400 mm crs** (113 mm^2/400 mm).

Shear Design

To estimate the maximum shear force on the wall, the flexural overstrength at the base of the wall, M_o, needs to be calculated:

$M_o = 1.25M_{n,provided}$ (for Grade 300 reinforcement)

$f'_m = 16$ MPa

$$\frac{N_n}{f'_m L_w t} = 0.070$$

$$p_{provided} = \frac{13\ \text{bars x } 113\ \text{mm}^2}{5000 \times 190} = 0.00155$$

and $$p\frac{f_y}{f'_m} = 0.00155 \times \frac{300}{16} = 0.029$$

From Table 2

$$\frac{M_n}{f'_m L_w^2 t} = 0.047$$

Therefore

$M_{n,provided} = 0.047 \text{ x } 16 \text{ x } 5000^2 \text{ x } 190 = 3580$ kNm

The overstrength value, $\phi_{o,w}$ is calculated as follows:

$$\varphi_{o,w} = \frac{M_o}{M^*} = \frac{1.25\,M_{n,provided}}{M^*} = \frac{1.25 \times 3580}{2730} = 1.64$$

Dynamic Shear Magnification Factor

For up to 6 storeys:

$$\omega_v = 0.9 + \frac{n}{10}$$

$$= 0.9 + \frac{6}{10} = 1.5$$

Hence, the design shear force at the wall base is

$$V_n = \omega_v \phi_{o,w} V^* = 1.5 \times 1.64 \times V^*$$

$$= 2.46V^*$$

$$= 2.46 \times 210$$

$$= 516.6 \text{ kN}$$

Check Maximum Shear Stress

$$v_n = \frac{V_n}{b_w d} = \frac{516.6 \times 10^3}{190 \times 0.8 \times 5000} = 0.68 \text{ MPa}$$

From Table 10.1 of NZS 4230:2004, the maximum allowable shear stress, v_g, for $f'_m = 16$ MPa is 1.8 MPa. Therefore OK.

Plastic Hinge Region

Within the plastic hinge region, $v_m = 0$. Therefore $v_p + v_s = 0.68$ MPa

and $$v_p = 0.9 \frac{N^*}{b_w d} \tan\alpha$$

As illustrated in Figure 10.2 of NZS 4230:2004, it is necessary to calculate the compression depth **a** in order to establish tanα.

To establish compression depth **a** using Table 6

$$p\frac{f_y}{f'_m} = 0.029 \quad \text{and} \quad \frac{N_n}{f'_m L_w t} = 0.0697$$

From Table 6

$$\frac{c}{L_w} = 0.126$$

Therefore $c = 0.126 \times 5000$

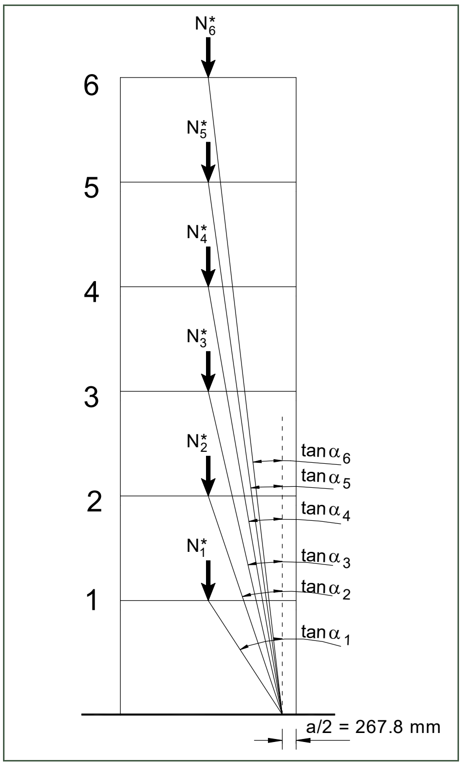
$= 630$ mm

$\Rightarrow$ $a = \beta c$ (for unconfined concrete masonry, $\beta = 0.85$)

$= 0.85 \times 630$

$= 535.5$ mm

Calculation of tanα



$$\tan\alpha_1 = \frac{2500 - 267.8}{3000} = 0.744$$

$$\tan\alpha_2 = \frac{2500 - 267.8}{6000} = 0.372$$

$$\tan\alpha_3 = \frac{2500 - 267.8}{9000} = 0.248$$

$$\tan\alpha_4 = \frac{2500 - 267.8}{12000} = 0.186$$

$$\tan\alpha_5 = \frac{2500 - 267.8}{15000} = 0.149$$

$$\tan\alpha_6 = \frac{2500 - 267.8}{18000} = 0.124$$

Figure 15: **Contribution of Axial Load**

Hence,
$$v_{p1} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.744 = 0.112 \text{ MPa}$$

$$v_{p2} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.372 = 0.066 \text{ MPa}$$

$$v_{p3} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.248 = 0.044 \text{ MPa}$$

$$v_{p4} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.186 = 0.033 \text{ MPa}$$

$$v_{p5} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.149 = 0.026 \text{ MPa}$$

$$v_{p6} = 0.9\frac{150 \times 10^3}{190 \times 0.8 \times 5000} \times 0.124 = 0.022 \text{ MPa}$$

$$\Rightarrow \quad v_p = v_{p1} + v_{p2} + v_{p3} + v_{p4} + v_{p5} + v_{p6} = 0.30 \text{ MPa}$$

Therefore, the required shear reinforcement:

$v_s = v_n - v_p$

$= 0.68 - 0.30$

$= 0.38$ MPa

$$v_s = C_3 \frac{A_v f_y}{b_w s}$$

where C_3 = 0.8 for a wall and the maximum spacing of transverse reinforcement = 200 mm since the wall height exceeds 3 storeys. Try f_y = 300 MPa.

$$0.38 = 0.8 \frac{A_v \times 300}{190 \times 200}$$

A_v = 60.2 mm^2/200 mm vertical spacing

Therefore use **R10 @ 200 crs** within plastic hinge region = 78.5 mm^2 per 200 mm spacing.

Outside Plastic Hinge Region

For example, immediately above level 2:

$$V_n = 1.5 \times 1.64 \times (60 + 50 + 40 + 30)$$

$$= 443 \text{ kN}$$

Therefore

$$v_n = \frac{443 \times 10^3}{b_w d} = 0.58 \text{ MPa}$$

From 10.3.2.6 of NZS 4230:2004

$$v_m = (C_1 + C_2) v_{bm}$$

where $C_1 = 33 p_w \frac{f_y}{300}$

$$= 33 \times \frac{13 \text{bars} \times 113}{b_w d} \frac{f_y}{300}$$

$$= 33 \times \frac{13 \times 113}{190 \times 0.8 \times 5000} \times \frac{300}{300}$$

$$= 0.064$$

and C_2 = 1.0 since h_e/L_w > 1.0

Therefore $v_m = (C_1 + C_2) v_{bm}$

$$= (0.064 + 1) \times 0.2\sqrt{16}$$

$$= 0.85 \text{ MPa} > v_n$$

Since $v_m > v_n$, only minimum shear reinforcement of 0.07% is required. Take s = 400 mm,

$$A_v = 0.07\% \times 400 \times 190 = 53.2 \text{ mm}^2$$

Therefore, use **R10 @ 400 crs** outside plastic hinge region.

3.7 Limited Ductile Wall with Openings

The seismic lateral loads for the 2 storey masonry wall of Figure 16 are based on the limited ductile approach, corresponding to $\mu = 2$. Design gravity loads (both dead and live) including self weight are 20 kN/m at the roof, and 30 kN/m at levels 0 and 1. It is required to design the reinforce-ment for the wall, based on the limited ductility provisions of NZS 4230:2004, using $f'_m = 16$ MPa and $f_y = 300$ MPa. The wall thickness is 190 mm.

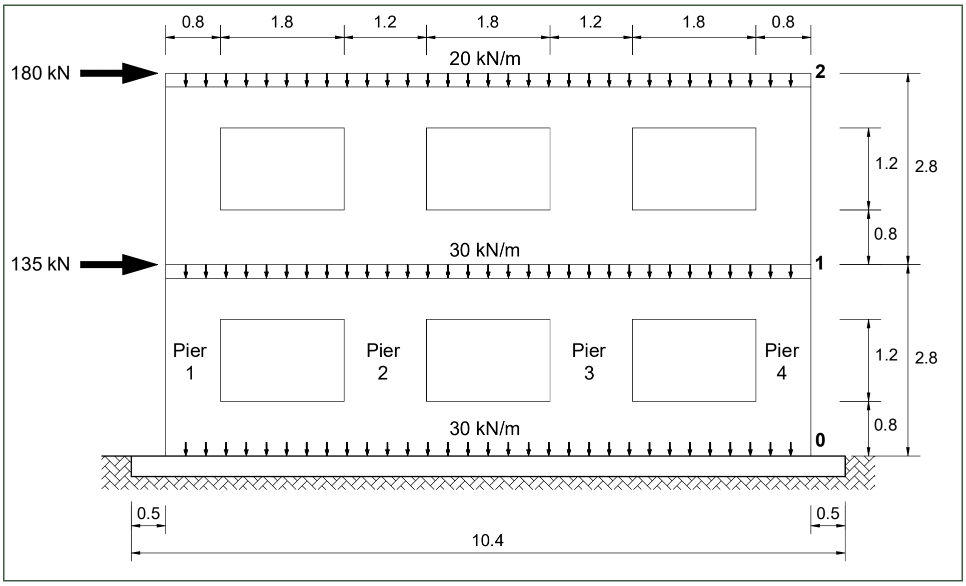


Figure 16: **Limited Ductile 2-Storey Masonry Wall with Openings.**

SOLUTION

As the structure is 2 storeys high, it may be designed for pier* hinging or spandrel* hinging as outlined in section 2.6.7.2(b) of NZS 3101:2006. Because of the relative proportions it is expected that pier hinging will initiate first, and this behaviour is assumed below. Consequently, the piers are identified as potential hinging areas. In accordance with section 3.7.3.3 of the standard, the spandrels are required to be designed for 50% higher moments than design level moments, with shear strength enhanced by 100% in spandrels and piers.

Axial Load

Assume each pier is loaded by the appropriate tributary area:

Axial load, 1st storey

Piers 1 and 4: $N_{G+Qu} = (20 + 30) \times (0.8 + 0.9) = 85$ kN
Piers 2 and 3: $N_{G+Qu} = 50 \times (1.2 + 1.8) = 150$ kN

Axial load, 2nd storey

Piers 1 and 4: $N_{G+Qu} = 20 \times (0.8 + 0.9) = 34$ kN
Piers 2 and 3: $N_{G+Qu} = 20 \times (1.2 + 1.8) = 60$ kN

Dimensional Limitations

Minimum thickness of piers:

$b_w = 190$ mm, $L_n = 1200$ mm

$$\frac{b_w}{L_n} = \frac{190}{1200} = 0.15$$

---

* Within this user guide, pier refers to the part of a wall or column between two openings, and spandrel refers to the deep beam above an opening.

This is more than the general seismic requirement of $b_w \geq 0.075L_n$ cited by the standard (7.4.4.1 of NZS 4230:2004).

Dimensional limitations of spandrels:

Spandrels at level 1 are more critical due to deeper beam depth. Therefore

$$b_w = 190 \text{ mm},\ h = 1600 \text{ mm and } L_n = 1800 \text{ mm}$$

$$\frac{L_n}{b_w} = \frac{1800}{190} = 9.5 \quad < 20$$

and
$$\frac{L_n h}{b_w^2} = \frac{1800 \times 1600}{190^2} = 79.8 \quad < 80$$

The spandrels are within the dimensional limitations required by the standard (clause 8.4.2.3).

Determination of Seismic Lateral Forces in 1st Storey Piers

It is assumed that the spandrels are sufficiently stiff to force mid-height contraflexure points in the piers. The traditional approach of allocating lateral force to inelastically responding members in proportion to their assumed stiffness has been reported[6] to commonly lead to significant errors, regardless of whether gross stiffness or some fraction of gross stiffness is assumed. This is because walls of different length in the same direction will not have the same yield displacement. This can be illustrated by substituting Eqns. 2 and 3 into Eqn. 6 to give:

$$\Delta_y = \frac{M_n}{M'_n} \times \frac{\varepsilon_y + \varepsilon_m}{L_w} \times \frac{h_w^2}{3}$$

which indicates that the yield displacement is inversely proportional to wall length. This means that the basic presumption of the traditional approach, to allocate lateral load to walls in proportion to their stiffness as a means to obtain simultaneous yielding of the walls, and hence uniform ductility demand, is impossible to achieve. It was also shown by Paulay[7] that the yield curvature (ϕ_y) of a structural wall is insensitive to axial load ratio. As a consequence, it is possible to define ϕ_y as a function of wall length alone.

The moments and shears in the piers can be found from the method suggested by Paulay[7]. This design approach assigns lateral force between piers in proportion to the product of element area, $A_n = b_w L_w$, and element length, L_w, rather than the second moment of area of the section, as would result from a stiffness approach, i.e. the pier strength should be allocated in proportion to L_w^2 rather than L_w^3. Consequently the pier shear forces and moments are as summarised in Tables 9 and 10.

Table 9: **Pier Shear Forces**

Pier	Length, L_w (m)	L_w^2 (m^2)	$\frac{L_{wi}^2}{\Sigma L_{wi}^2}$	V_E (kN)	
				1st Storey	2nd Storey
1	0.8	0.64	0.154	48.5	27.7
2	1.2	1.44	0.346	109.0	62.3
3	1.2	1.44	0.346	109.0	62.3
4	0.8	0.64	0.154	48.5	27.7
	Σ	4.16	1.0	315	180

6 Priestley, M. J. N., and Kowalsky, M. J. (1998) “Aspects of Drift and Ductility Capacity of Rectangular Cantilever Structural Walls”, Bulletin of NZNSEE, Vol. 31, No. 2, pp. 73-85.

7 Paulay, T. (1997) “A Review of Code Provision for Torsional Seismic Effects in Buildings”, Bulletin of NZNSEE, Vol. 30, No. 3, pp. 252-263.

Table 10: **Pier Shear Forces and Moments**

Parameter	Units	Pier 1	Pier 2	Pier 3	Pier 4	Σ
First Storey						
V^*_E	kN	48.5	109.0	109.0	48.5	315
M^*_E (1)	kNm	29.1	65.4	65.4	29.1	
M_{cl} (2)	kNm	67.9	152.6	152.6	67.9	
Second Storey						
V^*_E	kN	27.7	62.3	62.3	27.7	180
M^*_E (1)	kNm	16.6	37.4	37.4	16.6	
$M_{cl,\ top}$ (2)	kNm	27.7	62.3	62.3	27.7	
$M_{cl,\ bottom}$ (2)	kNm	38.8	87.2	87.2	38.8	

(1) Moments at critical pier i section
(2) Moments at spandrel centrelines, pier i

Note that in Table 10, the pier shear forces are used to establish the pier bending moments. For instance, the first storey bending moments of pier 1 are found from:

$$M_E^* = V_E^* \times \frac{h}{2} = 48.5 \times \frac{1.2}{2} = 29.1\,\text{kNm}$$

Spandrel moments and shears are found by extrapolating the pier moments to the pier/spandrel intersection points, then imposing moment equilibrium of all moments at a joint. At interior joints, the moments in the spandrels on either side of the joint are estimated, considering equilibrium requirements, by the assumption that the spandrel moment on one side of a joint centreline is equal to the ratio of the lengths of the adjacent span times the spandrel moment on the other side of the joint. For example, with regard to Figure 17b, at joint 2 the beam moment to the left of the centreline, M_{s21}, may be expressed as:

$$M_{s21} = \frac{\text{length of spandrel }(2-3)}{\text{length of spandrel }(1-2)} \times M_{s23} \qquad [16]$$

Hence

$$M_{s21} = \frac{\text{length of spandrel }(2-3)}{\text{length of spandrel }(1-2) + \text{length of spandrel }(2-3)} \times \sum \left(\begin{matrix}\text{pier centreline} \\ \text{moments at joint 2}\end{matrix} \right) \qquad [17]$$

More sophisticated analyses are probably inappropriate because of the deep members, large joints and influence of cracking and shear deformations. The resulting pier and spandrel moments and shears are plotted in Figure 17b. Axial forces in the piers are found from the resultant of beam shear (vertical equilibrium), and these are presented in Table 11.

Table 11: **Revised Total Axial Load**

Pier	$N^* = N_{G+Qu} + N_E$ (kN)	
	1st Storey	2nd Storey
1	85 - 103.8 = -18.8	34 - 21.4 = 12.6
2	156.5	61.3
3	143.5	58.7
4	188.8	55.4

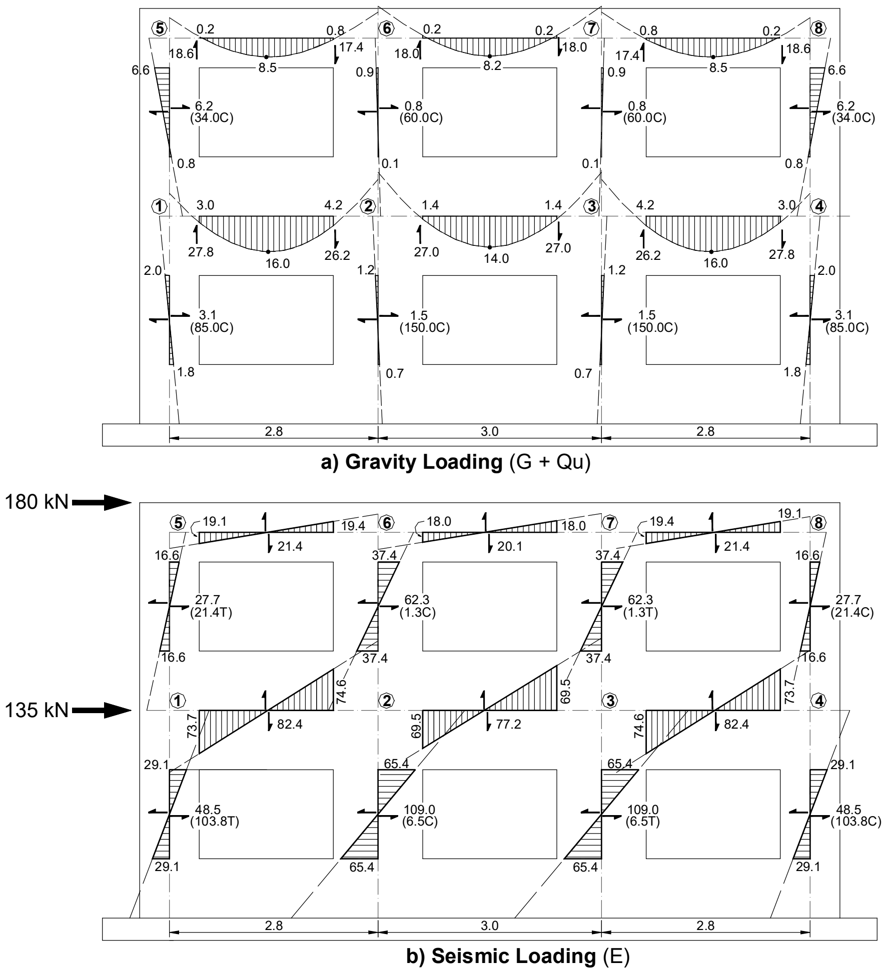


Figure 17: **Forces and Moments for the 2-Storey Masonry Wall** (Forces, Shears in kN, Moment in kNm, Axial Forces in parentheses)

Design of 1st Storey Piers

Flexural Design

Outer Piers

Outer piers are designed for the worst of Pier 1 and Pier 4 loading. Since the piers have been chosen as the ductile elements, the moments in Figure 17 are the design moments, i.e. $M^* = M^*_G + M^*_{Qu} + M^*_E$.

Pier 1

$N^* = -18.8$ kN

$M^* = M^*_G + M^*_{Qu} + M^*_E$ = -2.0 + 29.1 = 27.1 kNm (Note that $M^*_G + M^*_{Qu}$ = -2.0 kNm)

Therefore

$$Nn = \frac{N^*}{\phi} = \frac{-18.8}{0.85}$$

= -22.1 kN

and $$M_n \geq \frac{M^*}{\phi}$$

$$M_n \geq \frac{27.1}{0.85}$$

$$\geq 31.9 \text{ kNm}$$

Dimensionless Design Parameters

$$\frac{N_n}{f'_m L_w t} = \frac{-22.1 \times 10^3}{16 \times 800 \times 190} = -0.0091$$

and $$\frac{M_n}{f'_m L_w^2 t} = \frac{31.9 \times 10^6}{16 \times 800^2 \times 190} = 0.0164$$

From Figure 1, $p\frac{f_y}{f'_m} = 0.037$

<u>Pier 4</u>

$$N^* = 188.8 \text{ kN}$$

$$M^* = M^*_G + M^*_{Qu} + M^*_E = 2.0 + 29.1 = 31.1 \text{ kNm}$$

Therefore

$$N_n = \frac{N^*}{\phi} = \frac{188.8}{0.85} = 222.1 \text{ kN}$$

and $$M_n \geq \frac{M^*}{\phi}$$

$$M_n \geq \frac{31.1}{0.85}$$

$$\geq 36.6 \text{ kNm}$$

Dimensionless Design Parameters

$$\frac{N_n}{f'_m L_w t} = \frac{222.1 \times 10^3}{16 \times 800 \times 190} = 0.091$$

and

$$\frac{M_n}{f'_m L_w^2 t} = \frac{36.6 \times 10^6}{16 \times 800^2 \times 190} = 0.0188$$

From Figure 1, $p\frac{f_y}{f'_m} < 0.00$

$\Rightarrow$ Pier 1 governs

Now $p\dfrac{f_y}{f'_m} = 0.037$ for $f_y = 300$ MPa and $f'_m = 16$ MPa

$$\Rightarrow p = \frac{0.037 \times 16}{300} = 0.002$$

As the structure is designed as one of limited ductility, the requirements of clause 7.4.5.1 of NZS 4230:2004 apply for spacing and bar size. Consequently, it is required to adopt minimum bar size of D12 and minimum of 4 bars, i.e. 200 crs. With D12 at 200 crs, $p = \dfrac{\pi \times 12^2}{4 \times 200 \times 190} = 0.00297$. This exceeds the p = 0.002 required. Refer to Figure 18 for details.

Inner Piers

Inner piers are designed for the worst loading conditions of Piers 2 and 3.

From Figure 17, it may be determined that Pier 3 governs design due to larger bending moment and lighter axial compressive load.

Pier 3:

$N^* = 143.5$ kN

$M^* = M^*_G + M^*_{Qu} + M^*_E = 1.2 + 65.4 = 66.6$ kNm

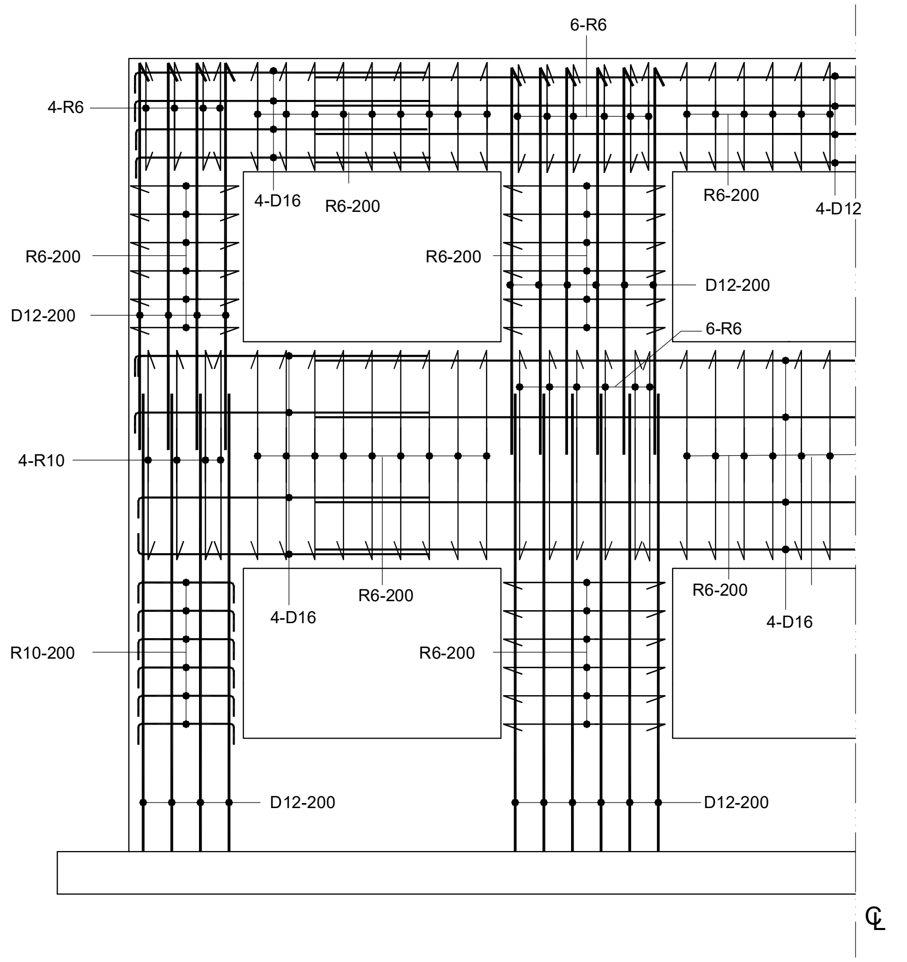


Figure 18: **Reinforcement for Design Example 3.7**

Therefore

$$N_n = \frac{N^*}{\phi} = \frac{143.5}{0.85}$$

$$= 168.8 \text{ Kn}$$

and $$M_n \geq \frac{M^*}{\phi}$$

$$M_n \geq \frac{66.6}{0.85}$$

$$\geq 78.2 \text{ kNm}$$

Dimensionless Design Parameters

$$\frac{N_n}{f'_m L_w t} = \frac{168.8 \times 10^3}{16 \times 1200 \times 190} = 0.046$$

and $$\frac{M_n}{f'_m L_w^2 t} = \frac{78.2 \times 10^6}{16 \times 1200^2 \times 190} = 0.0179$$

From Figure1, $p\frac{f_y}{f'_m} \approx 0.00$

Therefore use D12 @ 200 for the two inner piers to satisfy the requirements of clause 7.4.5.1.

Refer to Figure 18 for details.

Ductility Checks

Clause 7.4.6.1 of NZS 4230:2004 requires that for walls with contraflexure point between adjacent heights of lateral support:

$$c \leq 0.45 L_w^2 / L_n$$

where L_w is the wall length, and L_n is the unsupported height.

Note that calculations should be conducted using the amount of reinforcement required ($p_{required}$) rather than the amount of reinforcement actually provided, as the latter results in a higher moment capacity, and hence reduced ductility demand, for which a higher value of **c** could be tolerated.

Pier	$c_{max} = \frac{0.45L_w^2}{L_n}$	$\frac{N_n}{f'_m A_g}$	$p_{required}\frac{f_y}{f'_m}$	$c_{required}$ (from Table 6)	
1	240	-0.006	0.040	40	OK
2	540	0.050	0.000	83	OK
3	540	0.042	0.000	70	OK
4	240	0.082	0.000	91	OK
Units	mm	---	---	mm	

Shear Design, 1st Storey

From NZS 4230:2004: $\phi V_n \geq V_G^* + V_{Qu}^* + 2V_E^*$ where $\phi = 0.75$

Outer Piers

Pier 1 governs due to the presence of axial tension force, $V^* = -3.1 + 2 \times 48.5 =$ 93.9 kN (where $V_G^* + V_{Qu}^* = -3.1\,\text{kN}$ and $V_E^* = 48.5\,\text{kN}$)

$$\Rightarrow V_n = \frac{93.9}{0.75} = 125.2\,\text{kN}$$

Now for Type A masonry, $v_g = 0.45\sqrt{f'_m} = 0.45 \times \sqrt{16} = 1.8$ MPa

Check shear stress, b_w = 190 mm, d = 0.8 x 800 = 640 mm

$$v_n = \frac{V_n}{b_w d} = \frac{125.2 \times 10^3}{190 \times 640} = 1.03\,\text{MPa} \qquad \leq v_g$$

From Section 10.3 of NZS 4230:2004:

$$v_n = v_m + v_p + v_s$$

Shear stress carried by $v_m = (C1 + C2)v_{bm}$

where $C_1 = 33 p_w \dfrac{f_y}{300}$

and $p_w = 0.00297$

$$\Rightarrow C_1 = 33 \times 0.00297 \times \frac{300}{300} = 0.098$$

and $C_2 = 0.42\left[4 - 1.75\left(\dfrac{h_e}{L_w}\right)\right]$

$$\Rightarrow C_2 = 0.42\left[4 - 1.75 \times 1200/(2 \times 800)\right]$$

$$\Rightarrow C_2 = 1.12$$

Hence,

$v_m = (0.098 + 1.12)v_{bm}$ where $v_{bm} = 0.15\sqrt{f'_m}$ for $\mu = 2$.

$$\Rightarrow v_m = 0.73\,\text{MPa}$$

Shear stress carried by $v_p = 0.9\dfrac{N^*}{b_w d}\tan\alpha$

where $N^* = -18.8$ kN

$$\Rightarrow N_n = \frac{N^*}{\phi} = -22.1\,\text{kN}$$

and $\quad p = 0.00297$

$$\frac{N_n}{f'_m L_w t} = -0.0091$$

and $\quad p\dfrac{f_y}{f'_m} = 0.0557$

From Table 6, $\dfrac{c}{L_w} \approx 0.068$

For Pier 1 with L_w = 800 mm,

$$\Rightarrow c = 54.4 \text{ mm}$$

Therefore, a = 0.85 x c = 46.2 mm

Consequently, for pier in double bending $\tan\alpha = \dfrac{800-46.2}{1200} = 0.628$

$$\Rightarrow v_p = 0.9 \times \frac{-18.8 \times 10^3}{190 \times 0.8 \times 800} \times 0.628 = -0.087 \text{ MPa}$$

Shear stress to be carried by $v_s = v_n - v_m - v_p$

$$v_s = v_n - v_m - v_p = 1.03 - 0.73 - (-0.087)$$

$$= 0.39 \text{ MPa}$$

and $\quad v_s = C_3 \dfrac{A_v f_y}{b_w s} \qquad$ where C_3 = 0.8 for masonry walls

$\Rightarrow 0.39 = 0.8 \dfrac{A_v \times 300}{190 \times 200} \qquad$ Try f_y = 300 MPa and reinforcement spacing = 200 mm

$$\Rightarrow A_v = 61.8 \text{ mm}^2$$

Therefore, use <u>R10 @ 200 crs</u> (78.5 mm^2)

Inner Piers

Clearly, Pier 3 governs due to lighter compression load, $V^* = 1.5 + 2 \times 109.0 = 219.5$ kN

$$V_n = \frac{219.5}{0.75} = 292.7 \text{ kN}$$

Check shear stress, b_w = 190 mm, d = 0.8 x 1200 = 960 mm

$$\Rightarrow v_n = \frac{V_n}{b_w d} = \frac{292.7 \times 10^3}{190 \times 960} = 1.60 \text{ MPa} \qquad < v_g$$

Shear stress carried by $v_m = (C_1 + C_2)v_{bm}$

where $C_1 = 33p_w \dfrac{f_y}{300}$

and $p_w = 0.00297$

$\Rightarrow C_1 = 0.098$

and $C_2 = 0.42\left[4 - 1.75\left(\dfrac{h_e}{L_w}\right)\right]$

$\Rightarrow C_2 = 0.42\left[4 - 1.75 \times 1200/(2 \times 1200)\right]$

$\Rightarrow C_2 = 1.31$

Hence,

$v_m = (0.098 + 1.31) \times 0.15\sqrt{16}$ where $v_{bm} = 0.15\sqrt{f'_m}$ for $\mu = 2$.

$\Rightarrow v_m = 0.84$ MPa

Shear stress carried by $\mathbf{v_p = 0.9\dfrac{N^*}{b_w d}\tan\alpha}$

where $N^* = 143.5$ Kn

$\Rightarrow N_n = \dfrac{143.5}{0.85} = 168.8$ kN

and $p = 0.00297$

$\dfrac{N_n}{f'_m L_w t} = 0.046$

and $p\dfrac{f_y}{f'_m} = 0.0557$

From Table 6, $\dfrac{c}{L_w} = 0.122$

For Pier 3 with $L_w = 1200$, $c = 0.122 \times 1200 = 146.4$ mm

Therefore $a = 0.85 \times 146.4 = 124.4$ mm

Consequently $\tan\alpha = \dfrac{1200 - 124.4}{1200} = 0.90$ for pier in double bending

$$\Rightarrow v_p = 0.9 \times \frac{143.5 \times 10^3}{190 \times 0.8 \times 1200} \times 0.90 = 0.64 \text{ MPa}$$

Shear stress to be carried by $v_s = v_n - v_m - v_p$

$$v_s = v_n - v_m - v_p = 1.60 - 0.84 - 0.64$$

$$= 0.12 \text{ MPa}$$

$$v_s = C_3 \frac{A_v f_y}{b_w s}$$ where C_3 = 0.8 for masonry walls

$$\Rightarrow 0.12 = 0.8 \frac{A_v \times 300}{190 \times 200}$$ Try f_y = 300 MPa and reinforcement spacing = 200 mm

$$\Rightarrow A_v = 19.0 \text{ mm}^2$$

However, this is less than the p_{min} = 0.07% required by clause 7.3.4.3 of the standard. Therefore, use R6 @ 200 crs (28.2 mm^2) to give p = 0.074%.

Design of 2nd Storey

The procedure is the same as for 1st storey and is not repeated here. Minimum requirements of D12 @ 200 again govern flexure, but shear reinforcement in the outer piers can be reduced to 0.07% of the gross cross-sectional area of the wall (minimum reinforcement area required by clause 7.3.4.3).

Flexural Design, Level 2 Spandrels

Section 3.7.3 of NZS 4230:2004 requires

$$\phi M_n \geq M_G^* + M_{Qu}^* + 1.5M_E^*$$

Spandrels 1-2 and 3-4

Design for the maximum moments adjacent to Joint 3, $M_G^* + M_{Qu}^* = 4.2$ kNm and $M_E^* = 74.6$ kNm.

Therefore $M^* = 4.2 + 1.5 \times 74.6 = 116.1$ kNm

Note that beam depth = 1.6 m and N* = 0

$$M_n = \frac{116.1}{0.85} = 136.6 \text{ kNm}$$

Dimensionless Design Parameter

$$\frac{M_n}{f'_m L_w^2 t} = \frac{136.6 \times 10^6}{16 \times 1600^2 \times 190} = 0.0176$$

From Table 2,

$$p\frac{f_y}{f'_m} = 0.037$$

$$\Rightarrow p = \frac{0.037 \times 16}{300} = 0.00197$$

Therefore use D16 @ 400 crs (average p = 0.00265), i.e. cells 1, 3, 6 and 8 from top. See Figure 18 for details.

<u>Spandrel 2-3</u>

Design for the maximum moment of $M^* = 1.4 + 1.5 \times 69.5$ = 105.7 kNm, adjacent to Joint 2.

Therefore $\quad M_n = \dfrac{105.7}{0.85} = 124.4$ kNm

Dimensionless Design Parameter

$$\frac{M_n}{f'_m L_w^2 t} = \frac{124.4 \times 10^6}{16 \times 1600^2 \times 190} = 0.016$$

From Table 2,

$$p\frac{f_y}{f'_m} = 0.034$$

$$\Rightarrow p = \frac{0.034 \times 16}{300} = 0.0018$$

Therefore continue <u>D16 @ 400 crs</u> right through Spandrel 2-3.

Shear Design, Level 2 Spandrels

Design requirement $\quad \phi V_n \geq V_G^* + V_{Qu}^* + 2V_E^*$, and $\phi = 0.75$ for shear

Spandrels 1-2 and 3-4

$$V^* = 27.8 + 2 \times 82.4 = 192.6 \text{ kN (adjacent to Joint 4)}$$

$$V_n = \frac{192.6}{0.75} = 256.8 \text{ kN}$$

$$\Rightarrow v_n = \frac{V_n}{b_w d} = \frac{256.8 \times 10^3}{190 \times 0.8 \times 1600} = 1.06 \text{ MPa} \quad < v_g$$

Since beams are assumed not to be hinging (pier flexural demand, ϕM_n, was met, therefore flexural capacity of spandrels has an additional reserve strength of $1.5M_E^*$). Consequently, $v_{bm} = 0.2\sqrt{f'_m}$, see Table 10.1 of NZS 4230:2004.

$$v_m = (C_1 + C_2)v_{bm}$$

where $\quad C_1 = 33p_w \dfrac{f_y}{300}$ note that $p_w = 0.00265$

$$\Rightarrow C_1 = 0.087$$

and $\quad C_2 = 1$ for beams

$$\Rightarrow v_m = (0.087 + 1) \times 0.2\sqrt{16} = 0.87 \text{ MPa}$$

Therefore $v_s = v_n - v_m - v_p$

and $v_p = 0$

$\Rightarrow \quad v_s = 1.06 - 0.87 - 0$

$= 0.19$ MPa

$$v_s = C_3 \frac{A_v f_y}{b_w s}$$ note that $C_3 = 1.0$ for beams

Clause 10.3.2.10 requires spacing of shear reinforcement, placed perpendicular to the axis of component not to exceed 0.5d or 600 mm.

Therefore, maximum shear reinforcement spacing, $s_{max} = 600$ mm

$\Rightarrow$ Try s = 200 mm and $f_y = 300$ MPa

$$v_s = \frac{A_v \times 300}{190 \times 200}$$

$$\Rightarrow \quad 0.19 = \frac{A_v \times 300}{190 \times 200}$$

$$\Rightarrow \quad A_v = 24.1 \text{ mm}^2$$

Use R6 @ 200 crs (i.e. $A_v = 28$ mm^2 per 200 mm). This is also the minimum area of reinforcement of 0.07% required by clause 7.3.4.3 of the standard.

Spandrels 2-3

$$V^* = 27.0 + 2 \times 77.2 = 181.4 \text{ kN}$$

$$V_n = \frac{181.4}{0.75} = 241.9 \text{ kN}$$

$$\Rightarrow \quad v_n = \frac{V_n}{b_w d} = \frac{241.9 \times 10^3}{190 \times 0.8 \times 1600} = 0.99 \text{ MPa} \quad < v_g$$

$$v_m = (C_1 + C_2) v_{bm}$$

where $C_1 = 33 p_w \frac{f_y}{300}$ note that $p_w = 0.00265$

$\Rightarrow C_1 = 0.087$

and $C_2 = 1.0$ for beams

$$\Rightarrow \quad v_m = (0.087 + 1) \times 0.2\sqrt{16} = 0.87 \text{ MPa}$$

Therefore $v_s = v_n - v_m - v_p$

$\Rightarrow \quad v_s = 0.99 - 0.87 - 0$ (note that $v_p = 0$)

$v_s = 0.12$

$$v_s = \frac{A_v \times 300}{190 \times 200} \qquad \text{Try s = 200 mm and } f_y = 300 \text{ MPa}$$

$$\Rightarrow \qquad 0.12 = \frac{A_v \times 300}{190 \times 200}$$

$$\Rightarrow \qquad A_v = 15.2 \text{ mm}^2$$

Therefore use R6 @ 200 crs.

Design of Level 3 Spandrels

The design of level 3 spandrels is similar to above and is not included herein.

Beam-Column Joints

Check dimensional limitations

Minimum vertical dimension, h_b:

Interior joints (11.4.2.3a of NZS 4230:2004):

h_b = 1600 mm

d_{bc} = 12 mm

$$\text{Therefore } \frac{h_b}{d_{bc}} = \frac{1600}{12} = 133 \quad > 70$$

Exterior joints (11.4.2.5):

h_b = 800 mm

d_{bc} = 12 mm

$$\text{Therefore } \frac{h_b}{d_{bc}} = \frac{800}{12} = 67$$ This is about 4% shortfall of the requirement, therefore OK

Minimum horizontal dimension, h_c:

Interior joints (11.4.2.2b):

h_c = 1200 mm
d_{bb} = 16 mm

$$\text{Therefore } \frac{h_c}{d_{bb}} = \frac{1200}{16} = 75 \quad > 60$$

Exterior joints (11.4.2.4):

$$\begin{aligned} \text{required } h_c &= \text{cover} + L_{dh} + 10d_{bb} \\ &= 100 + 20d_b + 10d_{bb} \\ &= 100 + (20 \times 16) + (10 \times 16) \\ &= 580 \text{ mm} < h_c \text{ provided is 800 mm, therefore OK.} \end{aligned}$$

Joint Shear Design

The joints should be designed to the provisions of Section 11 of NZS 4230:2004. At level 2, the critical joints are 3 and 4. If there is doubt as to the critical joints then it is prudent to evaluate **all** joints.

An estimation of the joint shear force may be found by the appropriate slope of the moment gradient through the joint (Paulay and Priestley, 1992). Hence, the horizontal shear V_{jh} and vertical shear V_{jv} at a joint are approximated by:

$$V_{jh} \approx \frac{M_t + M_b - \frac{(V_{bL} + V_{bR})h'_c}{2}}{h'_b}$$

$$V_{jv} \approx \frac{M_L + M_R - \frac{(V_{col\,t} + V_{col\,b})h'_b}{2}}{h'_c}$$

where M_t, M_b, M_L and M_R are the moments at top, bottom, left and right of the joint. V_{bL} and V_{bR} are the shears applied to the left and right sides of the joint (from the beams) and, $V_{col\,t}$ and $V_{col\,b}$ are the shears applied to the top and bottom of the joint (from the columns). The h_b and h_c are the beam and column depths respectively, where $h'_b \approx 0.9h_b$ and $h'_c \approx 0.9h_c$. The h'_b and h'_c are approximate distance between the lines of action of the flexural compression found in the beams and columns on opposite sides of the joints.

Level 2 Joint Shear Design

Joint 3

Horizontal Joint Shear

Gravity induced joint shear:

$$V_{G+Qu,jh} = \frac{0.1 + 1.2 - \frac{1}{2}[27.0 + (-26.2)] \times 0.9 \times 1.2}{0.9 \times 1.6} = 0.60\text{ kN}$$

As illustrated here, joint shear resulted from gravity loads is small. Consequently, gravity induced joint shear could be considered negligible in this instance.

Earthquake induced joint shear:

$$V_{E,jh} = \frac{37.4 + 65.4 - \frac{1}{2}(77.2 + 82.4) \text{ x } 0.9 \text{ x } 1.2}{0.9 \text{ x } 1.6} = 11.5\text{ kN}$$

Limited ductility design requires

$$\phi V_n = V_{jh} = V_{G+Qu,jh} + 2V_{E,jh}$$

$\Rightarrow$ $V_{jh} = 0 + 2 \text{ x } 11.5$ (Gravity induced joint shear is considered negligible)

$= 23.0$ kN

Nominal shear stress in the joint

$$v_{jh} = \frac{V_{jh}}{b_c h_c} = \frac{23.0 \times 10^3}{190 \times 1200} = 0.10\text{ MPa} < v_g = 0.45\sqrt{16} = 1.8\text{ MPa}$$

Therefore OK

From section 11.4.5.2, since beams remain elastic (i.e. no hinging)

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{mh}$$

where $V_{mh} = 0.5V_{jh} = 11.5$ kN

but need not be taken less than $V_{mh} = v_m b_c h_c$

where $v_m = (C_1 + C_2) v_{bm}$

and $C_1 = 33p_w \frac{f_y}{300}$

$p_w = 0.00297$ (for D12 @ 200 crs)

$\Rightarrow \quad C_1 = 0.098$

$C_2 = 1.0$ for simplicity

$$\Rightarrow \quad v_m = (0.098 + 1) \times 0.2\sqrt{f'_m}$$

$$= 0.22 \times \sqrt{16}$$

$$= 0.88 \text{ MPa}$$

Therefore $V_{mh} = 0.88 \times 0.19 \times 1.2 \times 10^3 = 200.6$ kN

Hence

$$V_{sh} = \frac{23.0}{0.75} - 200.6 \ < \text{ZERO}$$

Therefore NO horizontal joint steel is required (i.e. $A_{jh} = 0$). The horizontal shear is carried by the horizontal component of the diagonal strut across the joint.

Vertical Joint Shear

Earthquake induced joint shear:

$$V_{E,jv} = \frac{69.5 + 74.6 - \left(\frac{62.3 + 109.0}{2}\right) \text{x } 0.9 \text{ x } 1.6}{0.9 \text{ x } 1.2}$$

$$= 19.2 \text{ kN}$$

$\phi V_n = V_{jv} = 2V_{E,jv}$ (Gravity induced joint shear is considered negligible in this instance)

$\Rightarrow \quad V_{jv} = 38.4$ kN

Nominal shear stress in the joint

$$v_{jv} = \frac{V_{jv}}{b_c h_b} = \frac{38.4 \times 10^3}{190 \times 1600} = 0.13 \text{ MPa} < v_g \qquad \text{Therefore OK}$$

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{mv}$$

where $V_{mv} = 0$ since potential plastic hinge regions are expected to form in the pier above and below the joint (see 11.4.6.2 of NZS 4230:2004).

Hence

$$V_{sv} = \frac{38.4}{0.75} - 0 = 51.2 \text{ kN}$$

and the total area of vertical joint shear reinforcement required:

$$A_{jv} = \frac{V_{sv}}{f_y} = \frac{51.2 \times 10^3}{300} \qquad \text{(Take } f_y = 300 \text{ MPa)}$$

$$= 170.7 \text{ mm}^2$$

Therefore, use 6-R6 to give $A_{jV} = 169.6 \text{ mm}^2$.

Joint 4

Horizontal Joint Shear

Earthquake induced joint shear:

$$V_{E,jh} = \frac{16.6 + 29.1 - \frac{1}{2} \text{x } 82.4 \text{ x } 0.9 \text{ x } 0.8}{0.9 \text{ x } 1.6} = 11.1 \text{ kN}$$

Limited ductility design requires

$\phi V_n = V_{jh} = 2V_{E,jh}$ (Gravity induced joint shear is considered negligible in this instance)

$$\Rightarrow \qquad V_{jh} = 2 \text{ x } 11.1$$

$$= 22.2 \text{ kN}$$

Nominal shear stress in the joint

$$v_{jh} = \frac{V_{jh}}{b_c h_c} = \frac{22.2 \times 10^3}{190 \times 800} = 0.15 \text{ MPa} \qquad < v_g$$

From section 11.4.5.2, since beams remain elastic (i.e. no hinging)

$$V_{sh} = \frac{V_{jh}}{\phi} - V_{mh}$$

where $V_{mh} = 0.5V_{jh} = 11.1$ kN

but need not be taken less than $V_{mh} = v_m b_c h_c$

where $v_m = (C_1 + C_2) v_{bm}$

and $C_1 = 33 p_w \dfrac{f_y}{300}$

$p_w = 0.00297$ (for D12 @ 200 crs)

$\Rightarrow \quad C_1 = 0.098$

$C_2 = 1.0$ for simplicity

$$\Rightarrow \quad v_m = (0.098 + 1) \times 0.2\sqrt{f'_m}$$

$$= 0.22 \times \sqrt{16}$$

$$= 0.88 \text{ MPa}$$

Therefore $\quad V_{mh} = 0.88 \times 0.19 \times 0.8 \times 10^3 = 133$ kN

Hence

$$V_{sh} = \frac{22.2}{0.75} - 133 < \text{ZERO}$$

Therefore NO horizontal joint steel is required (i.e. $A_{jh} = 0$). The horizontal shear is carried by the horizontal component of the diagonal strut across the joint.

<u>Vertical Joint Shear</u>

$$V_{E,jv} = \frac{73.7 - \left(\frac{27.7 + 48.5}{2}\right) x\, 0.9 \times 1.6}{0.9 \times 0.8} = 26.2 \text{ kN}$$

$\phi V_n = V_{jv} = 2V_{E,jv}$ (Gravity induced joint shear is considered negligible in this instance)

$\Rightarrow \quad V_{jv} = 52.3$ kN

Nominal shear stress in the joint

$$v_{jv} = \frac{V_{jv}}{b_c h_b} = \frac{52.3 \times 10^3}{190 \times 1600} = 0.17 \text{ MPa} < v_g \qquad \text{Therefore OK}$$

$$V_{sv} = \frac{V_{jv}}{\phi} - V_{mv}$$

where $V_{mv} = 0$, see 11.4.6.2 of NZS 4230:2004.

Hence

$$V_{sv} = \frac{52.3}{0.75} - 0 = 69.7 \text{ kN}$$

Therefore, the total area of vertical joint shear reinforcement required:

$$A_{jv} = \frac{V_{sv}}{f_y} = \frac{69.7 \times 10^3}{300} \qquad (\text{Take } f_y = 300 \text{ MPa})$$

$$= 232.4 \text{ mm}^2$$

Therefore, use <u>4-R10</u> to give $A_{jv} = 314.2 \text{ mm}^2$.

Level 3 Joint Shear Design

A similar process to that above is required, but not tabulated herein, see Figure 18 for detailed.

3.8 Strut-and-tie Design of Wall with Opening

Figure 19(a) shows a three-storey concrete masonry wall with openings and loading conditions that resemble a design example of a reinforced concrete wall reported by Paulay and Priestley (1992). It is noted that designers may elect to consider a more sophisticated loading pattern, with horizontal loads apportioned within the wall based upon tributary areas, rather than the simple lumped horizontal forces shown in Figure 19(a).

The concrete masonry wall shown in Figure 19(a) is to be designed for the seismic lateral forces corresponding with an assumed ductility of μ = 2. The relatively small gravity loads are approximated by a number of forces at node points given in Figure 19(a), and the strut-and-tie model for the gravity loads is represented in Figure 19(b). Wall width should be 190 mm, and $f'_m = 12$ MPa. It is required to design the flexural and shear reinforcement for the wall.

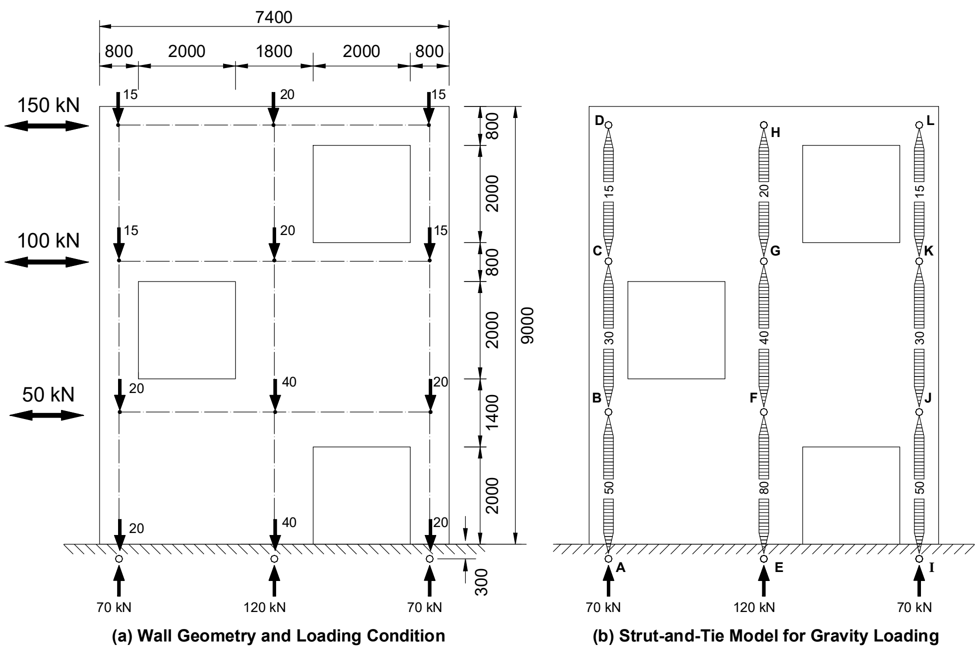


Figure 19: **Limited Ductile 3-storey Masonry Wall with Openings**

SOLUTION

Figures 20(a) and (b) show the strut-and-tie models for the squat wall with openings, corresponding to the seismic lateral forces being considered. For the purpose of limited ductile design, particular tension chords should be chosen to ensure yielding can best be accommodated. For example, members I-J and E-F in Figure 20(a) represent a good choice for this purpose.

Corresponding forces in other members should be determined and hence reinforcement provided so as to ensure that no yielding in other ties can occur. As these members carry only tension, yielding with cyclic displacements may lead to unacceptable cumulative elongations. Such elongations would impose significant relative secondary displacement on the small piers adjacent to openings, particularly those at I-J and A-B. The resulting bending moment and shear forces, although secondary, may eventually reduce the capacity of these vital struts.

In order to ensure that plastic hinges form inside the 1st storey vertical members, the quantity of reinforcement in the 2nd and 3rd storey vertical members should be sufficient to ensure that yielding does not occur in these members. Consequently, a simplified procedure is adopted in this example to design the vertical tie members above 1st storey for 50% more tension force than design levels.

From the given lateral forces the total overturning moment at 300 mm below the wall base is:

$$M^* = 150 \times (8.6 + 0.3) + 100 \times (5.8 + 0.3) + 50 \times (2.7 + 0.3)$$

$$= 2095 \text{ kNm}$$

Whilst the use of strut-and-tie analysis is specifically endorsed in section 7.4.8.1 of NZS 4230:2004, no advice is given in section 3.4.7 for an appropriate ϕ value to be used in conjunction with the analysis. In section 2.3.2.2(h) of NZS 3101:2006 a value of $\phi = 0.75$ is prescribed. This corresponds to the ϕ factor used for shear and torsion, which is consistent with the strut-and-tie procedure. Consequently, $\phi = 0.75$ is adopted here for use in strut-and-tie analysis of concrete masonry structures.

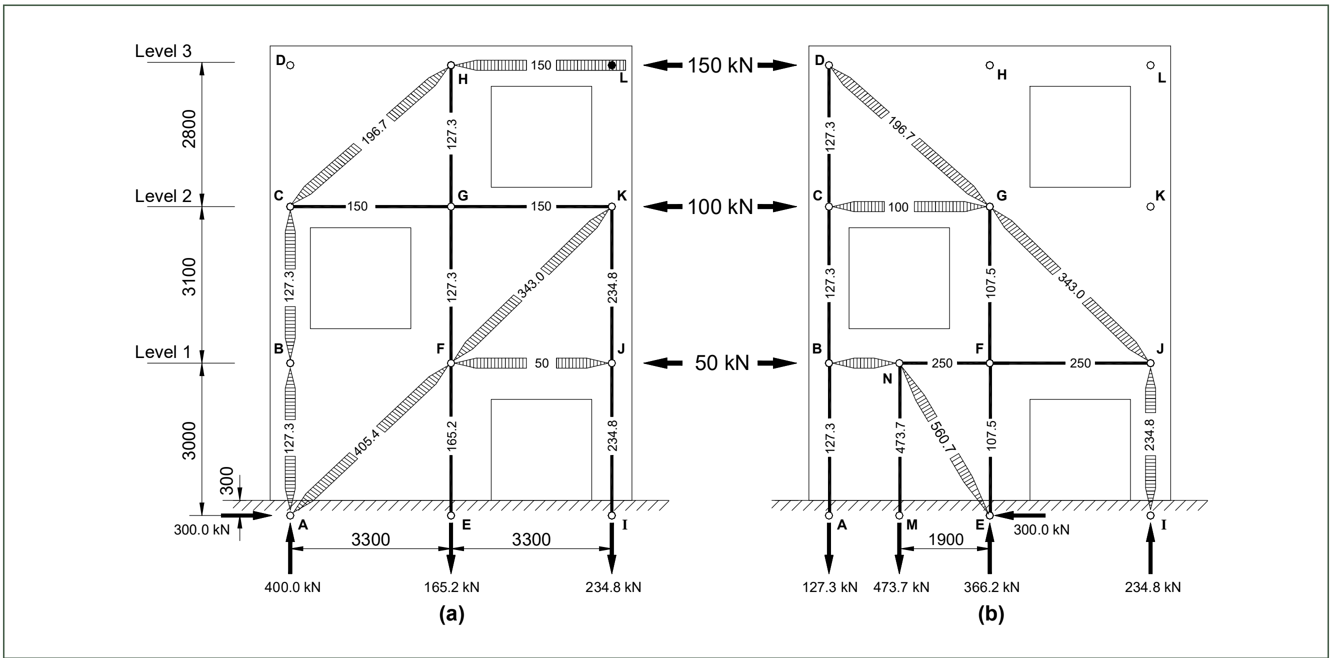


Figure 20: **Strut-and-Tie Models for Masonry Wall (seismic loading only)**

Design of Tension Reinforcement in Vertical Members

The area of tension reinforcement required in vertical ties, after considering the effect of axial loads, can be evaluated as follows:

$$\phi\left(A_{si}f_y + N_n\right) = T_i$$

$$\phi\left(A_{si}f_y + \frac{N_i^*}{\phi}\right) = T_i$$

Therefore

$$\phi A_{si}f_y = T_i - N_i^* \quad ^{(8)}$$

Figure 21 shows the strut-and-tie model for the squat wall when both seismic and gravity loads are considered.

[8] Paulay and Priestley (1992) adopted the procedure of $\phi A_{si}f_y = T_i - \phi N_i^*$, as this would result in a more conservative design.

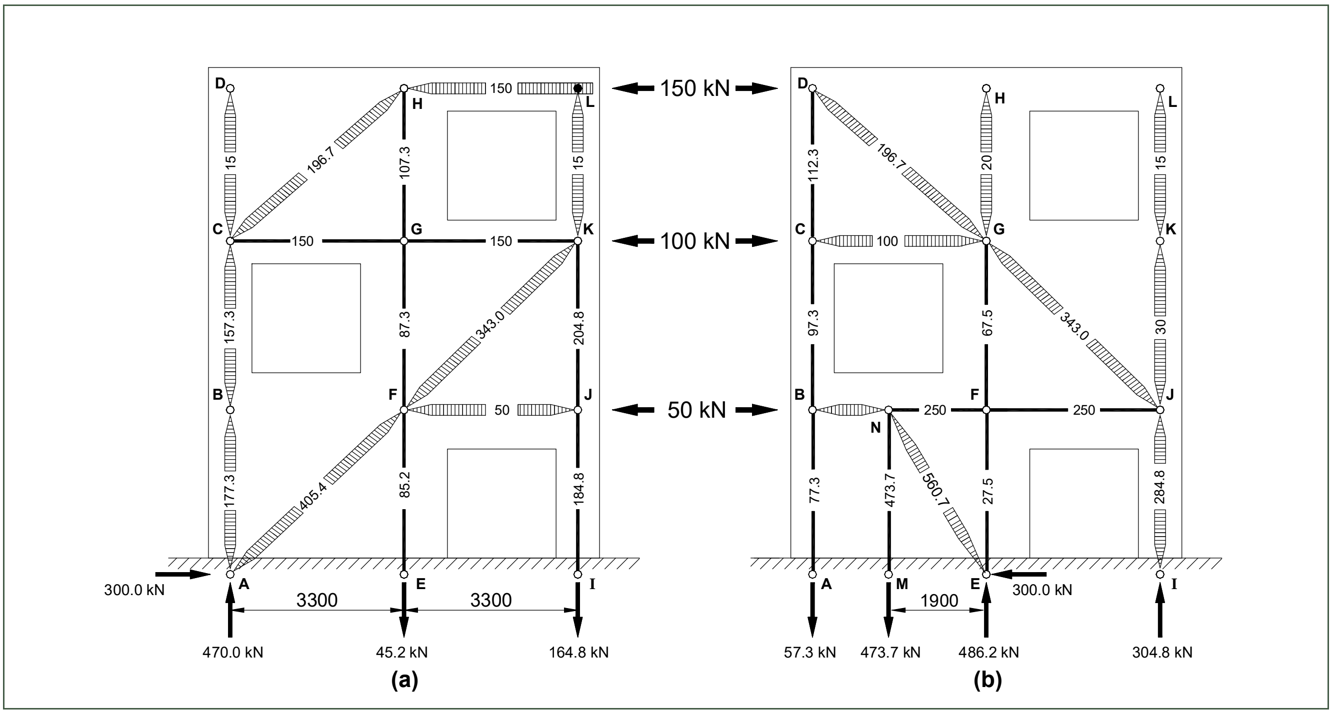


Figure 21: **Strut-and-Tie Models for Masonry Wall (Seismic and Gravity Loads)**

1st Storey Vertical Members

Consider earthquake $\vec{V}_E$ as in Figure 21(a)

Tie I-J $\quad \phi A_{IJ} f_y = 184.8$ kN

Therefore $\quad \phi A_{IJ} = \dfrac{184.8 \times 10^3}{0.75 \times 300}$ $\quad$ (taking $f_y = 300$ MPa)

$= 821.3$ mm^2

Try 4-D16 $\quad A_s = 804.2$ mm^2 $\quad$ (about 2% shortfall)

Tie E-F $\quad \phi A_{EF} f_y = 85.2$ kN

Therefore $\quad A_{EF} = \dfrac{85.2 \times 10^3}{0.75 \times 300}$

$= 378.7$ mm^2

Clause 7.4.5.1 of the standard requires minimum longitudinal reinforcement of D12 @ 400 crs within the potential plastic hinge zone. Consequently, adopt 5-D12 for Member E-F to give $A_s = 565.5$ mm^2.

Check moment capacity at wall base:

Tension forces provided:

$T_{IJ} = 804.2 \times 300 = 241.3$ kN

$T_{EF} = 565.5 \times 300 = 169.6$ kN

Therefore, total compression force at Node A, including gravity load:

$$C_m = T_{IJ} + T_{EF} + N_n$$

$$= 241.3 + 169.6 + \frac{260}{0.75}$$

$$= 757.6 \text{ kN}$$

Theoretical depth of neutral axis:

$$c = \frac{C_m}{0.85 \times 0.85 \times f'_m \times 190}$$

$$= \frac{757.6 \times 10^3}{0.85 \times 0.85 \times 12 \times 190}$$

$= 459.9$ mm $\approx 0.100L_w$ where $L_w = 800 + 2000 + 1800 = 4600$ mm

$< 0.2L_w$ (see clause 7.4.6.1 of NZS 4230:2004)

Moment capacity about the centre of the structure:

$M_n = (T_{IJ} + C_m) \times 3.3 = (241.3 + 757.6) \times 3.3$

$= 3296.4$ kNm

Therefore $\phi M_n = 0.75 \times 3395.7$

$= 2472.3$ kNm > M*

Consider earthquake $\vec{V}_E$ as in Figure 21(b)

<u>Tie A-B</u> $\phi A_{AB} f_y = 77.3$ kN

Therefore $A_{AB} f_y = \dfrac{77.3 \times 10^3}{0.75 \times 300}$ (taking $f_y = 300$ MPa)

$= 341.6 \text{ mm}^2$

Try <u>4-D12</u> $A_s = 452.4 \text{ mm}^2$

<u>Tie M-N</u> $\phi A_{MN} f_y = 473.7$ kN

Therefore $A_{MN} = \dfrac{473.7 \times 10^3}{0.75 \times 300}$ (taking $f_y = 300$ MPa)

$= 2105.3 \text{ mm}^2$

Try <u>8-D16 and 2-D20</u> $A_s = 2236.8 \text{ mm}^2$ (Note that D20 is the maximum bar size allowed for 190 mm wide masonry wall)

<u>Tie E-F</u> Use <u>5-D12</u> because member force would be critical when earthquake force acting in $\overleftarrow{V}_E$ direction, i.e. $A_s = 565 \text{ mm}^2$. Refer to Figure 22 for details.

Check moment capacity at wall base:

Tension forces provided:

T_{AB} = 452.4 x 300 = 135.7 kN

T_{MN} = 2236.8 x 300 = 671.0 kN

T_{EF} = 565.5 x 300 = 169.6 kN

Therefore, total compression force at Node I, including gravity load:

$$C_m = T_{AB} + (T_{MN} - T_{MN}) + T_{EF} + N_n$$

$$= 135.7 + (671.0 - 671.0) + 169.6 + \frac{260}{0.75}$$

$$= 652.1 \text{ kN}$$

Note that in the above calculation, it is recognised that the vertical component of strut E-N matches the force in tie M-N.

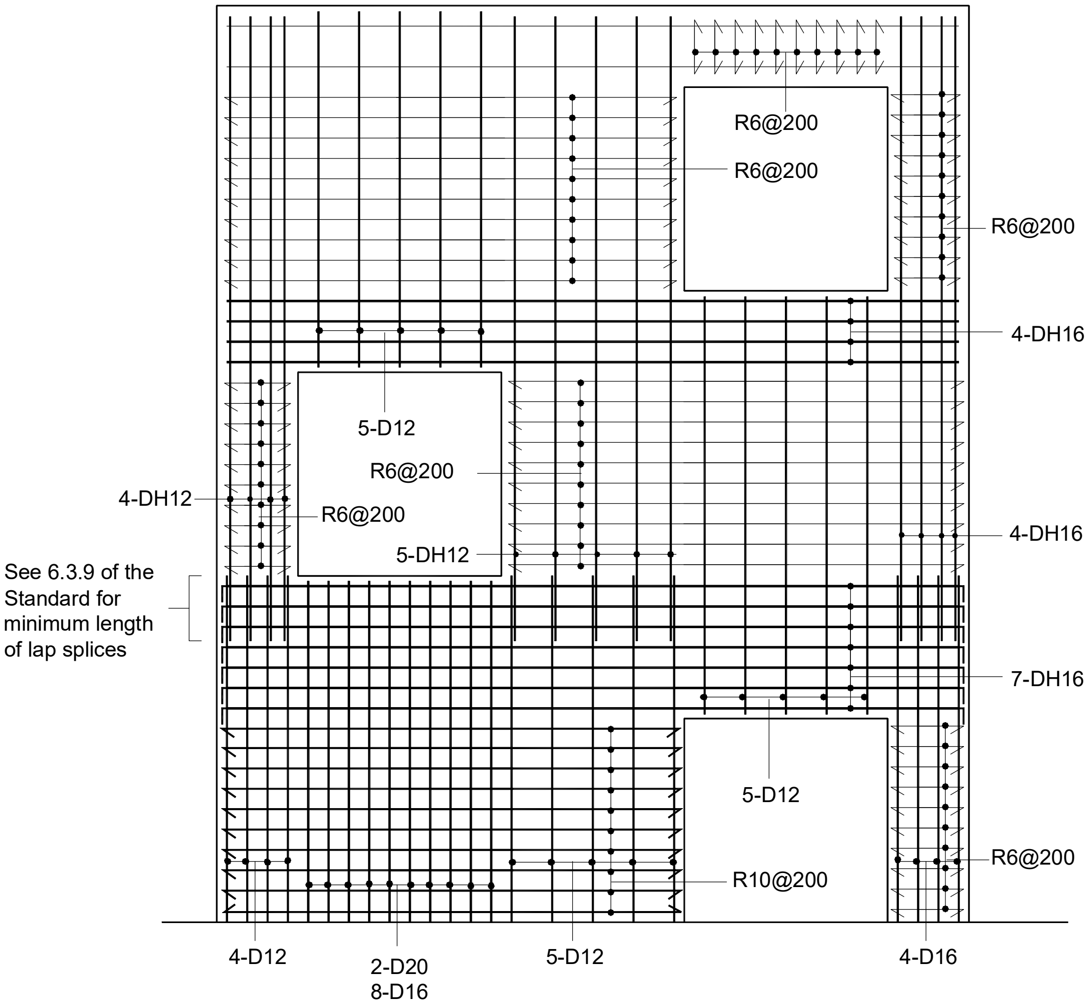


Figure 22: **Reinforcement for Design Example 3.8**

Theoretical depth of neutral axis:

$$c = \frac{C_m}{0.85 \times 0.85 \times f'_m \times 190}$$

$$= \frac{652.1 \times 10^3}{0.85 \times 0.85 \times 12 \times 190}$$

$$= 395.8 \text{ mm} \quad \approx 0.086 L_w$$

Moment capacity about the centre of the structure:

$$M_n = (T_{AB} + C_m) \times 3.3 + T_{MN} \times 1.9$$

$$= (135.7 + 652.1) \times 3.3 + 671.0 \times 1.9$$

$$= 3874.6 \text{ kNm}$$

Therefore $\phi M_n = 0.75 \times 3874.6$

$$= 2906 \text{ kNm} > M^*$$

2nd and 3rd Storey Vertical Members

To avoid the formation of plastic hinges, the amount of reinforcement in the 2nd and 3rd storey vertical members should be sufficient to ensure that yielding does not occur in these members. Hence, the 2nd and 3rd storey vertical members are intentionally designed for 50% higher tension forces than the design level tension forces.

Consider earthquake $\overleftarrow{V}_E$ as in Figure 21(a)

Tie J-L $\quad \phi A_{JK} f_y = 1.5 \times 204.8$

$$= 307.2 \text{ kN}$$

For tie J-L, the force in tie J-K is critical. Therefore, the design of tie K-L will match that of tie J-K.

Therefore $A_{JK} = \dfrac{307.2 \times 10^3}{0.75 \times 500}$ (take f_y = 500 MPa)

$$= 819.2 \text{ mm}^2$$

Try 4-DH16 $\quad A_s = 804.2 \text{ mm}^2$ (about 2% shortfall)

(Note that DH16 is the maximum bar size allowed in Table 1)

Tie F-H $\quad \phi A_{GH} f_y = 1.5 \times 107.3$

$$= 161.0 \text{ Kn}$$

For tie F-H, the force in tie G-H is critical. Therefore, the design of tie F-G will match that of tie G-H.

Therefore $A_{GH} = \dfrac{161.0 \times 10^3}{0.75 \times 500}$ (take f_y = 500 MPa)

$$= 429.3 \text{ mm}^2$$

Try 5-DH12 $\quad A_s = 565.5 \text{ mm}^2$

Consider earthquake $\vec{V}_E$ as in Figure 21(b)

Tie B-D $\quad \phi A_{CD} f_y = 1.5 \times 112.3$

$= 168.5$ kN

For tie B-D, the force in tie C-D is critical. Therefore, the design of tie B-C will match that of tie C-D.

Therefore $\quad A_{CD} = \dfrac{168.5 \times 10^3}{0.75 \times 500}$ (take f_y = 500 MPa)

$= 449.3$ mm^2

Try 4-DH12 $\quad A_s = 452.4$ mm^2

Tie F-H $\quad$ Use 5-DH12 because member force would be critical when earthquake force acting in $\overleftarrow{V}_E$ direction, i.e. A_s = 565.5 mm^2. Refer to Figure 22 for details.

Design of Tension Reinforcement in Horizontal Members

In section 3.7.3.3 of NZS 4230:2004, there are two equations given that permit a simplified capacity design approach to be used. However, in this example it has been necessary to place a significantly larger quantity of vertical reinforcement than required (i.e. member E-F), in order to satisfy spacing criteria.

This has resulted in a concern about relying upon these simplified expressions and instead a full capacity design is conducted below to establish the appropriate horizontal design forces.

To estimate the maximum tension force in horizontal ties, the flexural overstrength at wall base, M_o, needs to be calculated:

$M_o = 1.25 M_{n,provided}$

Consider earthquake $\overleftarrow{V}_E$ as in Figure 21(a)

$M_{n,provided} = 3296.4$ kNm

The overstrength value, $\phi_{o,w}$, is calculated as follows:

$$\phi_{o,w} = \frac{M_o}{M^*} = \frac{1.25 M_{n,provided}}{M^*}$$

$$= \frac{1.25 \times 3296.4}{2095}$$

$$= 1.97$$

Dynamic magnification factor:

For up to 6 storeys $\quad \omega_v = 0.9 + \dfrac{n}{10}$

$$= 0.9 + \frac{2}{10}$$

$$= 1.1$$

Hence, the design force for Member C-G-K is calculated as follow:

$T_{CK} = 1.1 \times 1.97 \times 150$

$= 325.1$ kN

Therefore

$\phi A_{ck} f_y = 325.1$ kN

$$A_{ck} = \frac{325.1 \times 10^3}{1.0 \times 500} \qquad \phi = 1.0 \text{ (see 3.4.7) and take } f_y = 500 \text{ MPa}$$

$= 650.2\ \text{mm}^2$

Try 4-DH16 $\qquad A_s = 804\ \text{mm}^2$

Consider earthquake $\vec{V}_E$ as in Figure 21(b)

$M_{n,provided} = 3874.6$ kNm

The overstrength value, $\phi_{o,w}$, is calculated as follow:

$$\phi_{o,w} = \frac{M_o}{M^*} = \frac{1.25 M_{n,provided}}{M^*}$$

$$= \frac{1.25 \times 3874.6}{2095}$$

$= 2.31 > \phi_{o,w} = 1.97$ when considering $\overleftarrow{V}_E$

Dynamic magnification factor:

For up to 6 storeys $\qquad \omega_V = 1.1$

Hence, the design force for Member N-F-J is calculated as follow:

$T_{NJ} = 1.1 \times 2.31 \times 250 = 635.3$ kN

Therefore

$\phi A_{NJ} f_y = 635.3$ kN

$$A_{NJ} = \frac{635.3 \times 10^3}{1.0 \times 500} \quad (\text{take } f_y = 500 \text{ MPa})$$

$= 1270.6\ \text{mm}^2$

Try 7-DH16 $\qquad A_s = 1407.4\ \text{mm}^2$

Design of Shear Reinforcement

It is assumed that shear forces are to be resisted by the bigger wall elements adjacent to openings, such that only these elements require design of shear reinforcement. For other part of the wall structure, it is only required to satisfy $p_{min} = 0.07\%$, i.e. use R6 @ 200 crs.

As V_G^* and V_{Qu}^* are typically negligible, therefore:

$$\phi V_n \geq \omega_v \phi_{o,w} V_E^* \qquad \text{where } \phi = 1.0 \text{ (3.4.7 of NZS 4230:2004)}$$

Shear Design, 1st Storey

$$V_E^* = 300 \text{ kN}$$

Therefore $$V_n = \frac{1.1 \times 2.31 \times 300}{1.0}$$

$$= 762.3 \text{ kN}$$

Check shear stress, b_w = 190 mm, d = 0.8 x 4600 = 3680 mm

$$v_n = \frac{762.3 \times 10^3}{190 \times 3680} = 1.09 \text{ MPa} < v_g = 1.50 \text{ MPa for } f'_m = 12 \text{ MPa}$$

From Section 10.3 of NZS 4230:2004:

$$v_n = v_m + v_p + v_s$$

Shear stress carried by $v_m = (C_1 + C_2)v_{bm}$

where $$C_1 = 33p_w \frac{f_y}{300}$$

note that $$p_w = \frac{9\text{bars} \times D12 + 8\text{bars} \times D16 + 2\text{bars} \times D20}{b_w d}$$

$$= \frac{3254.7}{190 \times 0.8 \times 4600}$$

$$= 0.0046$$

Therefore $C_1 = 0.15$

and $$C_2 = 0.42 \times [4 - 1.75 \times (3400/4600)]$$

$$= 1.14$$

$$\Rightarrow \quad v_m = (0.15 + 1.14) \times v_{bm}$$

$$= 1.29 \times 0.50 \qquad \text{note that } v_{bm} = 0.50 \text{ MPa for } \mu = 2$$

$$= 0.67 \text{ MPa}$$

Therefore the shear reinforcement required:

$$v_s = v_n - v_m - v_p \qquad \text{(take } v_p = 0 \text{ for simplicity)}$$

$$\Rightarrow \quad v_s = 1.09 - 0.67 - 0$$

$$= 0.42 \text{ MPa}$$

$$v_s = C_3 \frac{A_v f_y}{b_w s}$$ note that $C_3 = 0.8$ for masonry walls

$$\Rightarrow \quad 0.42 = 0.8 \times \frac{A_v \times 300}{190 \times 200}$$ (try $f_y = 300$ MPa and $s = 200$ mm)

$$A_v = 66.5 \text{ mm}^2$$

Therefore, use R10 @ 200 crs (78.5 mm^2) and $p = \frac{78.5}{190 \times 200} = 0.2\%$.

Shear Design, 2nd Storey

$$V_E^* = 250 \text{ kN}$$

Therefore $V_n = 1.1 \times 2.31 \times 250$

$= 635.3$ kN

Check shear stress, $b_w = 190$ mm, $d = 0.8 \times 4600 = 3680$ mm

$$v_n = \frac{635.3 \times 10^3}{190 \times 3680} = 0.91 \text{ MPa} < v_g = 1.50 \text{ MPa}$$

Shear stress carried by $v_m = (C_1 + C_2)v_{bm}$

where $C_1 = 33 p_w \frac{f_y}{300}$

$$= 33 \times \frac{5\,\text{bars} \times DH12 + 4\,\text{bars} \times DH16}{b_w d} \times \frac{500}{300} + 33 \times \frac{5\,\text{bars} \times D12}{b_w d} \times \frac{300}{300}$$

$= 0.10 + 0.03$

$= 0.13$

and $C_2 = 0.42 \times \left[4 - 1.75 \times \left(\frac{4200}{4600}\right)\right]$

$= 1.01$

$$\Rightarrow \quad v_m = (0.13 + 1.01) \times v_{bm}$$

$= 1.14 \times v_{bm}$ ($v_{bm} = 0.70$ MPa since outside plastic hinge region)

$= 1.14 \times 0.70$

$= 0.80$ MPa

Therefore the shear reinforcement required:

$v_s = v_n - v_m - v_p$ (take $v_p = 0$ for simplicity)

$\Rightarrow \quad v_s = 0.91 – 0.80 - 0$

$= 0.11$ MPa

$$v_s = C_3 \frac{A_v f_y}{b_w s}$$ where C_3 = 0.8 for masonry walls

$$\Rightarrow \quad 0.11 = 0.8 \times \frac{A_v \times 300}{190 \times 200}$$ (try f_y = 300 MPa and s = 200 mm)

A_v = 17.5 mm^2

Therefore, use R6 @ 200 crs (28.3 mm^2) and $p = \frac{28.3}{190 \times 200} = 0.07\%$. Note that p = 0.07% is the minimum reinforcement area required by 7.3.4.3 of NZS 4230:2004.

Shear Design, 3rd Storey

$V_E^* = 150$ kN

therefore V_n = 1.1 x 2.31 x 150

= 381.2 kN

Check shear stress, b_w = 190 mm, d = 0.8 x 4600 = 3680 mm

$$v_n = \frac{381.2 \times 10^3}{190 \times 3680} = 0.54 \text{ MPa} \quad < v_g$$

Shear stress carried by $v_m = (C_1 + C_2)v_{bm}$

$$\text{where} \quad C_1 = 33p_w \frac{f_y}{300} = 33 \times \frac{9 \text{ bars} \times \text{DH12}}{b_w d} \times \frac{500}{300} + 33 \times \frac{5 \text{ bars} \times \text{D12}}{b_w d} \times \frac{300}{300}$$

= 0.08 + 0.03

= 0.11

$$\text{and} \quad C_2 = 0.42 \times \left[4 - 1.75 \times \left(\frac{3600}{4600}\right)\right]$$

= 1.10

$$\Rightarrow \quad v_m = (0.11 + 1.10) \times v_{bm}$$ (v_{bm} = 0.70 MPa outside plastic hinge region)

= 1.21 x 0.70

= 0.85 MPa > v_n

Because $v_m > v_n$, the shear reinforcement needed in the 3rd storey pier is governed by the minimum reinforcement area required by clause 7.3.4.3, i.e. 0.07% of the gross cross-sectional area. Therefore, shear reinforcement in the 3rd storey pier can be reduced to R6 @ 200 crs.

4.0 Prestressed Masonry

A new addition to NZS 4230 is the inclusion of Appendix A related to the design of prestressed concrete masonry. As noted in the commentary, this section is primarily for application to wall components, but its use for other component types is not precluded.

Design information for unbonded post-tensioning is presented below.

This form of prestressing is recommended as it minimises structural damage and results in structures that exhibit little or no permanent horizontal deformation following earthquake excitation. It is noted that the provided information is more comprehensive than will be required for most conventional designs, and is included as background for the following example.

For additional information refer to research conducted by Laursen and Ingham at the University of Auckland[9,10].

4.1 Limit States

The flexural design procedure presented here is based on Limit State Design, as outlined by AS/NZS 1170.0:2002, which identifies two limit states, namely the Serviceability limit state and the Ultimate limit state.

The flexural serviceability limit state for prestressed masonry is concerned with flexural strength, stiffness and deflections. The following flexural states represent the limiting flexural moments for a wall to remain elastic for uncracked and cracked sections.

- *First Cracking:* This limit state corresponds to the state when the extreme fibre of the wall decompresses (the tensile strength of concrete masonry is disregarded)
- *Maximum Serviceability moment:* At this cracked section state, the compressive stress in the extreme compression fibre has reached its elastic limit set out by the standard as a stress limitation. Reinforcement and concrete masonry remain elastic in this state.

The flexural ultimate limit state for prestressed masonry is primarily concerned with flexural strength. Additionally for ductility purposes, overstrength, stiffness and deflections should be considered:

- *Nominal strength:* The nominal strength according to NZS 4230:2004 is per definition achieved when the concrete masonry fails in compression at the strain, ε_u, equals 0.003.
- *Overstrength*: This strength corresponds to the maximum moment strength developed by the wall, taking into account stress increase, yield and strain hardening of the prestressing tendons. At this stage, large deformations are expected and the maximum concrete masonry strain is likely to have surpassed 0.003. Past the maximum wall strength, the wall resistance gradually degrades until failure.

All of the above limit states generally need to be evaluated both immediately after prestress transfer and after long term losses.

4.2 Flexural Response of Cantilever Walls

This section considers the flexural design of prestressed concrete masonry cantilever walls with unbonded prestressing tendons, where the lateral force is assumed to be acting at the top of the wall or at some effective height h_e, refer to Figure 23.

For other structural shapes and loading configurations, the formulae should be modified accordingly. Note that the term "tendon" in the following sections refer to both prestressing strands and bars.

The applied forces and loads represented by the symbols V, M, N and P used in the following equations are all factored loads calculated according to the applicable limit state as defined in the AS/NZS 1170.0:2002. The axial force N is due to dead and live loads, P is the prestressing force (initial force after anchor lock-off or force after all long term losses), and V is the applied lateral force due to lateral actions. It is assumed that moment M only arises from lateral forces V, i.e. permanent loads and prestressing do not introduce permanent moment in the wall.

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9 Laursen, P. T. (2002) “Seismic Analysis and Performance of Post-Tensioned Concrete Masonry Walls”, Doctoral Thesis, University of Auckland, 281pp.

10 Laursen, P. T., and Ingham, J. M. (1999) “Design of Prestressed Concrete Masonry Walls”, Journal of the Structural Engineering Society of New Zealand, 12, 2, 21-39.

Figure 23 shows the various definitions of wall dimensions and forces.

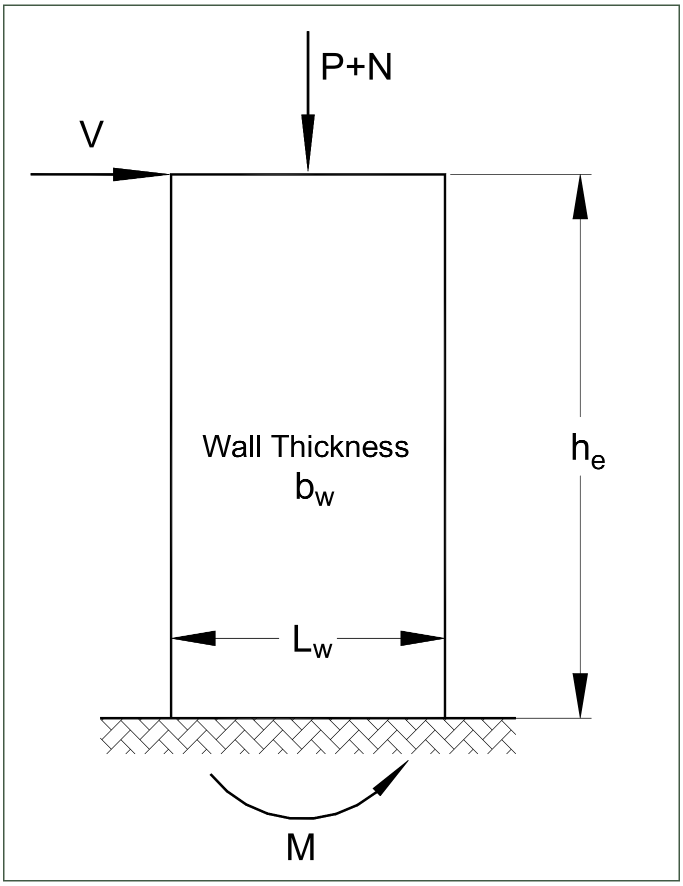

Figure 23: **Definition of Wall**

It is assumed for the flexural calculations that plane sections remain plane, i.e. a linear strain distribution across the wall length. This assumption enables analytical calculation of strength, stiffness and displacement, and implies distributed cracking up the wall height.

From laboratory wall tests it was observed that PCM wall flexural response was primarily due to rocking where a crack opened at the base, and that distributed flexural cracking did not develop[9]. This type of rocking behaviour is a feature of prestressing with unbonded tendons.

Despite this discrepancy between theory and observation, it appears that the assumption of plane section response and distributed wall cracks results in sufficiently accurate design rules.

4.2.1 First Cracking

The moment corresponding to first cracking M_{cr} may be evaluated by Eqn. 18. The formula is based on the flexural state at which one wall end decompresses and the other end compresses to a stress of twice the average masonry stress f_m:

$$M_{cr} = \frac{f_m b_w L_w^2}{6} = \frac{(P+N)L_w}{6}, \quad f_m = \frac{P+N}{L_w b_w} \qquad [18]$$

$$V_{cr} = \frac{M_{cr}}{h_e} \qquad [19]$$

where b_w is the wall thickness, L_w is the wall length, V_{cr} is the applied force at the top of the wall corresponding to the 1st cracking moment M_{cr} and h_e is the effective wall height.

The deflection of the top of the wall d_{cr} at V_{cr} should be based on the concrete masonry wall elastic properties and consists of a component due to shear deformation $d_{cr,sh}$ and a component due to flexure $d_{cr,fl}$:

$$d_{cr} = d_{cr,fl} + d_{cr,sh} = \frac{2}{3}\frac{h_e^2(P+N)}{E_m L_w^2 b_w} + \frac{2}{5}\frac{(1+v)(P+N)}{E_m b_w} \qquad [20]$$

where Poisson's ratio may be taken as $\nu = 0.2$. It should be noted that the shear deformation component $d_{cr,sh}$ can be of significant magnitude for squat walls under serviceability loads, whereas for the ultimate limit state it becomes increasingly insignificant.

The curvature at 1st cracking can be calculated as follows:

$$\phi_{cr} = \frac{2(P+N)}{E_m L_w^2 b_w} \qquad [21]$$

4.2.2 Maximum Serviceability Moment

Typically at the serviceability limit state, the applied lateral force has surpassed that necessary to initiate cracking at the base of the wall. The serviceability moment is limited by M_e which occurs when the stress in the extreme compression fibre at the base of the wall has reached kf'_m, as shown in Figure 24. For prestressed concrete masonry, k (symbol adopted in this manual) is set out in Table A.1 of NZS 4230:2004, which is reproduced from Table 16.1 of NZS 3101:1995, with k typically ranging between 0.4 and 0.6, dependent on load category. In NZS 3101:2006 this criteria was modified, where clause 19.3.3.5.1(b) prescribes an upper limit of k = 0.6.

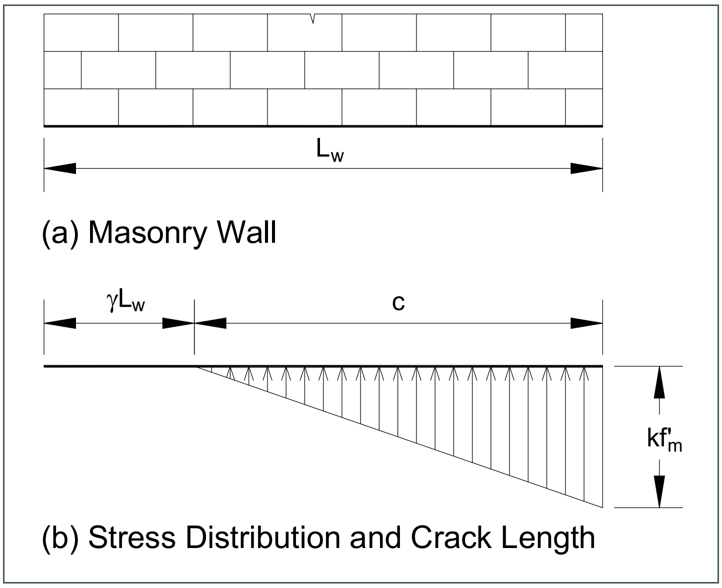

Figure 24: **Maximum Serviceability Moment**

It is noted that Eqn. 22 must be satisfied before use of the equations relating to the maximum serviceability moment can be applied, though this requirement is generally fulfilled.

$$kf'_m > 2f_m \qquad [22]$$

The masonry is assumed to remain linearly elastic, hence the masonry strain ε_{ms} corresponding to kf'_m can be found from:

$$\varepsilon_{ms} = \frac{kf'_m}{E_m} \qquad [23]$$

By adopting k = 0.55 from load category IV (infrequent transient loads), it may be shown that the maximum serviceability moment can be calculated as[9]:

$$M_e = \frac{f_m}{6}\left(3 - \frac{4f_m}{kf'_m}\right)L_w^2 b_w = f_m\left(0.5 - 1.2\frac{f_m}{f'_m}\right)L_w^2 b_w = V_e h_e \qquad [24]$$

where V_e is the corresponding lateral force. The corresponding curvature at the wall base, ϕ_e, is:

$$\phi_e = \frac{(kf'_m)^2}{2f_m E_m L_w} = 0.15\frac{{f'_m}^2}{f_m E_m L_w} \qquad [25]$$

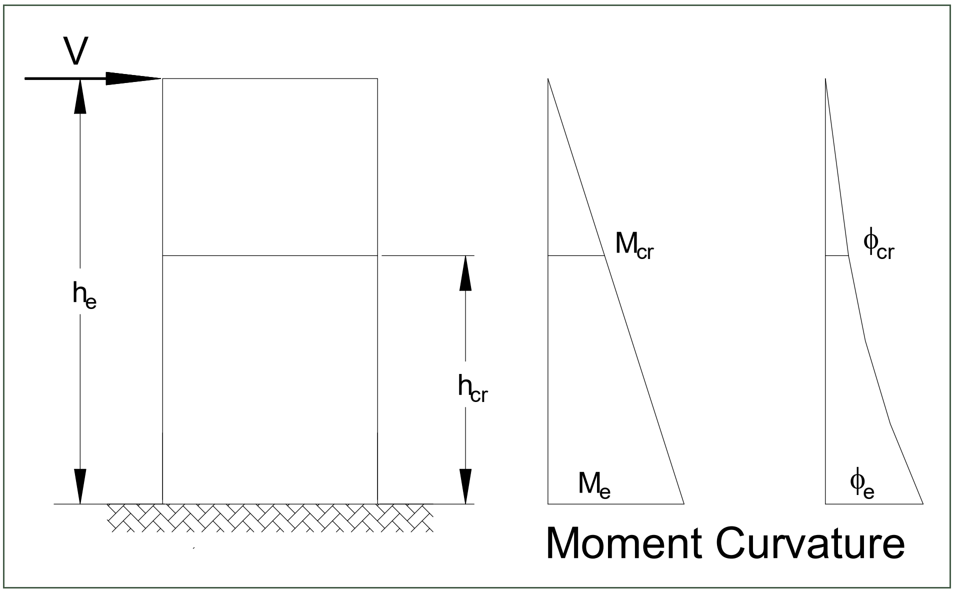

Figure 25: **Curvature Distribution at Maximum Serviceability Moment**

Figure 25 shows the variation of moment and curvature along the height of the wall at the maximum serviceability moment, assuming plane section response. The curvature varies from ϕ_e at the base to ϕ_{cr} at the height, h_{cr}, at which the 1st cracking occurs. Between the heights h_{cr} and h_e the curvature varies linearly between ϕ_{cr} and zero. It can be shown that the curvature varies linearly with the non-dimensional crack length, γ, as defined in Figure 24. Eqn. 26 defines the non-dimensional crack length at the base of the wall at the maximum serviceability moment, again assuming k = 0.55:

$$\gamma_e = 1 - \frac{2f_m}{kf'_m} = 1 - 3.6\frac{f_m}{f'_m} \qquad [26]$$

Eqn. 27 defines the resulting cracked wall height.

$$h_{cr} = h_e\left(\frac{M_e - M_{cr}}{M_e}\right) \qquad [27]$$

The total displacement d_e of the top of the wall can then be calculated by integration along the wall height with the following result:

$$d_e = d_{e,fl} + d_{e,sh} \qquad [28]$$

$$d_{e,fl} = \frac{2f_m h_{cr}}{E_m L_w \gamma_e}\left[(h_e - h_{cr})\left(\frac{\gamma_e}{1-\gamma_e}\right) + \frac{h_{cr}}{\gamma_e}\left(\frac{\gamma_e}{1-\gamma_e} + \ln\left|1-\gamma_e\right|\right)\right] + \frac{\phi_{cr}}{3}(h_e - h_{cr})^2 \qquad [29]$$

which may be approximated assuming k = 0.55 as:

$$d_{e,fl} = \left(0.30 - 0.029\frac{f_m}{f'_m}\right)\frac{f'_m h_e^2}{E_m L_w}$$

and

$$d_{e,sh} = \frac{12(1+\nu)h_e}{5E_m L_w b_w} V_e \qquad [30]$$

In Eqns. 29 and 30, $d_{e,fl}$ and $d_{e,sh}$ represent the flexural and shear deformations, respectively. At this flexural state, it is assumed that the relatively small deformations of the wall do not result in significant tendon force increase or migration of the tendon force eccentricity.

4.2.3 Nominal Strength

At the ultimate limit state, an equivalent rectangular stress block is assumed with a stress of 0.85 f'_m (α= 0.85) and an extreme fibre strain of ε_u = 0.003, corresponding to the definition of nominal strength in NZS 4230:2004 for unconfined concrete masonry. For confined masonry NZS 4230:2004 recommends using an average stress of 0.9Kf'_m (α = 0.9K with f'_m based on unconfined prism strength) and ε_u = 0.008. The corresponding moment M_n and lateral force V_f can be evaluated by simple equilibrium, as shown in Figure 26, with the following equation:

$$M_n = (P+\Delta P)\left(\frac{L_w}{2} + e_t - \frac{a}{2}\right) + N\left(\frac{L_w}{2} - \frac{a}{2}\right) = V_f h_e \qquad [31]$$

where a is the length of the equivalent ultimate compression block given by:

$$a = \frac{P+\Delta P + N}{\alpha f'_m b_w} \qquad [32]$$

In these equations, ΔP accounts for the increase in tendon force that arises from the flexural deformation and e_t accounts for the associated tendon force eccentricity. Both ΔP and e_t may initially be assumed to equal zero for simple use. This approach is similar to the method used in NZS 3101:2006. A better estimate of the nominal strength may be obtained from Eqn 31, when taking into account the tendon force increase ΔP and the associated tendon force eccentricity e_t.

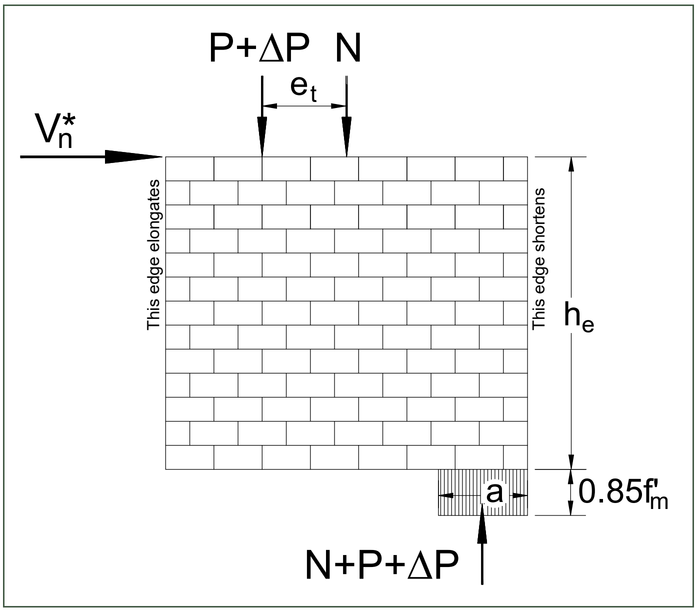

Figure 26: **Wall Equilibrium at Nominal Flexural Strength**

It is observed from Figure 26 that there is moment reversal near the top of the wall due to e_t which results in reversal of curvature. This effect is not taken into account below when calculating wall deformations because it has a negligible effect on the predicted wall behaviour at nominal flexural strength.

The total lateral displacement, d_n, is given by the sum of the flexural displacement, d_{nfl}, and shear displacement, d_{nsh}, corresponding to M_n, and may be evaluated using Eqn. 33:

$$d_n = d_{nfl} + d_{nsh} \text{ where} \qquad [33]$$

Unconfined: $$d_{nfl} = (2.30\xi_n^2 - 1.38\xi_n + 0.856)\frac{f'_m h_e^2}{E_m L_w} \qquad [34]$$

Confined: $$d_{nfl} = (7.63\xi_n^2 - 5.40\xi_n + 1.69)\frac{f'_m h_e^2}{E_m L_w} \qquad [35]$$

$$d_{n,sh} = \frac{12(1+v)h_e}{5E_m L_w b_w} V_f \qquad [36]$$

$$\xi_n = \frac{P + \Delta P + N}{f'_m L_w b_w} \qquad [37]$$

Eqns. 34 and 35 were developed using numerical integration and curve fitting, and are thus of an approximate nature, and are valid for axial load ratios, ξ_n, of 0.05 to 0.25. The extreme fibre strain was taken as ε_u = 0.003 for unconfined concrete masonry and 0.008 for confined concrete masonry. Detailed information on derivation of these equations may be found in Laursen[9].

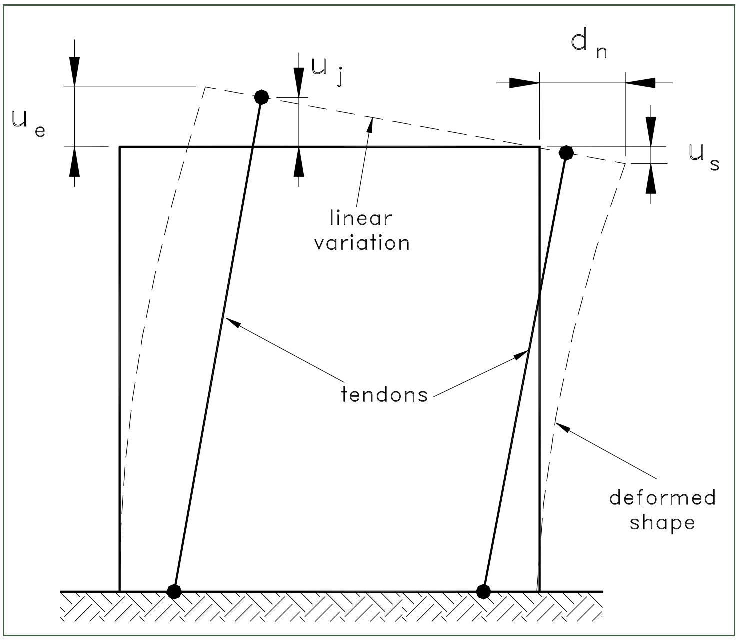

Figure 27: **Wall Deformation at Nominal Flexural Strength**

The total tendon force increase ΔP at ε_u of 0.003 (or 0.008) is difficult to evaluate for pre-stressed walls with unbonded tendons because the tendon stress increase depends on the deformation of the entire wall between points of anchorage. However, the force increase (or decrease) in each tendon in the wall cross section may be evaluated based on the estimated wall end elongation, u_e, (tension end) and shortening (compression end), u_s, assuming a linear variation of vertical deformation across the wall top as shown in Figure 27. The following equations were established for unconfined and confined concrete masonry[9]:

Unconfined: $$u_e = (4.01\xi_n^2 - 2.37\xi_n + 0.835)\frac{f'_m h_e}{E_m} \quad [38]$$

$$u_s = (3.36\xi_n^2 - 2.12\xi_n - 0.073)\frac{f'_m h_e}{E_m}$$

Confined: $$u_e = (22.5\xi_n^2 - 10.4\xi_n + 1.83)\frac{f'_m h_e}{E_m} \quad [39]$$

$$u_s = (1.67\xi_n^2 - 1.64\xi_n - 0.142)\frac{f'_m h_e}{E_m}$$

In these equations, elongation is positive and shortening is negative. It is clear that the tendon force increase due to vertical deformation will increase the axial load ratio. Iteration using Eqns. 38 or 39 is therefore needed to find $\Delta P = \Sigma \Delta P_j$ such that the calculated axial force ratio at nominal flexural strength, ξ_n, injected in the equations on the right hand side in fact corresponds to the calculated tendon force increase on the left hand side of the equations.

The effective total tendon force eccentricity relative to the wall centre line can be evaluated by:

$$e_t = \frac{\Sigma(P_j + \Delta P_j)y_j}{\Sigma(P_j + \Delta P_j)} \qquad \text{where} \quad \Delta P_j = \frac{u_j}{L_j} A_{psj} E_{ps} \quad [40]$$

P_j and ΔP_j are the initial tendon force and tendon force increase of the j'th tendon, and y_j is the horizontal location of the j'th tendon with respect to the wall centre line taken as positive towards the tension end of the wall. The tendon vertical extension, u_j, is defined in Figure 27 and L_j is the tendon length (approximately the height of the wall h_w, which is significantly longer than h_e for multi-storey building). A_{psj} is the area of the j'th tendon and E_{ps} is the elastic modulus of the prestressing steel. It must be ensured that P_j+ΔP_j does not exceed the tendon yield strength.

Iteration process for calculation of M_n and d_n:

1. Calculate ξ_n using Eqn. 37 using $\Delta P = 0$.
2. Calculate u_e and u_s using Eqns. 38 or 39.
3. Calculate $\Delta P = \Sigma \Delta P_j$ using Eqn. 40.
4. Calculate ξ_n using Eqn. 37 using ΔP from (3).
5. Repeat steps (2) to (4) until convergence of ξ_n.
6. Calculate M_n using Eqn. 31 and d_n using Eqn. 33.

The masonry design codes BS 5628:2005[11] and AS 3700:2001[12] present formulae for calculating the tendon stress increase, but are not applicable for in-plane wall bending because they were developed for out-of-plane response. NZS 3101:2006 recognises that the design tendon force for unbonded tendons will exceed the tendon force following losses. Using the notation presented here, the increase in tendon force is given by:

[11] BS 5628:2005, Part 2: "Code of Practice for use of Masonry. Structural Use of Reinforced and Prestressed Masonry", British Standards Institution, London.

[12] AS 3700-2001, "Masonry Structures", Standard Association of Australia, Homebush, NSW, Australia.

$$\Delta P = A_{ps}\left(70\ \text{MPa} + \frac{f'_m b_w L_w}{100 A_{ps}}\right) \qquad [41]$$

$$f_{se} = \frac{P}{A_{ps}}, \quad f_{ps} \le f_{py} \quad \text{and} \quad f_{ps} \le f_{se} + 400\ \text{MPa} \qquad [42]$$

where A_{ps} is the total prestressing tendon area, f_{ps} is the resulting average tendon stress corresponding to $P+\Delta P$, f_{py} is the tendon yield stress, and f_{se} is the tendon stress corresponding to P. This equation seems to provide reasonable results but has not been validated for all wall configurations. It would be prudent to assume a total tendon force increase of ½ - ¾ times the result calculated by Eqn. 41 when the prestressing tendons are approximately evenly distributed along the length of the wall. Eqn. 43 evaluates the resulting tendon eccentricity, e_t, due to the total tendon force increase, assuming that the tendon force increase, ΔP, acts at an eccentricity of $L_w/6$ and that the tendons are evenly distributed across the wall.

$$e_t = \frac{L_w \Delta P}{6(P + \Delta P)} \qquad [43]$$

Having calculated ΔP and e_t, the nominal flexural strength, M_n, and corresponding displacement, d_n, can then be evaluated using Eqns. 31 and 33.

4.2.4 Yield Strength

Contrary to reinforced concrete walls, the yield strength for unbonded prestressed walls is typically found at displacements beyond the displacement at nominal flexural strength. Structural testing has consistently shown that the behaviour of unbonded prestressed walls loaded beyond the nominal strength is dominated by rocking as illustrated in Figure 28. Even for walls without specially placed confinement plates, experimental observations consistently demonstrate that the wall is able to support compression strains far beyond 0.003. In Figure 28, the wall has rocked over by a displacement, d_{ty}, corresponding to a rotation θ. At this state, it is assumed that the extreme tendon at the tension side of the wall yields, resulting in a tendon strain increase of:

$$\Delta\varepsilon_{py} = \frac{(f_{py} - f_{ps})}{E_{ps}} \qquad [44]$$

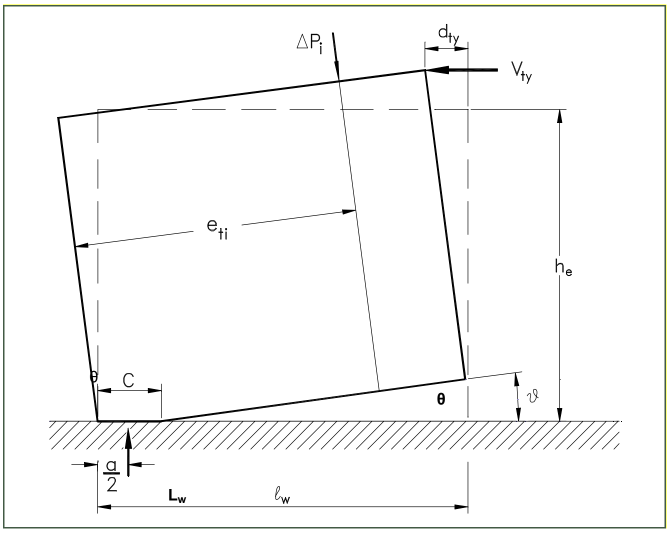

Figure 28: **Rocking Response**

where E_{ps} the modulus of elasticity for the tendon steel, and f_{ps} is taken as the tendon stress in the extreme tendon at nominal strength. If a wall is displaced laterally beyond d_{ty}, some reduction of prestress should be anticipated upon unloading. Notably, this does not mean that wall strength is permanently reduced because the tendons can be fully activated by subsequent loading excursions. The wall rotation θ can be related to the wall displacement increase at first tendon yield d_{ty} and the tendon strain increase $\Delta\varepsilon_{py}$ in the following way:

$$\theta = \frac{\Delta\varepsilon_{py} h_e}{e_{te} - c} \quad \Rightarrow \quad d_{ty} = \theta h_e = \frac{\Delta\varepsilon_{py} h_e^2}{e_{te} - c} = \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{te} - c} \qquad [45]$$

where $a = \beta c$, and it is assumed $\beta = 0.85$ for unconfined masonry and $\beta = 0.96$ for confined masonry. In this equation, e_{te} is the eccentricity of the extreme tendon at the wall tension side with respect to the compressive end of the wall. The length of the compression zone, c, is calculated at the nominal strength based on Eqn. 32, thus assuming that the wall rocks about an axis at the distance, c, from the extreme compression fibre in the wall. As d_{ty} is considered as the displacement increment beyond d_n, the stress state in the extreme tendon should rigorously be taken as f_{ps}, however using f_{se} (initial tendon stress in unloaded state) instead of f_{ps} in Eqn. 45 generally results in little error.

Given θ, the force increase in the individual tendons can be calculated as:

$$\Delta P_i = \frac{\theta(e_{tj} - c)}{h_e} E_s A_{psi} = (f_{py} - f_{ps}) A_{psj} \frac{e_{tj} - c}{e_{te} - c} \qquad [46]$$

$$\Delta P_y = \Sigma \Delta P_{tyj} \qquad [47]$$

where e_{tj} is the location of the j'th tendon with respect to the compression end of the wall, A_{psj} is the area of the j'th tendon and ΔP_y is the total tendon force increase above that at M_n. Note that Eqn. 46 assumes linear variation of the tendon force increase with respect to the lateral location of the tendons. The resulting moment increase M_{ty} is then given by:

$$M_{ty} = \sum_{j=1}^{n} \Delta P_{tyj} \left(e_{tj} - \frac{a_y}{2} \right) = \sum_{j=1}^{n} \Delta P_{tyj} e_{tj} - \frac{a_y}{2} \Delta P_y \qquad [48]$$

where n is the total number of tendons along the length of the wall and the compression zone length at first yield may be calculated as:

$$a_y = \frac{P + \Delta P_y + N}{\alpha f'_m b_w} \qquad [49]$$

Finally the yield moment M_y and displacement d_y can be evaluated as:

$$M_y = (N + P + \Delta P)\left(\frac{L_w}{2} - \frac{a_y}{2} \right) + M_{ty} = V_y h_e \qquad [50]$$

$$d_y = d_n + d_{ty} \qquad [51]$$

4.2.5 Flexural Overstrength

The maximum credible strength of an unbonded prestressed wall may be evaluated by assuming that all tendons have reached their yield strength. Consequently, the flexural overstrength, M_o, may be evaluated as:

$$M_o = (N + P_y)\left(\frac{L_w}{2} - \frac{a_o}{2} \right) = V_o h_e \qquad [52]$$

where a_o is the length of the equivalent ultimate compression block and P_y is the total tendon force when all tendons are yielding given by:

$$a_o = \frac{N + P_y}{\alpha f'_m b_w} \quad \text{and } P_y = A_{ps} f_{py} \qquad [53]$$

At this state, it is assumed that the tendon closest to the flexural compression zone has reached it's yield stress. The resulting displacement can then be evaluated using the following equation which is similar to Eqn. 45:

$$d_o = d_n + \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{tc} - a_o/\beta} \qquad [54]$$

In this equation e_{tc} is the distance from the compression end of the wall to the closest tendon and f_{ps} is the tendon stress in the same tendon at nominal strength. It is noted that Eqn. 54 is not appropriate if the closest tendon is located within the flexural compression zone, i.e. $e_{tc} < c$, and that if the tendon closest to the compression zone is near to the location of the flexural neutral axis, unrealistically large values of d_o are calculated.

When all tendons are located near the wall centreline, the wall yield strength coincides with the wall overstrength. It can be argued for conservatism that the tendon yield stress, f_{py}, in Eqn. 53 should be replaced with the tendon ultimate strength, f_{pu}, in order to establish the maximum credible wall flexural strength. It is, however, unnecessary to modify Eqn. 54 accordingly because the tendon strain at ultimate strength is of the order of 5% and therefore not attainable in reality for walls of any geometry.

4.2.6 Ultimate Displacement Capacity

The ultimate displacement is limited by the strain capacity of the tendons as well as the crushing strain of the masonry. Generally, the tendon ultimate strain is of the order of 5% which would result in unrealistically high displacement. Consequently, concrete masonry failure is expected. Confinement by the foundation is likely to increase the failure masonry strain beyond 0.003. As the extreme concrete masonry fibres fail, there is a tendency for the compression zone to migrate towards the centre of the wall, reducing the wall strength gradually.

Experiments at the University of Auckland have shown drift ratio capacities of 1% - 2% for prestressed grouted concrete masonry walls of various aspect ratios[9], suggesting high displacement capacity. It is noted that this limit state may occur before tendon yielding, depending on the wall aspect ratio, the prestressing steel area and the initial tendon stress f_{se}.

The drift ratio or the drift angle is defined as the ultimate displacement d_u divided by the effective height:

$$\gamma = \frac{d_u}{h_e} \qquad [55]$$

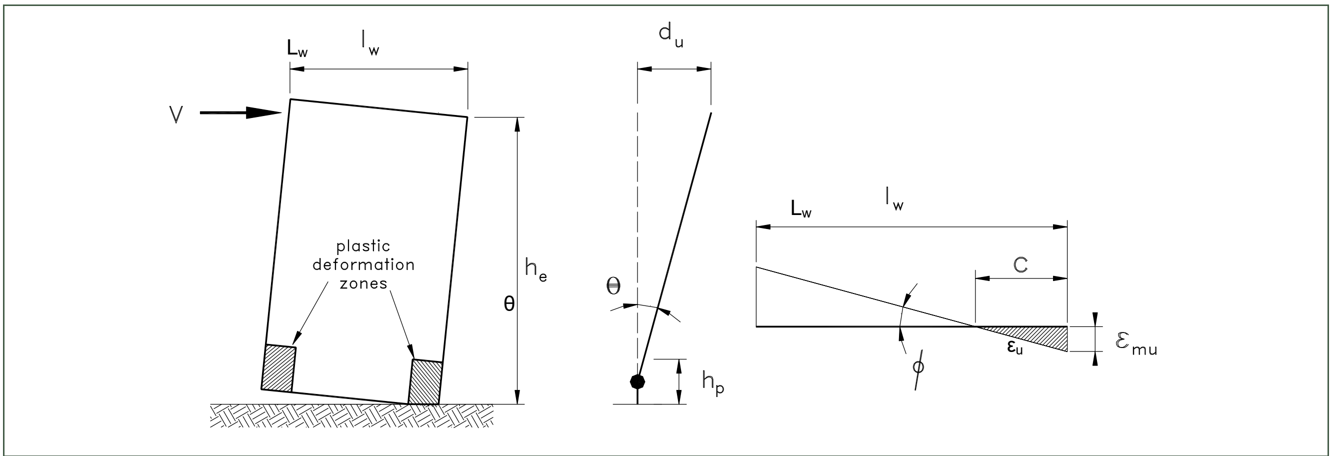

Figure 29: **Vertical Strain Evaluation at Ultimate Displacement Capacity**

Evaluation of the extreme masonry strain at displacements beyond nominal flexural strength necessitates definition of a plastic hinging zone at the bottom of the wall. Assuming that all lateral displacement at the top of the wall is due to rotation, θ, of the plastic hinge as shown in Figure 29, the masonry extreme fibre strain, ε_u, can be related to the wall lateral displacement, d_u:

$$d_u = \theta\left(h_e - \frac{h_p}{2}\right) \text{ and } \theta = \phi h_p = \frac{\varepsilon_u}{c} h_p \qquad [56]$$

$$d_u = \frac{h_p\left(h_e - \frac{h_p}{2}\right)}{c}\varepsilon_u \text{ where } c = \frac{a}{\beta} = \frac{P + \Delta P + N}{\alpha f'_m b_w \beta} = \frac{\xi_u L_w}{\alpha\beta} \qquad [57]$$

In this equation, ΔP should correspond to the actual tendon stress state at the displacement d_u. It is emphasized that Eqn. 57 is of idealised nature and simply attempts to relate the lateral displacement to the masonry strain state in the compression toe region at the wall state where initiation of strength degradation due to masonry crushing is anticipated to commence. Eqn. 56 assumes that the total rotation occurs at a height of $h_p/2$ above the wall base. This is consistent with the current thinking for plastic hinge zone rotation for reinforced concrete masonry walls[1]. For evaluation of d_u, it is acceptable to interpolate between the axial forces calculated at nominal flexural strength, first tendon yield and overstrength relative to the displacements d_n, d_y and d_o, as applicable (with a maximum of $N+P_y$). The base shear corresponding to d_u can be based on Eqn. 31 using the appropriate axial force or on interpolation between V_f, V_y and V_o with a maximum of V_o.

5.0 Prestressed Masonry Shear Wall

Consider the wall shown in Figure 30(a). It is assumed that the five storey wall is 15 m high, 3.6 m long, 190 mm thick and prestressed with five high strength prestressing strands (A_{psj} = 140 mm^2). Half height 20 series concrete masonry units (100 mm high) are used in the plastic deformation zone; regular 20 series masonry units are used elsewhere. The wall self weight is calculated to be 225 kN and the additional dead load of the floors and roof amounts to 0.5 MPa at the base of the wall.

SOLUTION

Gravity load, N = Wall self weight + additional dead load
= 225 kN + 0.5 x (3600 mm x 190 mm)
= 225 kN + 342 kN
= 567 kN

Calculations are performed on the equivalent single degree of freedom structure shown in Figure 30(b) with an assumed effective height, $h_e = 2/3 \times h_w$ = 10 m[**]. The tendons are placed symmetrically about the wall centre line at zero, ±200 mm and ±400 mm eccentricities from the wall centre line (the five strands are represented with one line in Figure 30). In the calculation, the tendon elastic elongation capacity is based on the actual tendon length, approximated as h_w, using an effective tendon elastic modulus of $E_{ps} \times h_e/h_w$. An initial tendon stress of $0.67f_{pu}$ is selected, based on an estimated first tendon yield at a lateral drift of about 1.5% assuming that the wall rocks as a rigid body around the lower corners.

A total prestressing force of $A_{ps} \times f_{ps}$ = 700 x 1187 = 831 kN is found, resulting in an initial axial load ratio of ξ = 0.114 (f'_m = 18 MPa).

Confinement plates are imagined embedded in the horizontal bed joints in the wall corners by the base over a height of $2 \times h_p = 2 \times 0.076 \times 10$ m = 1.5 m and K = 1.08 is assumed[9]. The confinement plate length is taken as $2 \times \xi L_w$ or about 800 mm. It is assumed that the height of the plastic hinge zone is $0.076 \times h_e$ = 0.76 m (the value of 0.076 was found experimentally by Laursen[9]) and the ultimate flexural strain is 0.008, taken from section 7.4.6.4 or Figure 7.1 of NZS 4230:2004.

** The use of $h_e = 2/3h_w$ is an approximate presentation of moment and shear characteristic in a multi-storey wall with a triangular distribution of lateral loads. For specified lateral loads and storey heights, the relationship may be accurately evaluated from $h_e = \Sigma(h_i V_i)/\Sigma V_i$.

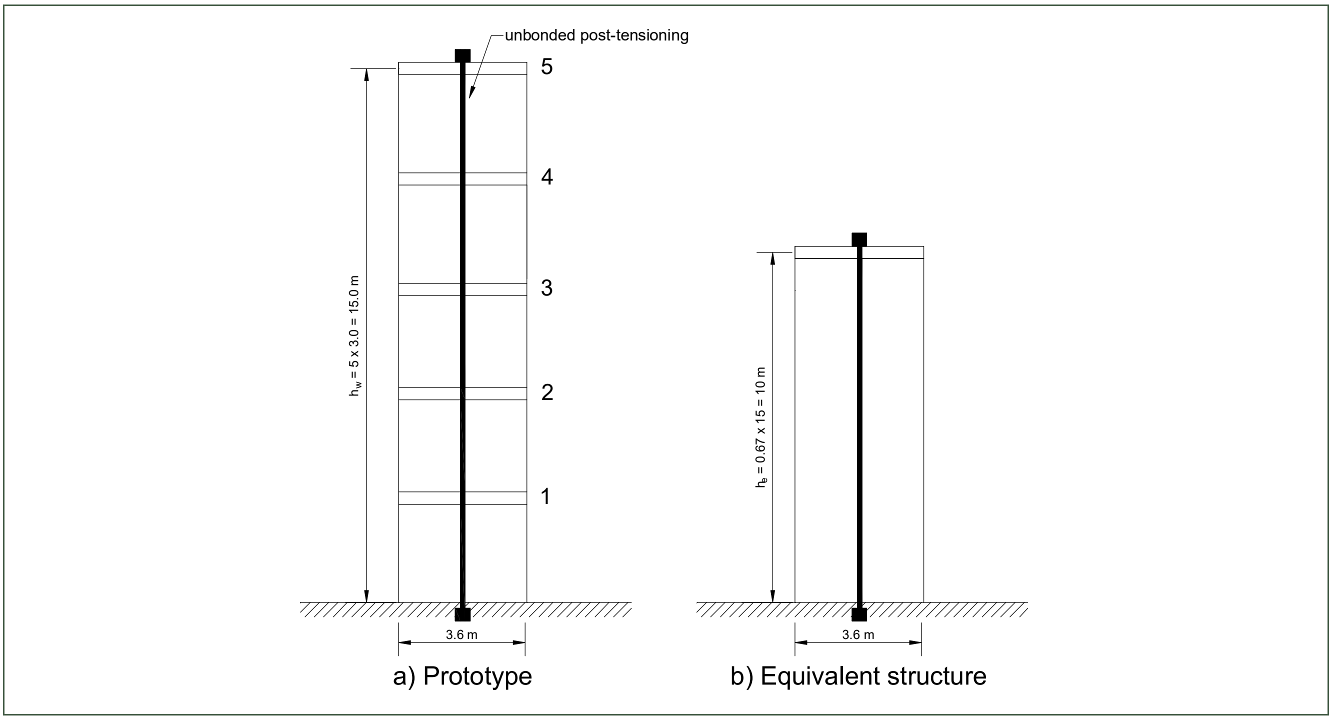

Figure 30: **Post-tensioned concrete masonry cantilever wall**

SOLUTION SUMMARY

Table 12 and Figure 31 present the predicted wall in-plane response with the base shear V, lateral displacement d and tendon force increase ΔP related to the equivalent structure shown in Figure 30(b). Material properties and wall dimensions are specified in Figure 31. Specific details on the calculation example may be found over the page. It is seen in Figure 31 that wall softening initiates between the maximum serviceability moment and the nominal strength limit states. The wall ultimate displacement capacity is reached 83 mm after the nominal strength limit state. The displacement at first tendon yield and wall overstrength is, in this case, only of theoretical interest.

SOLUTION CALCULATIONS

First cracking limit state:

Eqn. 18:

$$M_{cr} = \frac{(567+831)\times 3.6}{6} = 839 \text{ kNm}$$

Eqn. 19:

$$V_{cr} = \frac{839}{10} = 83.9 \text{ kN}$$

Eqn. 20:

$$d_{cr} = \frac{2}{3}\times\frac{(567+831)\times 10^2}{14400\times 3.6^2\times 0.19} + \frac{2}{5}\times\frac{(1+0.2)\times(567+831)}{14400\times 0.19} = 0.0029 \text{ m}$$

Maximum serviceability moment:

Eqn. 24:

$$M_e = 2.04\times\left(0.5-1.21\times\frac{2.04}{18}\right)\times 3.6^2\times 0.19 = 1820 \text{ kNm}$$

$$V_e = \frac{1820}{10} = 182 \text{ kN}$$

Eqn. 28:

$$d_e = \left(0.3-0.029\times\frac{2.04}{18}\right)\times\frac{18\times 10^2}{14400\times 3.6} + \frac{12}{5}\times\frac{(1+0.2)\times 10}{14400\times 3.6\times 0.19}\times\frac{182}{1000} = 0.0108 \text{ m}$$

Table 12: Predicted force and displacement

	First cracking	Maximum serviceability moment	Nominal strength	Ultimate displacement capacity	First tendon yield	Wall over-strength	
V	83.9	182	227	242	248	253	**kN**
d	2.9	10.8	41.2	124	158	310	**mm**
ΔP	0	0	34	109	140	231	**kN**

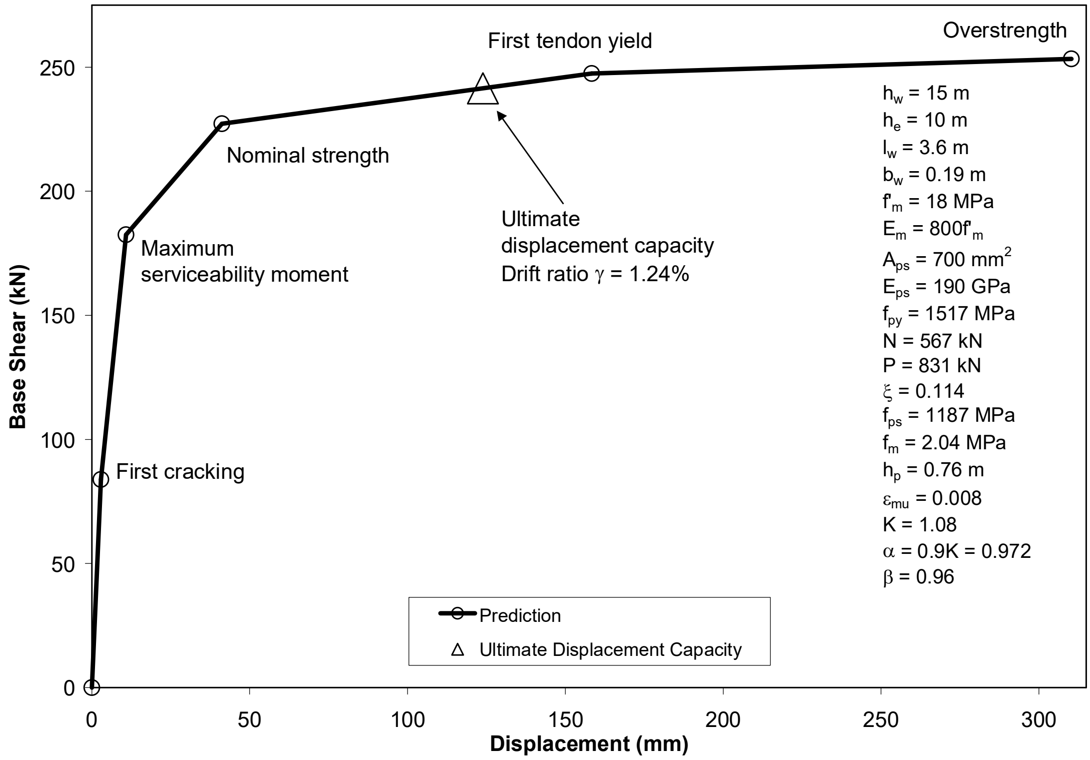

Figure 31: **Predicted in-plane response**

Nominal strength:

First iteration using $\xi_n = 0.114$:

Eqn. 39: $u_e = 0.0117$ m and $u_s = -0.00384$ m

Eqn. 40: $\Delta P_1 = 10.1$ kN, $\Delta P_2 = 8.5$ kN, $\Delta P_3 = 7.0$ kN, $\Delta P_4 = 5.5$ kN, $\Delta P_5 = 3.9$ kN

and $\Delta P = 35.0$ kN, $e_t = 0.004$ m $\rightarrow \xi_n = 0.116$

Second iteration using $\xi_n = 0.116$:

Eqn. 39: $u_e = 0.0115$ m and $u_s = -0.00387$ m

Eqn. 40: $\Delta P_1 = 9.8$ kN, $\Delta P_2 = 8.3$ kN, $\Delta P_3 = 6.8$ kN, $\Delta P_4 = 5.3$ kN, $\Delta P_5 = 3.8$ kN

and $\Delta P = 34.0$ kN, $e_t = 0.004$ m $\rightarrow \xi_n = 0.116$ (therefore OK)

Eqn. 32:

$$a = \frac{831 + 34 + 567}{0.972 \times 18 \times 0.19} = 0.431\ \text{m}$$

Eqn. 31:

$$M_n = (831 + 34) \times \left(\frac{3.6}{2} + 0.004 - \frac{0.431}{2} \right) + 567 \times \left(\frac{3.6}{2} - \frac{0.431}{2} \right) = 2272\ \text{kNm}$$

$$V_f = \frac{2272}{10} = 227\ \text{kN}$$

Eqn. 33:

$$d_n = (7.63 \times 0.116^2 - 5.40 \times 0.116 + 1.69) \times \frac{18 \times 10^2}{14400 \times 3.6} + \frac{12}{5} \times \frac{(1 + 0.2) \times 10}{14400 \times 3.6 \times 0.19} \times \frac{227}{1000}$$

$$= 0.0412\ \text{m}$$

Stress in tendon furthest away from compression end of wall:

$$f_{ps1} = \frac{(P_1 + \Delta P_1)}{A_{ps1}} = \frac{831/5 + 9.8}{140} = 1257\ \text{MPa}$$

Stress in tendon closest to compression end of wall:

$$f_{ps5} = \frac{(P_5 + \Delta P_5)}{A_{ps5}} = \frac{831/5 + 3.8}{140} = 1214\ \text{MPa}$$

First tendon yield:

$c = a/\beta = 0.431/0.96 = 0.449$ m   ($\beta = 0.96$ for confined masonry)

Eqn. 45:

$$d_{ty} = \frac{1517 - 1257}{190000 \times 10/15} \times \frac{10^2}{3.6/2 + 0.4 - 0.449} = 0.1172\ \text{m}$$

where $h_e/h_w = 10/15$ modifies E_{ps} to reflect the actual tendon length.

Eqn. 46:

$$\Delta P_{ty1} = (1517 - 1257) \times 140 \times \frac{3.6/2 + 0.4 - 0.449}{3.6/2 + 0.4 - 0.449} = 36.4\ \text{kN}$$

$$\Delta P_{ty2} = 32.2\ \text{kN}$$

$$\Delta P_{ty3} = 28.1\ \text{kN}$$

$$\Delta P_{ty4} = 23.9\ \text{kN}$$

$$\Delta P_{ty5} = 19.8\ \text{kN}$$

Eqn. 47:

$$\Delta P_y = 140.4\ \text{kN}$$

Eqn. 49:

$$a_y = \frac{831 + 140 + 567}{0.972 \times 18 \times 0.19} = 0.463\ \text{m}$$

Eqn. 48:

$$M_{ty} = 36.4 \times \left(\frac{3.6}{2} + 0.4 \right) \ldots\ldots 19.8 \times \left(\frac{3.6}{2} - 0.4 \right) - \frac{0.463}{2} \times 140 = 229\ \text{kNm}$$

Eqn. 50: $$M_y = (831 + 34 + 567) \times \left(\frac{3.6}{2} - \frac{0.463}{2}\right) + 229 = 2475 \text{ kNm}$$

$$V_y = \frac{2475}{10} = 248 \text{ kN}$$

Eqn. 51: $$d_y = 0.041 + 0.1172 = 0.158 \text{ m}$$

Overstrength:

Eqn. 53: $$P_y = 5 \times 140 \times 1517 = 1062 \text{ kN}$$

$$a_o = \frac{1062 + 567}{0.972 \times 18 \times 0.19} = 0.490 \text{ m}$$

Eqn. 52: $$M_o = (1062 + 567) \times \left(\frac{3.6}{2} - \frac{0.490}{2}\right) = 2533 \text{ kNm}$$

$$V_o = \frac{2533}{10} = 253 \text{ kN}$$

Eqn. 54: $$d_o = 0.0412 + \frac{1517 - 1214}{190000 \times 10/15} \times \frac{10^2}{3.6/2 - 0.4 - 0.490/0.96} = 0.310 \text{ m}$$

Ultimate displacement capacity:

First iteration:

Assume: $$c = \frac{\frac{1}{2}\left(a + a_y\right)}{\beta} = \frac{\frac{1}{2} \times (0.431 + 0.463)}{0.96} = 0.466 \text{ m}$$

Eqn. 57: $$d_u = \frac{0.76 \times \left(10 - \frac{0.76}{2}\right)}{0.466} \times 0.008 = 0.126 \text{ m}$$

Second iteration:

Using d_u found in Eqn. 57, interpolate between a and a_y to find c.

$$c = \frac{a + \frac{a_y - a}{d_y - d_n}(d_u - d_n)}{\beta} = \frac{0.431 + \frac{0.463 - 0.431}{0.158 - 0.041} \times (0.126 - 0.041)}{0.96} = 0.473 \text{ kN}$$

Eqn. 57: $$d_u = \frac{0.76 \times \left(10 - \frac{0.76}{2}\right)}{0.473} \times 0.008 = 0.124 \text{ m} \rightarrow \text{OK}$$

The wall strength at d_u is found by interpolation between nominal strength and first tendon yield limit states with respect to displacement:

$$V_u = V_f + \frac{V_y - V_f}{d_y - d_n}(d_u - d_n) = 227 + \frac{248 - 227}{0.158 - 0.041} \times (0.124 - 0.041) = 242 \text{ kN}$$

4.2 Standard Specifications for Concrete Masonry Buildings Not Requiring Specific Engineering Design

Contents

Preface

This guide is published by the Cement & Concrete Association of New Zealand and the Concrete Masonry Association of New Zealand. It is a revision to section 4.2 of the New Zealand Concrete Masonry Manual.

This guide to NZS 4229 'Concrete Masonry Buildings Not Requiring Specific Engineering Design' is not a substitute for the Standard. It can only be used in conjunction with the Standard.

Standards New Zealand has kindly allowed us to reproduce flowcharts and scope diagrams from the Standard and their permission is gratefully acknowledged.

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Introduction

The New Zealand Standard NZS 4229 `Code of practice for concrete masonry buildings not requiring specific engineering design' was first introduced in 1986, having replaced NZS 1900 Chapter 6.2, and being modelled on NZS 3604 `Code of practice for light timber frame buildings not requiring specific design'.

Since 1986 when NZS 4229 was first released, considerable testing has been conducted providing additional information on the performance of concrete masonry, particularly when subjected to horizontal loads and when using partial grout-filling. From this testing it was recognised that there was considerable opportunity to streamline and simplify the design procedure.

This has been accomplished with the release of the updated Standard, NZS4229:1999.

Because NZS 4229 was a new document in1986, a guide to the use of the Standard was prepared, including four design examples. Just as the new version of NZS 4229 is a more streamline document, so too has this guide been simplified.

In part, this has been accomplished by assuming that much of the material being innovative and potentially misunderstood in 1986 is now widely accepted, so that a comprehensive treatment of that material is now unwarranted.

However, with the latest revision of NZS 4229 a number of additional factors have occurred leading to additional loading. The document has been adjusted to suit the loading provisions of AS/NZS 1170 which has seen:

1. New seismic boundary zones.

2. Influence of soil conditions on seismic actions.

The seismic zoning is now in four zones – 1, 2, 3 and 4, which replaces the A, B, C zoning system. Broadly:

- Zone C is now Zone 1;
- Zone B is Zone 2; and
- Zone A has been divided into two regions with Zones 3 and 4.

However, various towns and cities previously in one zone have been moved to a new zone, e.g. Wanganui was Zone A but is now Zone 2.

The second complication relates to the establishment of sub-soil classes A, B, C, D and E.

These sub-soil classes influence the magnitude of seismic forces reaching the building. All the Tables for seismic demand in NZS 4229 have been written around a Class D sub-soil.

It is now expected that territorial authorities will designate the areas of sub-soil fitting the A, B, C, D and E definitions. The influence can be quite significant, e.g. Soil Class A is solid rock and within Table 4.3 of NZS 4229 there is a modification factor of 0.63 on demand values tabulated, i.e. from the default D soil to solid rock A the demand values are multiplied by the 0.63 factor.

This guide is divided into three sections. The first section defines the scope of the Standard. Buildings outside this scope require specific engineering design.

Section 1 also introduces the basic structural elements of a masonry building, and how these elements interact to transmit loads through the building.

Section 2 of the guide contains a number of design notes clarifying aspects of the design procedure. These design notes are referred to in Section 3, where four design examples of increasing complexity are considered.

Finally, when using this guide it is important to appreciate that it is still necessary to work from the Standard.

The objective of this guide is to provide worked examples demonstrating use of the Standard, but the guide is not a substitute for the Standard.

1.1 Scope

NZS 4229 is a simplified document which may be used to design a range of concrete masonry buildings. As detailed in Section 1 of the Standard, only buildings with the following principal limitations may be designed using NZS 4229 (full reference to the Standard is advised):

(a) Buildings which are not dedicated to the preservation of human life or for which the loss of function would have a severe impact on society, and/or which do not as a whole contain people in crowds, and/or which are not publicly owned and have contents of high value to the community.

(b) Buildings where the total height from the lowest ground level to the highest point of the roof does not exceed 10m, and for which the ratio of the total building height to minimum building width does not exceed 2.5.

(c) Buildings whose configuration complies with the types shown in Figure 1.1 (page 4-5), and whose floor plan does not exceed 600 m^2 for a single storey building, 250 m^2 for a two-storey masonry building, 350 m^2 for a two-storey building where the upper storey is constructed of timber and the external wall of the lower storey is of masonry, or 250 m^2 for a two or three-storey building where the upper storey or stories are constructed of timber, the lower storey is constructed of masonry, and the top storey is contained within a roof space.

(d) Buildings where the live load on suspended floors does not exceed 2 kPa for balconies, or exceed 1.5 kPa otherwise.

(e) Buildings where the roof is constructed of timber, complies with NZS 3604, and has a slope which does not exceed 45°.

(f) Buildings where suspended timber floors comply with NZS 3604 and suspended concrete floors comply with NZS 3101 and do not have a dead load exceeding 4.5 kN/m^2.

Buildings which do not comply with the criteria listed above must be specifically designed using NZS 4230. Note also that in addition to restrictions on building type, Section 3 of the Standard details specific site conditions which are required before the Standard may be used.

1.2 Bracing

One of the most time-consuming exercises when completing a design using NZS 4229 is evaluation of the bracing demand and the bracing capacity for the building.

The approach used in NZS 4229 for determining bracing demands and capacities corresponds to that used in NZS 3604. Using this approach, a `bracing unit' is a specially defined measure of force, where 100 BU's represents 5 kN (or approximately half a ton). Earthquake and wind loading on the buildings is then described as a bracing demand, and the strength of individual walls is described as bracing capacity. The building has sufficient strength when the bracing capacity exceeds the bracing demand.

Criteria for establishing the bracing demand are detailed in Section 4 of the Standard, with criteria for establishing the bracing capacity detailed in Section 5 of the Standard.

In Section C5.1.1 of NZS 4229 it is noted that wall bracing elements of materials other than masonry may be used, provided they are rated at a level having the equivalent strength and stiffness to the masonry wall panels detailed in the Standard.

On this basis, the use of light timber framing to provide partial bracing capacity is not considered in the Standard, nor in this guide. Furthermore, it is noted that for structures where masonry wall panels predominate the structural design, no account should be taken of the bracing capacity of supplementary light timber framing. However, for structures predominated by the use of light timber framing, NZS 3604:1999 provides guidance on the incorporation of concrete masonry bracing elements in Section 8.3.2. In particular, Section 8.3.2.5 of NZS 3604:2011 describes the use of NZS 4229 in the evaluation of bracing provisions for isolated concrete masonry bracing elements in a light timber framed structure. The design examples provided in this guide should assist in clarifying this procedure.

1.3 Lintels

Lintels support vertical (gravity) loading as shown in Figure 1.2. Consequently, lintel design is based upon establishing the vertical load acting on the lintel, and on the distance which the lintel must span. Typically it is assumed that roof trusses transmit vertical roof loads to the exterior walls of the building. On this basis, minimal load is expected to bear on the lintels of interior masonry walls, resulting in longer permissible lintels spans. Lintels are considered in Section 11 of the Standard, and the process for evaluating loading on the lintel is addressed in Section 6 of the Standard. Note that when determining the load acting on the lintel, the eaves overhang should be included in the span dimension, rather than using the distance between supporting walls.

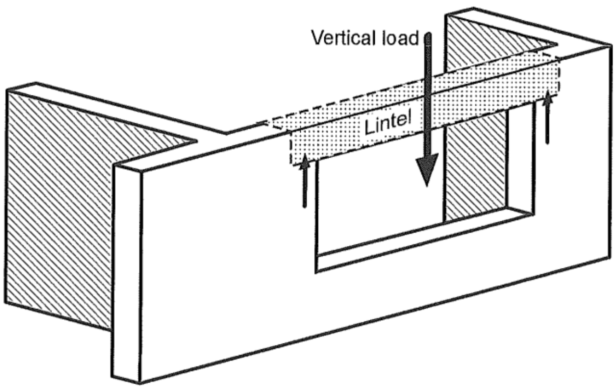


Figure 1.2: Lintel Load Actions

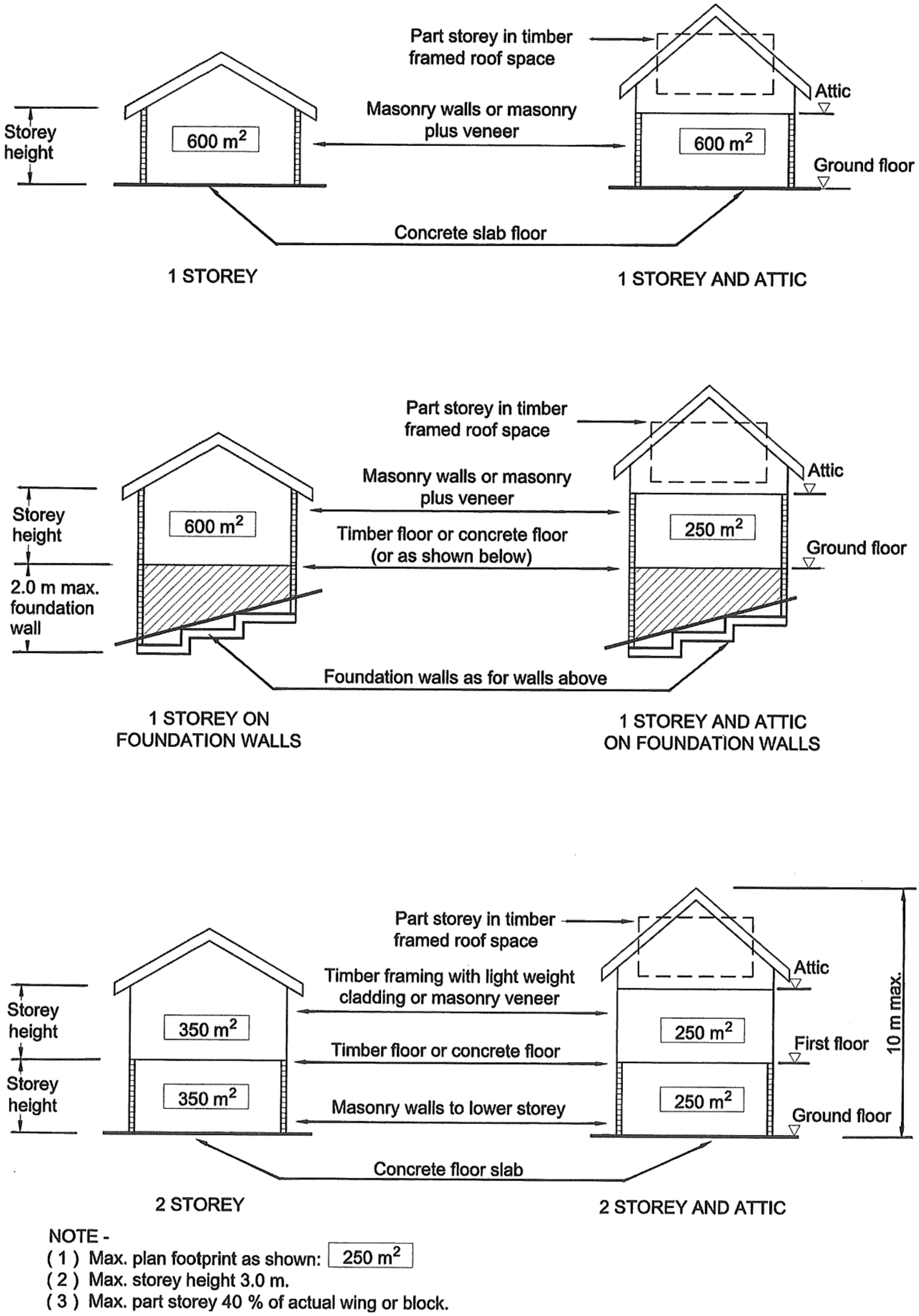
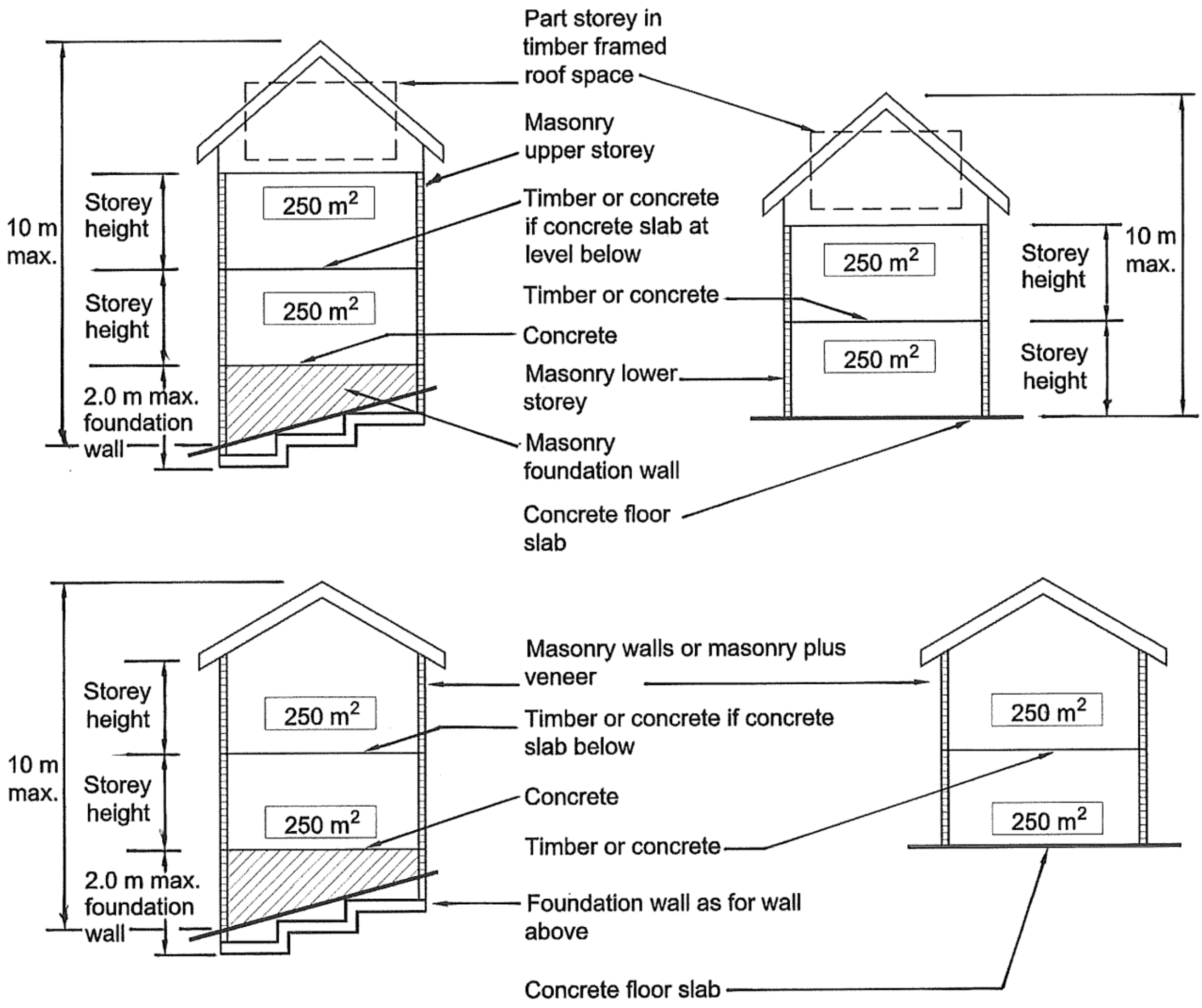


Figure 1.1: Building types covered by NZS 4229



NOTE -
(1) Max. plan footprint as shown: $250\ m^2$
(2) Max. storey height 3.0 m.
(3) Max. part storey 40 % of actual wing or block.

Figure 1.1: Building types covered by NZS 4229 (continued)

1.4 Walls

Whereas lintels support vertical loads as explained above, it is generally found that concrete masonry walls will easily carry even very large vertical loads, and that their design is instead dependent on horizontal loads arising from earthquake and wind.

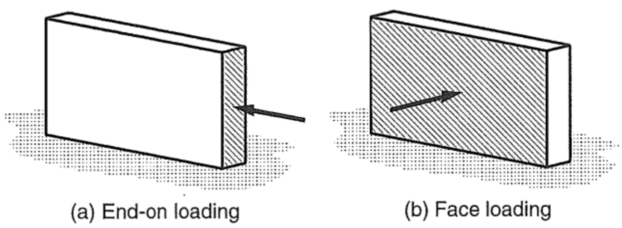


Figure 1.3: Wall Load Actions

Walls may be loaded in two ways as shown in Figure 1.3. When masonry walls are loaded end on, forces are transmitted through the wall by the combined actions of flexure and shear. For walls without openings, the entire height and width of the panel are effective in transmitting load, and are therefore considered in determining the bracing capacity of the panel.

However, for walls with openings it is the smaller panels between the openings which limit the strength of the panel, and so it is these dimensions which are used in evaluating the panel bracing capacity. For these smaller panels, the wall above and below the panel is effective in transmitting loads from diaphragms and bond beams to the foundation, but do not influence the bracing capacity.

When walls are face loaded they are significantly weaker. The procedure used to design walls for face loading is shown in Figure 1.4, where vertical reinforcement spans between the foundation and the bond beam at the top of the wall. Studies have shown that satisfactory performance is obtained if vertical reinforcement is provided every 800 mm along the length of the wall. The design of wall reinforcement is considered in Section 8 of the Standard.

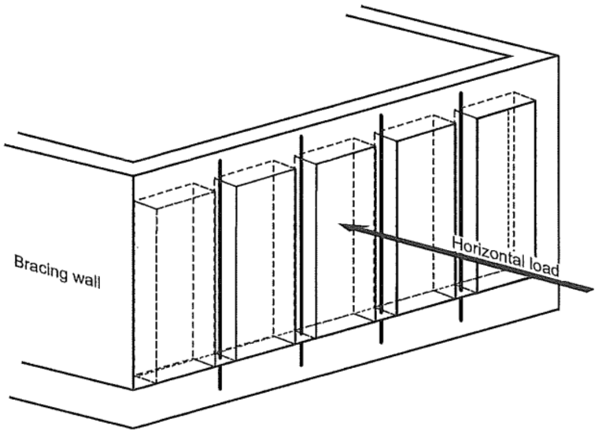


Figure 1.4: Wall Load Actions

When considering multi-storey walls, the height of the wall for calculating bracing capacity in that storey is defined as being from the floor to the top of the bond beam. Consequently, multi-storey walls may be treated as a series of individual single-story walls when considering bracing capacity.

1.5 Bond Beams and Diaphragms

As explained above, horizontal face loading on a wall is transmitted by vertical wall reinforcement to the foundation, and to the bond beam at the top of the wall. The bond beam must then transmit these horizontal forces to the stiff bracing walls oriented in the perpendicular direction, as shown in Figure 1.5. This action is very similar to that of lintels (see Figure 1.2), except that bond beams transmit horizontal forces whereas lintels transmit vertical forces. Bond beams are considered in Section 10 of the Standard.

Because bond beams generally span further than lintels, it is normally found that the bond beam design governs and that the bond beam reinforcement can be used in the lintel. However, when the bond beam span (measured between the centre lines of the supporting bracing walls) is too large, the bond beam becomes ineffective. It is then necessary to either provide intermediate masonry walls, thereby reducing the bond beam span, or to use a structural diaphragm as shown in Figure 1.6. Note that structural diaphragms are stiff elements which effectively transmit forces in a similar manner to the way in which walls transmit end-on loading. Note also that when using a diaphragm, the main function of the bond beam is to ensure that the vertical wall reinforcement is satisfactorily anchored at the top of the wall. In this case a smaller bond beam depth may be used, and lintels may require additional reinforcement beyond that provided in the bond beam. Diaphragms maybe either timber or concrete, and are considered in Sections 8 and 9 of the Standard. Criteria are given in the Standard for the maximum diaphragm length, and for the maximum length/width ratio of the diaphragm.

1.6 Footings

As for lintels, footings are designed for vertical (gravity) loading. The load on the footing comes from the weight of the roof, the weight of any suspended floors (including live loads) and the weight of walls. Footings are considered in Section 6 of the Standard, where design charts and tables permit the various weights acting on the footing to be readily determined. As the design of the footing depends on the weight of all other parts of the building, the footing design is generally the last part of the design process. Note that the footing design assumes good ground conditions, as defined in Section 1.3 of the Standard. Ground conditions not covered by this description fall outside the scope of the Standard.

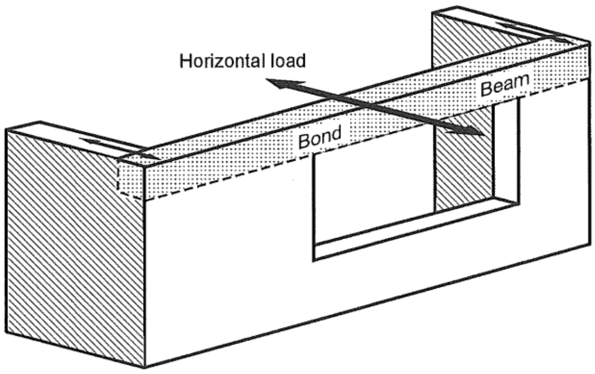


Figure 1.5: Bond Beam Load Actions

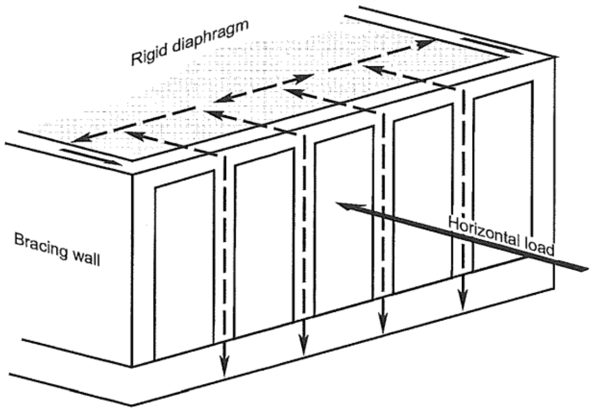


Figure 1.6: Diaphragm Load Actions

2.0 Design Notes for Use with NZS 4229

A number of changes have been made to NZS 4229:2012 with the objective of producing a more user-friendly design Standard. Important changes to the Standard which impact on the design procedure are considered in this section. Those changes, and additional information which may clarify the design procedure, are presented here as design 'notes'.

In the design examples of Section 3 these notes are referred to at the appropriate point in the design procedure.

Note 1: Design Example and Flowcharts

In Appendix A of NZS 4229:2012 are flowcharts and a design example aimed at simplifying use of the Standard. These flowcharts are reproduced here as Figures 2.1 and 2.2 (pages 8-11). The provision of these flowcharts is of major advantage in completing a design.

Note, however that the design examples included herein do not make specific reference to these flowcharts.

Note 2: Use of 15 Series Masonry

Prior to publication of the New Zealand Building Code in 1992, criteria related to the prevention of property damage in the event of fire often made it necessary to use 20 series concrete masonry. More recently, these regulations have been relaxed and it is now only necessary to provide for personal safety in the event of fire. This has permitted greater use of 15 series concrete masonry, which in general will result in cost savings of approximately 15%.

Using NZS 4229:2012, 15 series concrete masonry should be satisfactory for many building designs, and in general it is recommended that partial grout-filled 15 series masonry be assumed at the beginning of the design. Should this prove unsuitable, solid grout-filled 15 series masonry can instead be used where necessary.

If this too proves unsuitable, it may be necessary to use 20 series masonry.

Note however that the use of solid grout-filling or increased wall thickness has only minimal influence on bracing capacity, such that the provision of internal masonry walls or changes to the geometry of wall openings may instead be a more appropriate design modification.

Note 3: Bracing Demand Evaluation

The Standard has been written in a format similar to that of NZS 3604 'Light timber frame buildings not requiring specific design'. Using this format, the earthquake loads on the structure are determined based on the location of the project and the type of structure being built. These loads are expressed as `bracing unit' demands and are expressed as `per square metre' of floor plan because earthquake loads are proportional to the weight of the structure.

Once the bracing unit demand due to earthquake loads has been evaluated, the procedure is repeated for wind loading. Wind loading is considered second because in most locations earthquake loads are expected to govern design. Consequently, a simplified design approach for wind loading is presented. This uses the Extra High Wind values of NZS 3604. If, when using this simplified approach, wind loading is found to govern, it may be advantageous to conduct a more thorough (and less conservative) assessment of wind loading using the procedures detailed in NZS 3604.

Wind loading is evaluated for two orthogonal directions, based upon wind blowing across and along the ridge line, as shown in Figure 2.3.

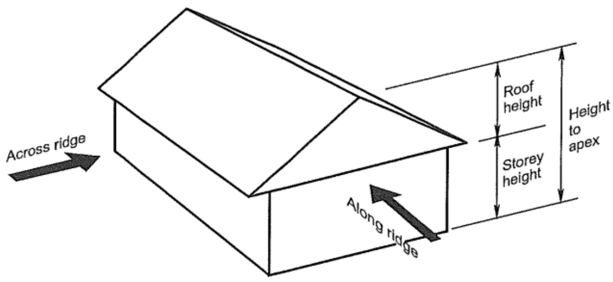


Figure 2.3: Wind Loading

Because bracing demand due to wind loading is conducted for two orthogonal directions, it is possible that for one direction wind loading will govern design, but that for the other direction the bracing demand is dictated by earthquake loading.

Note 4: Partially Grout-filled Masonry

Testing has now shown that in the majority of applications, satisfactory structural performance can be achieved using partially grout-filled masonry. Unless solid-filled masonry is required for other reasons, it is recommended that partial grout-filling be assumed at the beginning of the design.

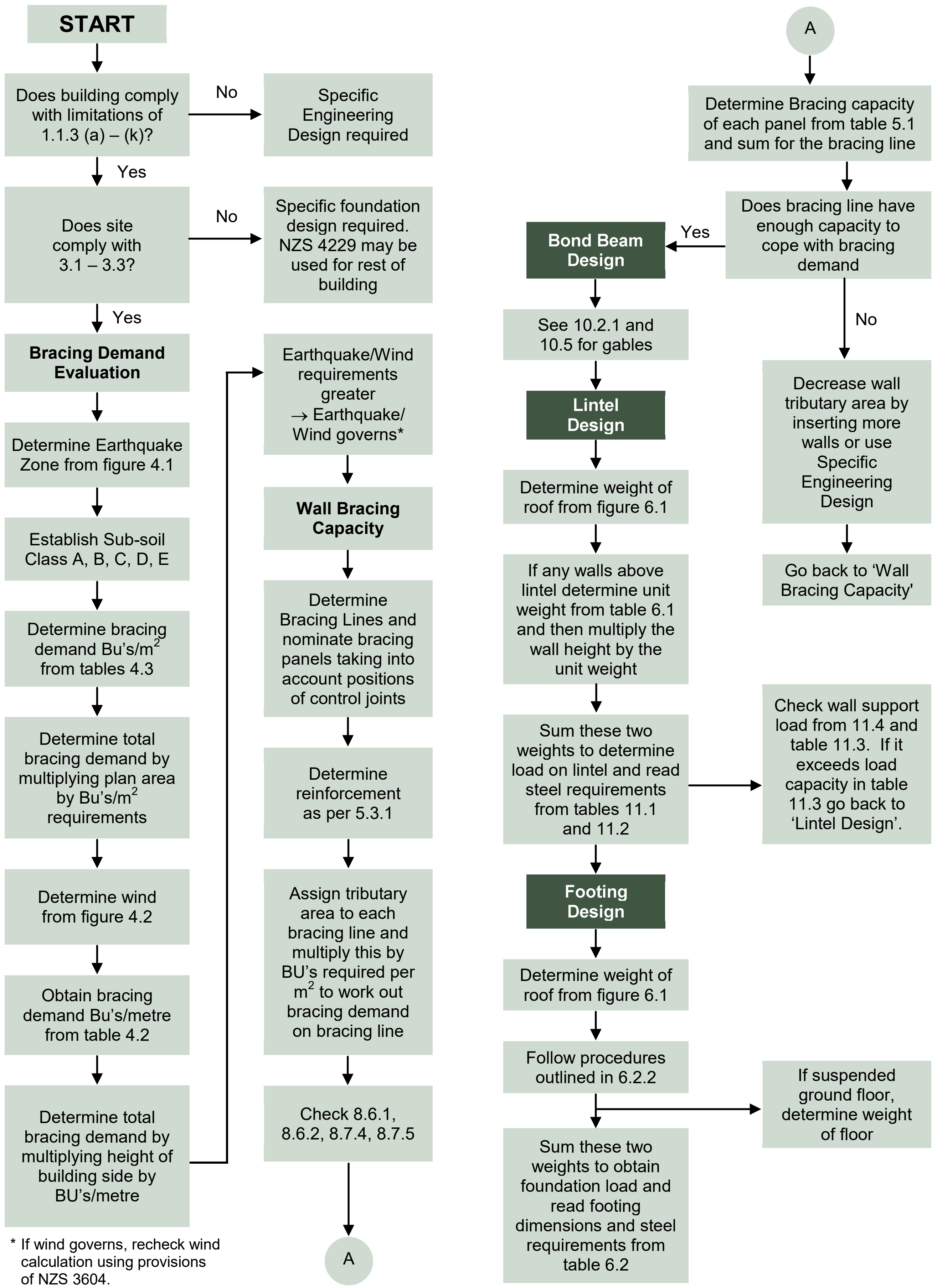


Figure 2.1: Flow Chart for a single storey design

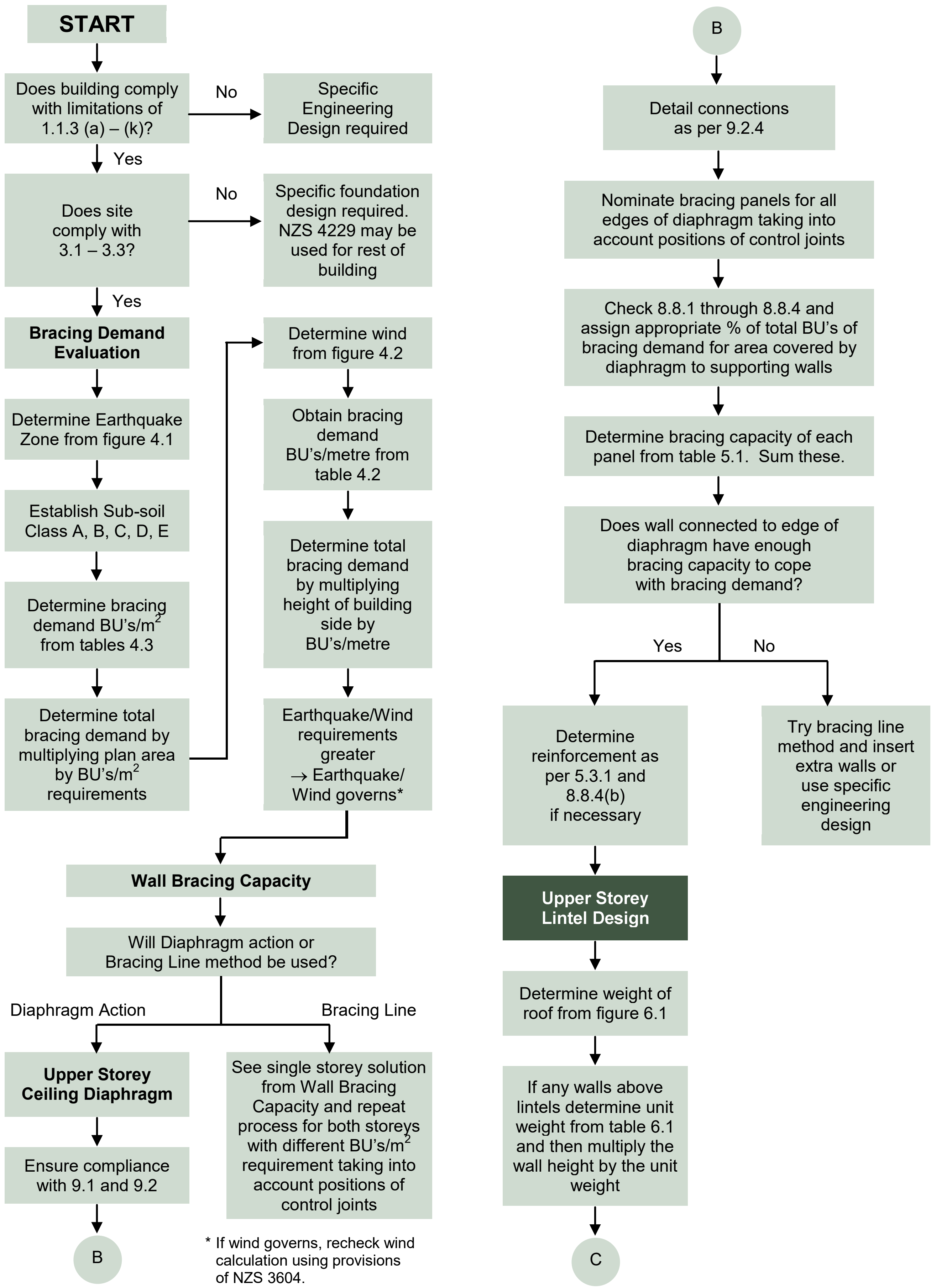


Figure 2.2: Flow Chart for a two storey design

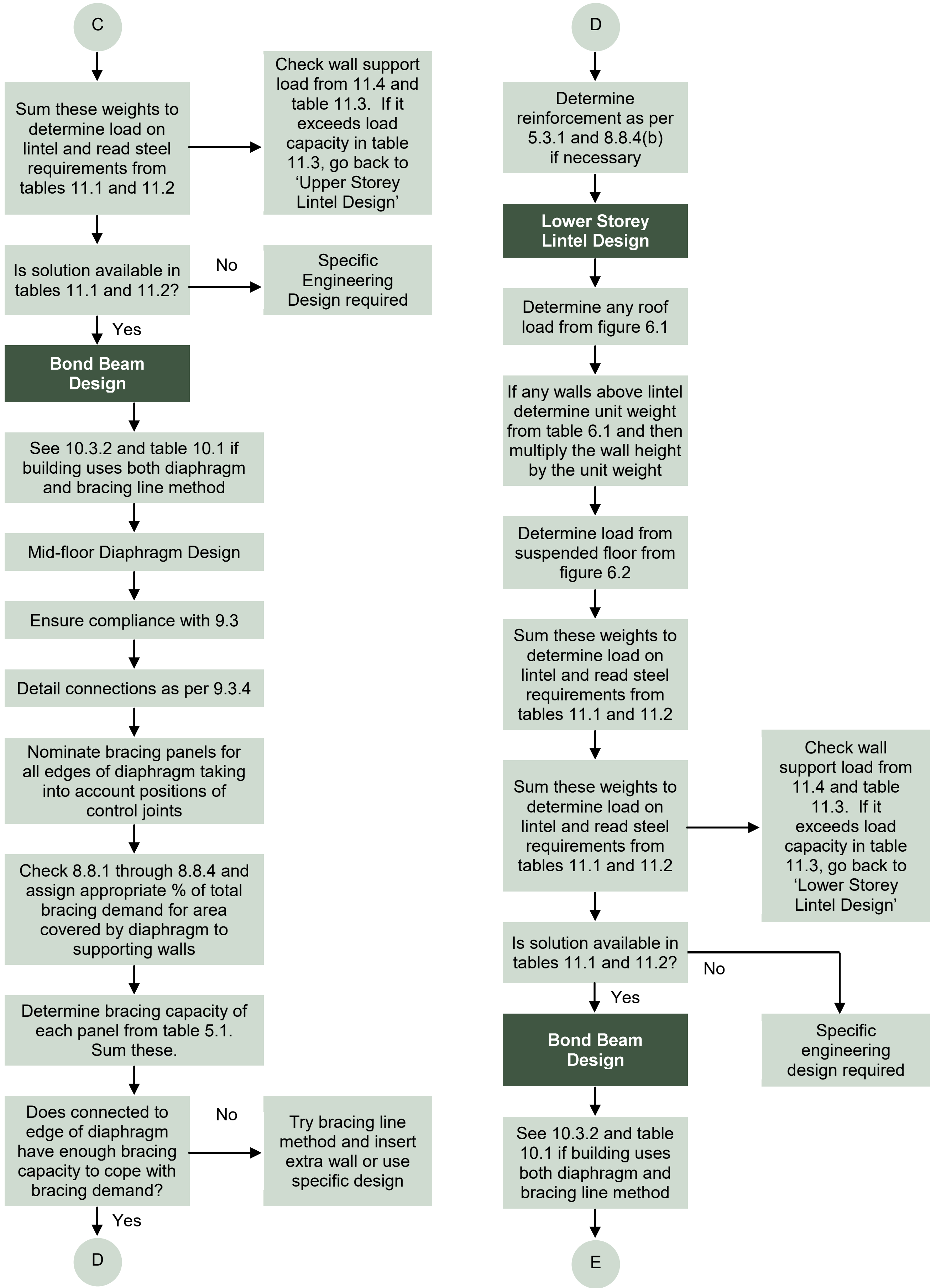


Figure 2.2: Flow Chart for a two storey design (continued)

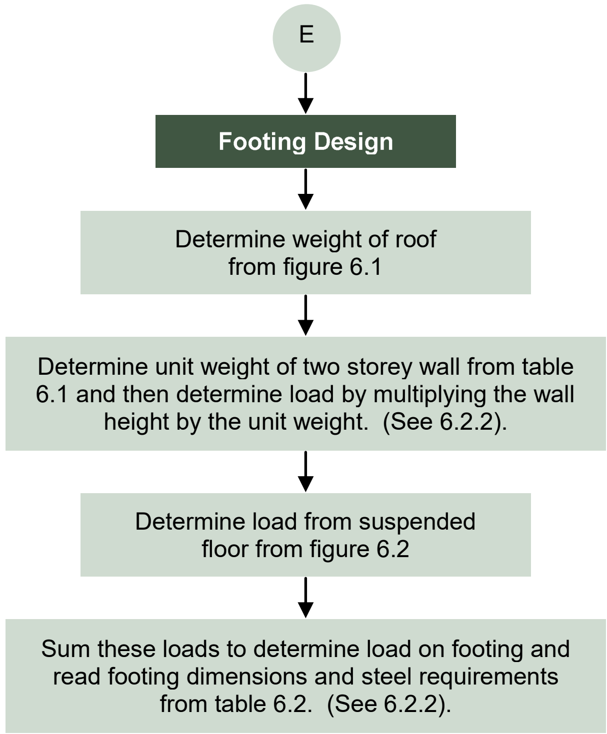


Figure 2.2: Flow Chart for a two storey design (continued)

Note 5: Shrinkage Control Joints

One of the most challenging aspects of completing a masonry building design is deciding where to position the shrinkage control joints.

The following guidelines are based on the requirements of Section 13 of NZS 4229, which covers shrinkage control.

(a) Vertical control joints should be placed at not more than 6 m centres. Where masonry walls meet at right angles, either externally or internally, a control joints should be placed between 600 mm and 5.2 metres from the wall intersection to maintain a maximum of 6 metres between control joints. See figure 2.4.

(b) Vertical control joints preferably should not be placed at the edge of openings, as this creates complications in dealing with the reinforced lintel. Control joints should be placed at least 200 mm from the edge of openings to avoid this reinforced lintel area. (Note that this guideline applies to reinforced masonry construction and differs from the requirements for unreinforced concrete veneer.)

(c) Sometimes, for various reasons, there may be a preference to place control joints at the edge of openings. This can be done by using a special construction detail – which may require specific engineering design.

(d) Vertical control joints should be placed at any change of wall height exceeding600mm.

(e) Vertical control joints should be placed at any change in wall thickness.

Note 6: Maximum Distance between Structural Walls

From Table 10.1 of the Standard it may be established that 15 series bond beams for singular storey heights have a maximum span of between 5.3 m and 8.0 m, dependent on wall, bond beam construction, sub-soil conditions and earthquake zone. Furthermore, Table 8.3 presents the maximum distance between any internal walls, or between internal and external walls. Once bracing lines have been established, this information may be used to identify whether a structural diaphragm is required.

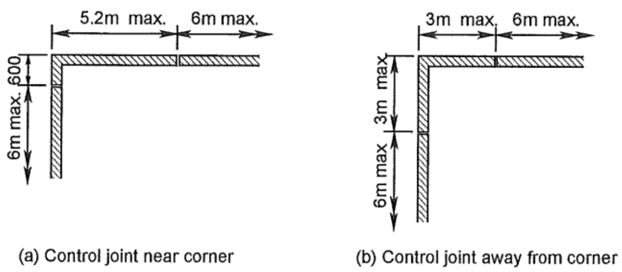


Figure 2.4: Location of Shrinkage Control Joints at Corners

Note 7: Capacity Exceeding Demand

For the building to perform satisfactorily during an earthquake it is necessary for the total capacity of all the bracing panels running in the same direction to be greater than the total bracing demand on the entire building.

As earthquakes may occur in any direction, this requires that the building be checked for both of its perpendicular wall directions. However, in addition each wall must have a minimum capacity. Consequently, even if it has been shown that the building has sufficient total capacity, each bracing line must still be checked to ensure that its capacity exceeds the minimum. This minimum requirement is dependent upon whether or not a structural diaphragm is being used.

Note 8: Gable Ends

Gable shaped walls are considered in Section 10.5 of NZS 4229:2012. In this section there is reference to a 'raking' bond beam which is to be provided to the top of every gable shaped wall. This is accomplished by cutting the masonry units directly below the final course, resulting in an assembly as shown in Figure 2.5.

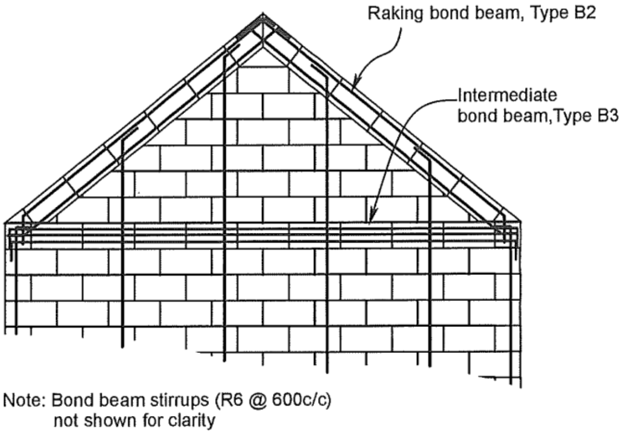


Figure 2.5: Gable End Reinforcing Details

Note that in Figure 2.5 all vertical wall reinforcement is projected to the top of the gable and an intermediate bond beam Type B3 is placed directly below the gable shaped wall.

Although not discussed in the Standard, it is evident that when a structural diaphragm is positioned at the base of the gable, the bond beam will not act in the manner illustrated in Figure 1.5. Consequently, it is recommended that a Type B3 bond beam only be used when a structural diaphragm is not present at the base of the gable.

Because the summed length of the raking bond beam will frequently be greater than that permitted for conventional bond beam design, the use of a sloping roof diaphragm may be required. NZS 4229 may be used for diaphragms with a slope of not greater than 25°. For roof pitches greater than 25°, specific engineering design will be required.

Note 9: Typical Block Densities

For projects located north of Taupo it can be expected that a pumice mix will be used in the manufacture of the concrete masonry units. Consequently, for projects located in this region a block density of 1850 kg/m^3 should be assumed in design. South of Taupo pumice is scarce, and a block density of 2200 kg/m^3 should instead be assumed. This information should be considered when obtaining the wall weight from Table 6.1 of the Standard.

Note 10: Bracing Line Comprised of Discontinuous Internal Walls

Frequently internal walls will be slightly offset along the length of a bracing line. Pairs of discontinuous walls may be treated as a single bracing line if they are both parallel to the bracing line, there is one wall positioned on each side of the bracing line, and each internal wall is no more than 1 m from the bracing line. This is detailed in Section 8.7.5 of the Standard.

Note 11: Minimum Wall Capacity

As explained in Note 7, individual bracing lines must have a certain minimum bracing capacity. Generally this minimum capacity will be dictated by earthquake loading. For walls not connected to a structural diaphragm this minimum capacity will be based on the tributary area which the bracing line must support. For external walls this will correspond to a tributary width of half the distance to the adjacent bracing line, but not less than 2m (see NZS 4229 Clause 8.6.1(a)).For internal walls the minimum capacity will similarly correspond to half the distance between adjacent walls in both directions, but not less than 4m (see NZS 4229 Clause8.7.4(a)).

Individual bracing lines not connected to a structural diaphragm must also be checked for wind loading, although in general this will not be critical. When considering wind loading, the same tributary width of 2m or half the distance to the adjacent bracing line (whichever is greater) is adopted for external walls (see Clause NZS 4229 8.6.1(b)).Similarly, for internal walls the tributary area is the greater of 4 m or half the distance to the next parallel bracing line (see Clause NZS4229 8.7.4(b)), this width defines the area on which the critical wind force acts. Note however that when considering wind loading, it is the geometry of perpendicular walls which governs bracing demand.

Finally, when using a structural diaphragm the minimum strength is not based upon tributary areas, but is instead defined by Clauses 8.8.2and 8.8.4. In general every wall must be able to support 60% of the entire bracing demand on the building.

It is important to note that the diaphragm method can not be used in any earthquake zone where a Class E sub-soil occurs, or where a Class D sub-soil in an earthquake Zone 3 exists. Specific Engineering building design is required.

3.0 Design Examples

References in NZ 4229	Design Calculations	Design Results
A2 A1	In Section A2 of Appendix A in NZS 4229 there is an example illustrating the design procedure for a single storey house. Also contained in Appendix A of NZS 4229 are two flow charts which demonstrate the design process (see Note1, Section 2). Four additional examples are presented here in an effort to further demonstrate the correct use of NZS 4229. These examples consider:	
	(i) A single storey house with 15 series external walls and a heavyweight roof (Section 3.1), demonstrating the use of a ceiling diaphragm.	
	(ii) A two storey house, with the lower storey constructed of 15 series external masonry walls and the upper storey being a light timber frame, having a timber mid-height floor diaphragm, and with timber internal partitioning for both storeys (Section 3.2). This design also demonstrates the detailing of the gables.	
	(iii) A two storey house with 15 series masonry external walls for the lower storey, lightweight timber cladding for the upper storey, a lower storey internal masonry wall and upper storey timber internal partitioning(Section 3.3). This demonstrates the use of multiple diaphragms and the L shaped configuration of the building.	
	(iv) A two storey house with 15 series masonry external walls and a suspended concrete floor (Section 3.4). This design demonstrates the use of the bracing line method for the upper storey.	
A1	In each of the design examples, the flowcharts from Section AI of NZS 4229have been consulted to complete the design. These flow charts are reproduced in Figures 2.1 and 2.2. Note however that the adopted procedure presented here does not specifically match that of the flowcharts.	
	To simplify the examples a number of design aspects are not covered. These include: consideration of site compliance, concrete slab-on-ground design, and the detailing of connections.	
	For each design example the page is subdivided into three columns. The centre column contains the calculations necessary to complete the design, with the left column containing the appropriate references in NZS 4229:2012. The right column lists the design results.	
	Text in the centre column also provides reference to the notes in Section 2, which may be consulted for greater understanding of the design procedure.	

3.1 Design Example 1

References in NZ 4229	*Design Calculations*	*Design Results*
	Design example 1 is a single storey house to be located in Christchurch on a sub-soil Class C, built with 15 series external walls (see Note 2, page 7) and a heavyweight roof. Details of the house are shown in Figure. 3.1.1.	

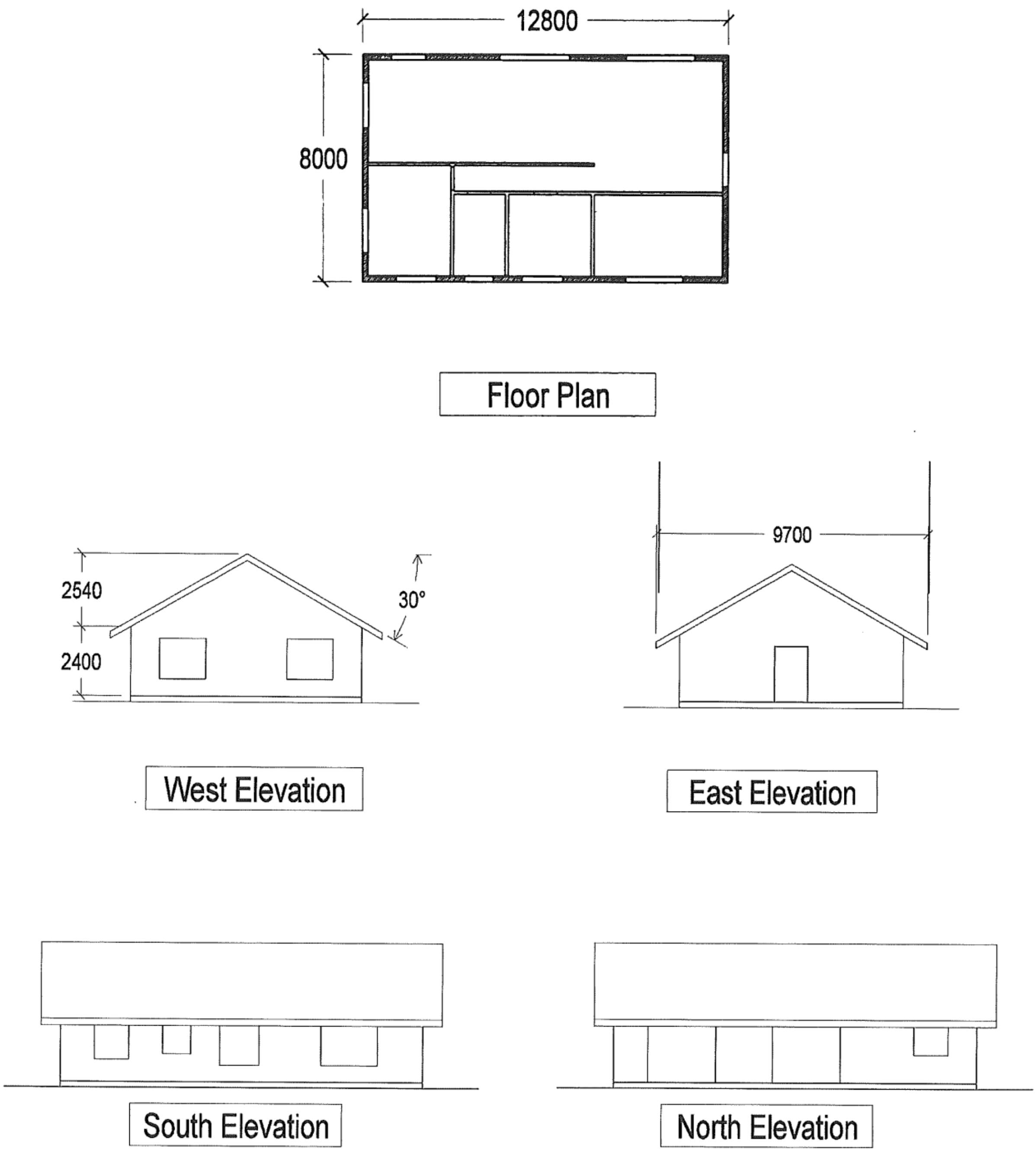


Figure 3.1.1: Design Example 1

References in NZ 4229	Design Calculations	Design Results
	Step 1 – Bracing Demand Evaluation (see Note 3, page 7)	
Figure 4.1 or Table 4.1	*Step 1A – Determine Earthquake Zone*	
	House located in Christchurch.	
	→ Earthquake Zone 2	EQ Zone 2
	Step 1B – Determine Bracing Unit demand for earthquake loading (see Note 4)	
	Single storey 15 series partially filled masonry with heavyweight roof, earthquake Zone 2.	
Table 4.3	→ Demand = 17 + 4 = 21 BU/m^2. Amend for sub-soil Class C 21 x 0.79 = 16.6 BU/m^2.	EQ demand 17 BU/m^2
	Step 1C – Determine total bracing demand for earthquake loading	
	Plan Area = 12.8 x 8 = 102.4 m^2	
1.1.3	Floor plan is less than 600 m^2, so the building type is covered by NZS 4229 (see Figure 1.1)	
	→ EQ Demand = 102.4 x 17 = 1741 BU's	EQ demand 1741 BU's
	Step 1D – Determine Bracing Unit demand for wind loading (default EH)	
	Single storey structure, storey height = 2.4 m, roof height = 2.5 m and height to apex ≤ 10 m.	
	Note that the roof height of 2.5 m is not given in table 4.2 of NZS 4229. The demand is calculated by interpolating values for 2 m and 3 m.	
	→ Demand across ridge = 80 + 0.5 x (111 - 80) = 96 BU/m	Wind demand 96 BU/m across
Table 4.2	→ Demand along ridge = 93 + 0.5 x (111 - 93) = 102 BU/m	102 BU/m along ridge
	Step 1E – Determine total bracing demand for wind loading	Total wind demand 1229 BU's across
	Wall length for wind across ridge = 12.8 m	
	→ Total wind demand across ridge = 96 x 12.8 = 1229 BU's	
	Wall length for wind along ridge = 8 m	
	→ Total wind demand along ridge = 102 x 8 = 816 BU's	816 BU's along ridge
	Step 1F – Determine total bracing demand	
	Earthquake demand = 1741 BU	Design demand 1741 BU's
	Worst case wind demand = 1229 BU	
	→ Earthquake loading governs	
	Step 2 – Determine Bracing Capacity	
	Step 2A – Determine bracing lines, location of shrinkage control joints (see Note 5) and bracing panels	

References in NZ 4229	Design Calculations	Design Results

The adopted bracing lines for design example 1 are shown in Figure 3.1.2. These bracing lines are redrawn in Figure 3.1.3 to show the location of shrinkage control joints (see Note 5, page 11) and the location of individual bracing panels.

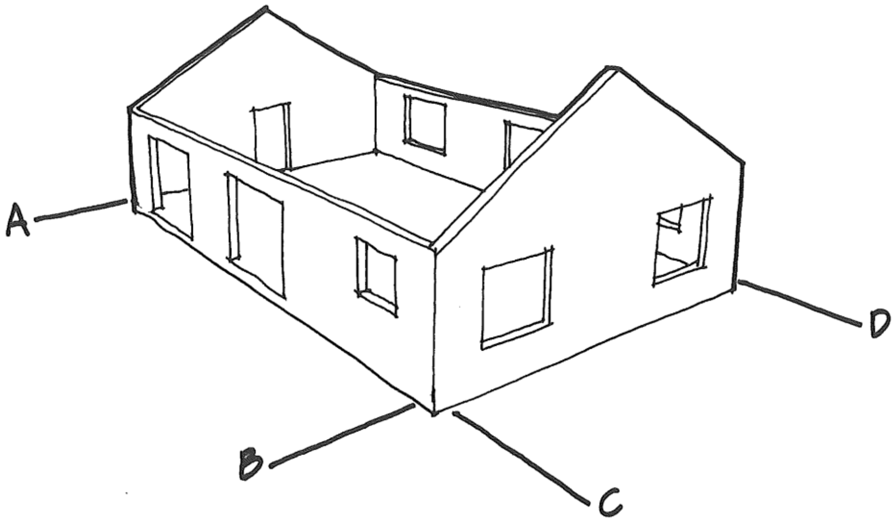


Figure 3.1.2: Bracing Lines for Design Example 1

Step 2B – Method of Bracing

Table 10.1 — 9.2.2

From Figs. 3.1.1 and 3.1.2 it may be established that the distance between bracing lines A and B is 12.6m (measured between the centre lines of the supporting bracing walls), and the distance between bracing lines C and D is 7.8 m. As detailed in Note 6, page 11, the distance between bracing lines A and B is such that a ceiling or roof diaphragm is required. As the roof pitch of 30° falls outside the scope of the Standard, a ceiling diaphragm is required. Note that an alternative to the provision of a roof or ceiling diaphragm would have been the use of an internal masonry wall, thereby reducing the bond beam span to a permissible dimension and allowing use of the bracing line method.

Design result: Ceiling diaphragm required

Step 2C – Determine required minimum capacity of individual bracing lines

8.8.2
8.8.4

Having established in Step 2B that a structural diaphragm was required, minimum requirements for individual walls are given in Clauses 8.8.2 and 8.8.4. In general it is required that each individual wall be capable of supporting 60% of the total demand on the building, although this may be reduced to 30% if the opposite wall can be shown to support 100% of the total demand (see 8.8.4). This information is shown in Table 3.1.1 for a design demand of 1741BU's, as determined in Step 1F.

8.8.2

***Table 3.1.1:* Individual Bracing Line Demands**

Bracing Line	Individual Demand (BU's)
A	1741 x 0.6 = 1045
B	1741 x 0.6 = 1045
C	1741 x 0.6 = 1045
D	1741 x 0.6 = 1045

References in NZ 4229	Design Calculations	Design Results

Step 2D – Determine bracing capacity of each bracing line

Having established in Step 2C the required minimum capacity of bracing line A (1045 BU's), it is now necessary to establish the capacity of individual bracing panels in that bracing line. Note that these bracing panels are shown shaded in Figure 3.1.3.

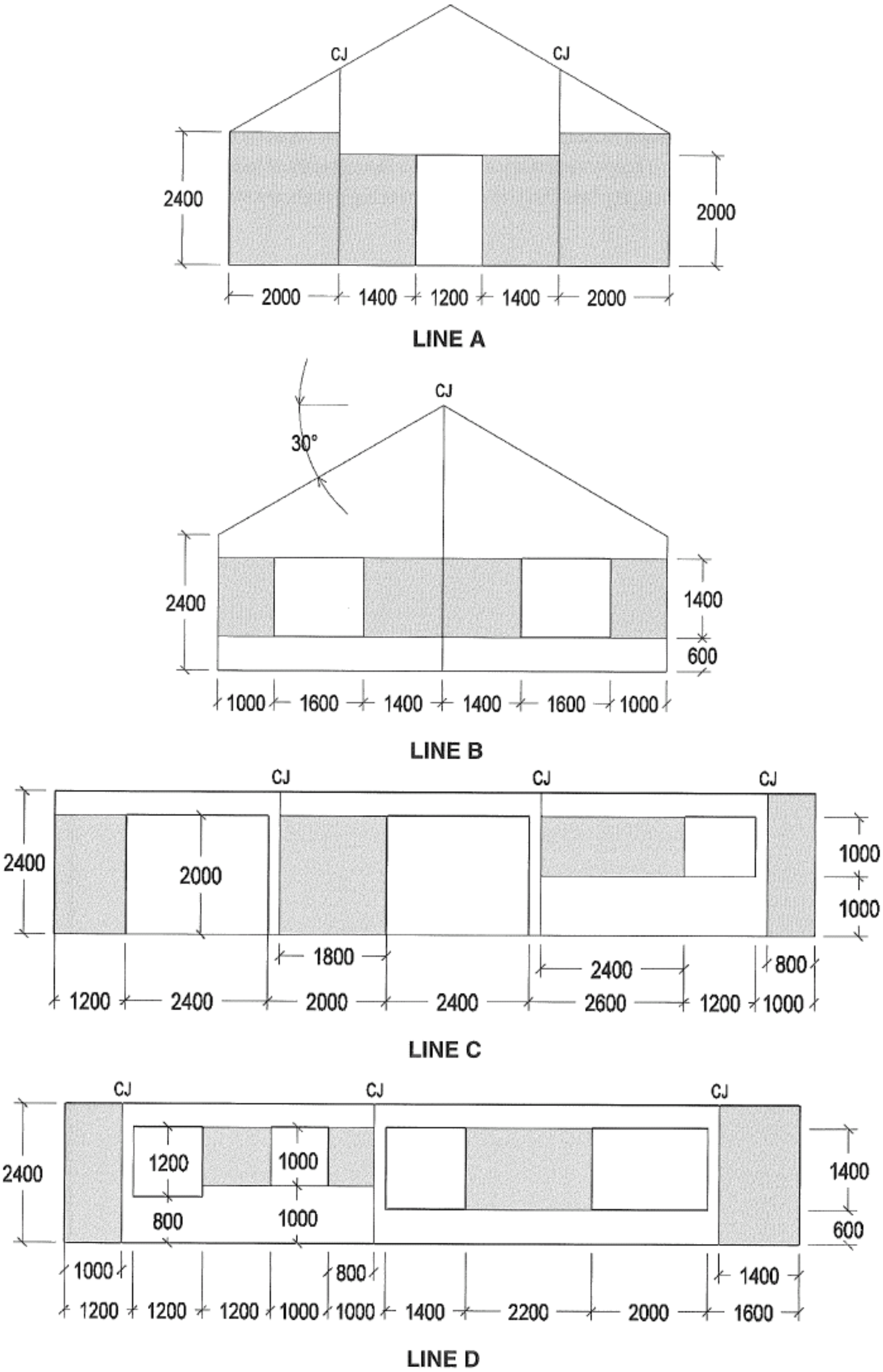


Figure 3.1.3: Bracing Panels for Design Example 1

Having identified the individual bracing panels, their capacity can be found from Table 5.1 of the Standard. This information is recorded below in Table3.1.2 for bracing line A, using 15 Series Partial Fill Masonry.

References in NZ 4229	Design Calculations	Design Results

Note that as a panel length of 1.4m is not given in Table 5.1 of NZS 4229, the values given for lengths of 1.2 m and 1.6 m may be interpolated:

$$\text{BU for 1.4 panel length} = \frac{305 + 480}{2} = 393 \text{ BU's}$$

Table 5.1

Table 3.1.2: **Bracing Capacity of Bracing Line A**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
A	2.4	2.0	15	P	585
	2.0	1.4	15	P	393
	2.0	1.4	15	P	393
	2.4	2.0	15	P	585
					1956

The capacity of bracing line A is 1956 BU's, exceeding the demand listed in Table 3.1.1 of 1045 BU's and the total demand on the structure of 1741 BU's, as detailed in Step 1F.

→ Bracing Line A OK

Bracing Line A OK

As explained in Note 7, page 12, because bracing line A will support the total demand upon the building, it merely remains to ensure that each additional parallel bracing line has sufficient individual capacity as established in Table 3.1.1. Checking the remaining bracing lines running in the same direction as bracing line A:

Table 5.1

Table 3.1.3: **Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
B	1.4	1.0	15	P	333
	1.4	1.4	15	P	535
	1.4	1.4	15	P	535
	1.4	1.0	15	P	333
					1736

As before, the capacity of bracing line B was sufficient to satisfy the demand calculated in Step 2C (1045 BU's).

→ Bracing Line B OK

Bracing Line B OK

As all the bracing lines oriented in the north-south direction have now been checked, it follows that the structure has sufficient capacity for this direction of loading.

→ Building has sufficient capacity in the N-S direction

Now checking panels aligned in the perpendicular direction:

References in NZ 4229	Design Calculations	Design Results

Table 5.1

Table 3.1.4: **Bracing Capacity of Bracing Line C**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
C	2.0	1.2	15	P	305
	2.0	1.8	15	P	580
	1.0	2.4	15	P	1670
	2.4	0.8	15	P	155
					2710

Note that for this bracing line a single panel has sufficient capacity to carry the entire demand on the bracing line.

Bracing Line C OK

→ Bracing Line C OK

Checking the remaining bracing line:

Table 5.1

Table 3.1.5: **Bracing Capacity of Bracing Line D**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
D	2.4	1.0	15	P	208
	1.0	1.2	15	P	560
	1.0	0.8	15	P	330
	1.4	2.2	15	P	1095
	2.4	1.4	15	P	335
					2528

Note that again a single panel had sufficient capacity to support the demand on the bracing line, and that the capacity of bracing line D exceeded the total demand on the structure in the E-W direction of loading.

→ Bracing Line D OK

Bracing Line D OK

Step 2E – Conclusion

From Step IF it was established that the total bracing demand on the house was 1741 BU's, with all bracing lines also having individual bracing demands of 1045 BU's (Step 2C), based upon use of a structural diaphragm.

From Tables 3.1.2, 3.1.3, 3.1.4 and 3.1.5 it has been shown that each panel has sufficient individual capacity, and that panel lines A and B have sufficient total capacity in the North-South direction, and panel lines C and D have sufficient total capacity in the East-West direction.

→ Building has sufficient bracing capacity.

Bracing Capacity OK

References in NZ 4229	Design Calculations	Design Results
	Step 3 – Determine wall reinforcement	
5.3.1	All walls are partial filled 15 series masonry.	All wall rebar D12 @ 800 mm Vertical
Table 8.2(a)	→ Vertical reinforcement is D12 @ 800 mm	
	Horizontal reinforcement is D16 @ 2300 mm	
	Note that the Standard specifies maximum spacing of horizontal reinforcementof2800mm.However the walls are only 2.4 m high, therefore no horizontal wall reinforcement is required other than that for lintels and bond beams.	
	Step 4 – Design timber ceiling diaphragm	
9.1.2 9.2.1	As established in Step 2B, a horizontal timber ceiling diaphragm is required. This diaphragm has a length of 12.8 m and a width of 8m (length less than twice the width).	
	Maximum dimension is 12.4 m	
9.2.2	Less than 16 m → OK	
	Storey height = 2.4m	
9.2.3 Table 9.1 Figure 9.1	Use plaster board ceiling diaphragm in compliance with NZS 9.2.2 (d), nail fixed with 30 mm long 2.5 mm diameter flat head nails at 150 mm centres into framing member at sheet edges, with construction as shown in Figure 9.1 of the Standard.	
	Step 5 – Bond Bean Design	
	Structural diaphragm system being used (see Step 2B and Step 4).	Bond beam depth 200 mm, rebar 1-D16
10.3.1 10.3.2	→ Bond beam reinforcement 1-D16, with a bond beam depth of 200 mm	
10.5.1 10.5.3 Table 10.1 9.2.2 Figure 6.1	Note that bracing lines A and B are gable shaped walls (see Note 8). For these walls a raking bond beam Type B2 shall be provided to the top of the gable. As explained in Note 8, page 12, an intermediate bond beam Type B3 is not required at the top of bracing lines A and B, immediately beneath the gable shaped wall (see Figure 2.5), because a timber ceiling diaphragm frames into the wall at the base of the gable. Furthermore, for a raking bond beam span of 9.2 m, Table10.1 of the Standard indicates a sloping roof diaphragm will be required. As the roof pitch is 30°, the limitations of Section 9.2.2 of the Standard require specific design of the roof diaphragm or specific design of the gable ended wall. Should a specifically designed roof diaphragm be adopted, it will then be unnecessary to use a ceiling diaphragm, in which case a Type B3 bond beam will be required at the top of the wall, directly below the gable (see Figure 2.5).	B2 bond beam at top of gable shaped walls
	Step 6 – Lintel Design	Roof weight 7 kN/m
	Step 6A – Determine weight of roof	
	Heavyweight roof with a total span of 9.7m (see Figure 3.1.1 and Section 1.3).	
	→ Contributing weight = 7 kN/m	

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 6B- Lintel reinforcement	
	From Figure 3.1.3 it may be seen that all lintels are 390 mm deep 15 series, with the longest lintel span being 2.4 m. From Step 6.1 the lintel load is 7 kN/m.	
Table 11.2(a)	→ All lintel reinforcement 2-D12 with R6 stirrups @ 200 c/c	Lintel rebar 2-D12 with R6 @ 200 mn
	Note however that lintels may make advantage of the D 16 from the bond beam (see Step 5), as shown in Figure 3.1.4.	

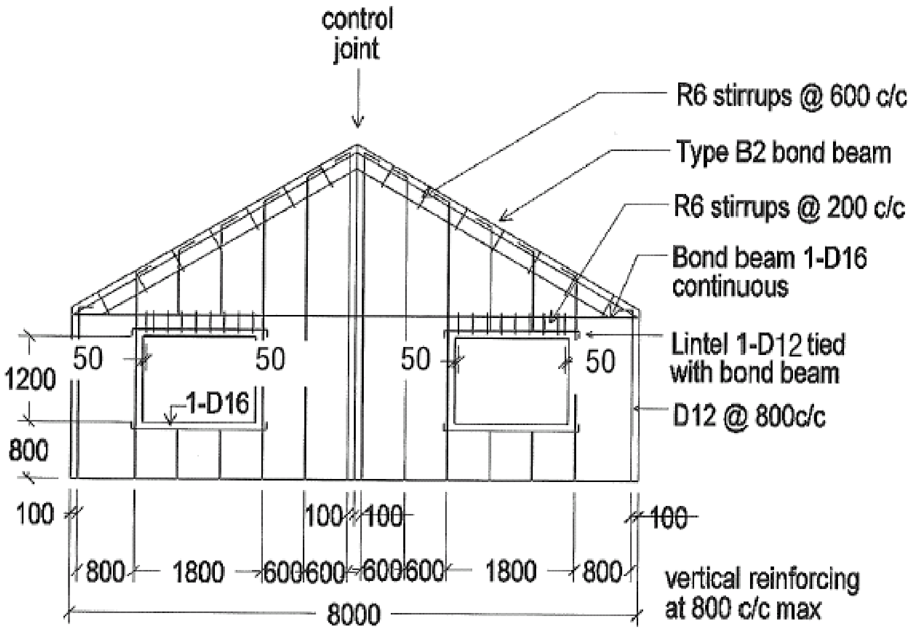


Figure 3.1.4: Bond Beam and Lintel Reinforcement for West Wall (Bracing Line B)

References in NZ 4229	*Design Calculations*	*Design Results*
8.3.2 8.3.5 Figure 8.1	Note also in Figure 3.1.4 that vertical reinforcement must be provided at all ends and corners of walls and on either side of shrinkage control joints and wall openings, and that horizontal reinforcement must be provided immediately above and below all openings.	
Table 6.1	***Step 7 – Footing Design***	
6.2.2	*Step 7A – Determine wall weights*	
	Building located in Christchurch (see Note 9). Assume block density of 2200 kg/m^3 with 4th cores filled with grout. Nominal wall thickness 150 mm.	Weight of wall 6 kN/m
	→ Factored unit weight of wall 2.5 kN/m^2	
	Wall height 2.4 m	
	→ Weight of wall 2.5 x 2.4 = 6.0 kN/m	

References in NZ 4229	Design Calculations	Design Results
	Step 7B- Consider wall axial load at foundation	Load on footing 13.0 kN/m
	Total weight on footing (for wall lines C & D) is weight of roof plus weight of wall. Note that weight of roof was established in Step 6A.	
	→ Weight on footing = 7 + 6 = 13.0 kN/m	
	The weights on the footings for the gable-ended walls (lines A & B) are different from those of the perpendicular walls (lines C & D). These footings carry the weight of the masonry gables but do not carry any significant roof weight.	
	Gable end wall weights are calculated as follows: • Factored unit weight of wall (from Step 7A) = 2.5 kN/m^2 • Height of wall = 2.4 m • Average height of gable = 2.3/2 = 1.15	
	Total weight on footing = (2.4 + 1.15) x 2.5 = 8.9 kN/m	
	This figure is less than that for the walls carrying the roof weight and therefore the higher figure is used to determine footing design.	Load on footing 13.0 kN/m
Table 8.1	This weight is well below the limiting wall capacity of 68 kN/m reported in Table 8.1 of the Standard.	
Table 6.2	→ Wall load capacity OK	Wall load capacity OK
	Step 7C- Detail footing	
	Weight on footing is 13kN/m. Therefore:	
Figure 6.5	Footing { width 300 mm { depth 200 mm { reinforcement 2-D12, with R6 stirrups @ 600 c/c	Footing: width 300 mm depth 200 mm steel 2-D12 with R6 stirrups @ 600 c/c
	See Figure 6.5 of the Standard for the footing cross-section detail.	

3.2 Design Example 2

References in NZ 4229	*Design Calculations*	*Design Results*
	Design example 2 is a two-storey house with a timber suspended floor and a lightweight roof, located in Wanganui on a sub-soil Class D. The upper storey is constructed of timber framing and lightweight timber cladding, and the lower storey is constructed of 15 series concrete masonry external walls (see Note 2) including gable ends (see Note 8) and internal timber partitioning. Details of the house are shown in Figure 3.2.1.	

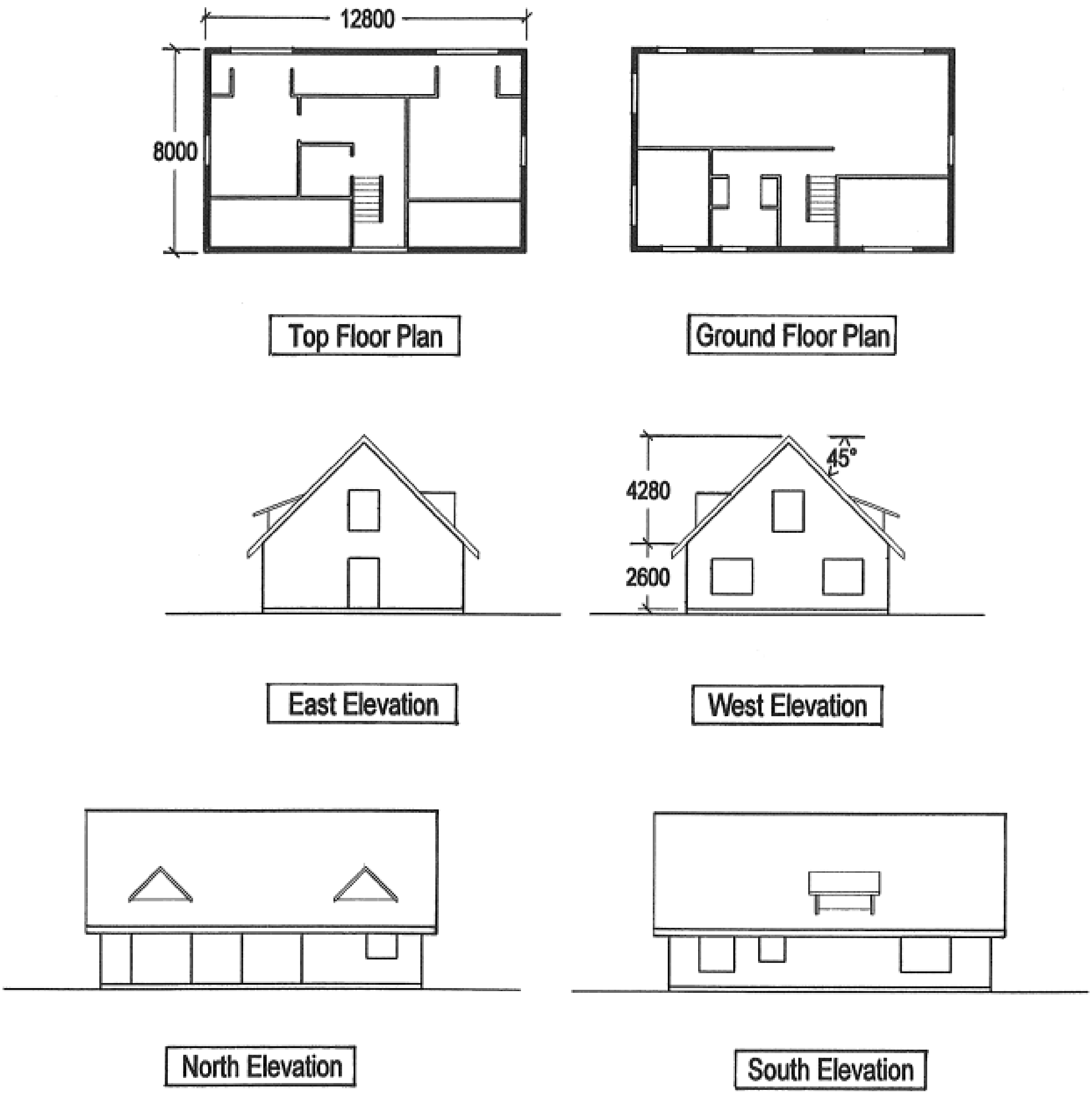


Figure 3.2.1: Design Example 2

References in NZ 4229	Design Calculations	Design Results
Figure 1.1	From Figure 3.2.1 it may be seen that the upper storey floor plan has the same plan area as the ground floor. Considering Figure 1.1 of the Standard it follows that design example 2 falls within the scope of the Standard as a two storey structure having a plan area less than 350 m^2, rather than being treated as a single storey structure with an attic.	
Figure 4.1 or Table 4.1	***Step 1 – Bracing Demand Evaluation (see Note 3)***	EQ Zone 2
	Step 1A – Determine Earthquake Zone	
	House located in Wanganui.	
	→ Earthquake Zone 2	
	Step 1B – Determine Bracing Unit demand for earthquake loading (see Note 4)	
Table 4.3	Two-storey building with first storey 15 series partially filled masonry, second storey timber frame with lightweight timber cladding, having a lightweight roof, and an intermediate timber floor. Building located in earthquake Zone 2.	
	→ Demand = 28 BU/m^2. No correction factor for D sub-soil.	EQ demand 28 BU/m^2
	Note that in determining the above demand, the influence of the masonry gables has been ignored. This is justified because of the absence of upper storey walls in the E-W direction due to the roof pitch.	
	Step1C – Determine total bracing demand for earthquake loading	
Figure 1.1	Plan Area = 12.8 x 8 = 102.4 m^2 Plan area is less than 350 m^2, so that the structure is within the scope of NZS 4229.	
	→ EQ Demand = 102.4 x 28 = 2867 BU's	EQ demand 2867 BU's
	Step 1D – Determine Bracing Unit demand for wind loading (default EH)	Wind demand 185 BU/m across 128 BU/m along ridge
	For the purpose of evaluating wind loading, the structure is effectively single storey, with a storey height = 2.6 m, roof height = 4.3, and height to apex <10 m.	
Table 4.2	→ Demand across ridge = 168 + 0.3 (216 – 160) = 185	
	Demand along ridge = 128 + 0.3 (128 – 128) = 128	
	Step 1E – Determine total bracing demand for wind loading	
	Longest wall for wind across ridge = 12.8 m	
	→ Wind demand across ridge = 185 x 12.8 = 2368 BU's	Total wind demand 2368 BU's across 1024 BU's along
	Longest wall for wind along ridge = 8 m	
	→ Wind demand along ridge = 128 x 8 = 1024 BU's	
	Step1F – Determine total bracing demand	
	Earthquake demand = 2867 BU's	Design demand 2867 BU's
	Worst case wind demand = 2368 BU's	
	→ Earthquake loading governs	

References in NZ 4229	Design Calculations	Design Results

Step 2 – Determine Bracing Capacity

Step 2A – Determine bracing lines, location of shrinkage control joints (see Note 5) and bracing panels

Bracing lines for the building are shown in Figure 3.2.2. Given that the upper storey is primarily constructed of timber framing with lightweight timber cladding, only the lower storey is considered. Note however that Section 8.3.2 of NZS 3604 details consideration of masonry bracing capacity in conjunction with the design of light timber framing. On this basis, minimum bracing panels for use in the upper storey design have been identified.

The bracing panels for design example 2 are shown in Figure 2.3.

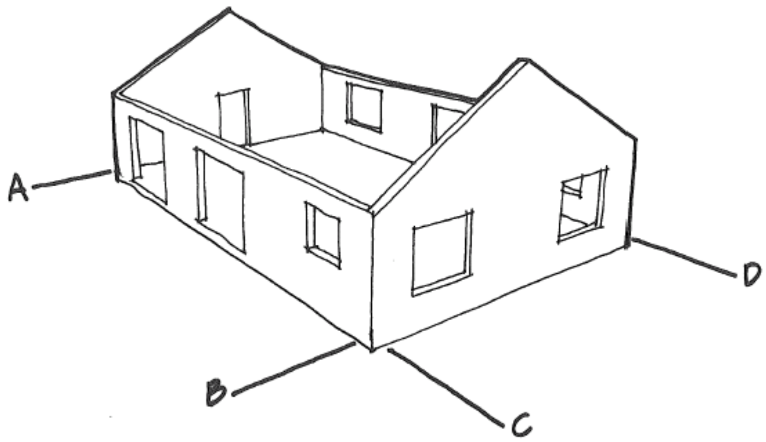


Figure 3.2.2: Bracing Lines for Design Example 2

Step 2B – Method of Bracing

Table 10.1

From Figures 3.2.1 and 3.2.2 it may be established that the distance between bracing lines A and B is 12.6 m (measured between the centre lines of the supporting bracing walls), and the distance between bracing lines C and D is 7.8 m. As detailed in Note 6, page 11, the distance between bracing lines A and B is such that a timber mid floor diaphragm is required. As a timber mid-floor is to be used, the requirement to provide a mid-floor diaphragm is not difficult to accommodate.

Timber floor diaphragm required

Step 2C - Determine required minimum capacity of individual bracing lines

8.8.2
8.8.4

Having established in Step 2B that a structural diaphragm is required, minimum requirements for individual walls are given in Clauses 8.8.2 and 8.8.4. In general it is required that each individual wall be capable of supporting 60% of the total demand on the building, although this may be reduced to 30% if the opposite wall can be shown to support 100% of the total demand (see 8.8.4).This information is shown in Table 3.2.1 for a design demand of 3174 BU's, as determined in Step 1F.

8.8.2

Table 3.2.1: **Individual Bracing Line Demands**

Bracing Line	Individual Demand (BU's)
A	2867 x 0.6 = 1720
B	2867 x 0.6 = 1720
C	2867 x 0.6 = 1720
D	2867 x 0.6 = 1720

References in NZ 4229	Design Calculations	Design Results
8.6.1(b)	*Step 2D – Determine bracing capacity of each bracing line*	

The bracing panels within each bracing line of design example 2 are shown in Figure 3.2.3. Note that these bracing panels differ from those of design example 1, despite similarities between the two examples, because of the larger storey height of design example 2 and the need to relocate shrinkage control joints in the latter example due to the presence of the gable window on bracing line B.

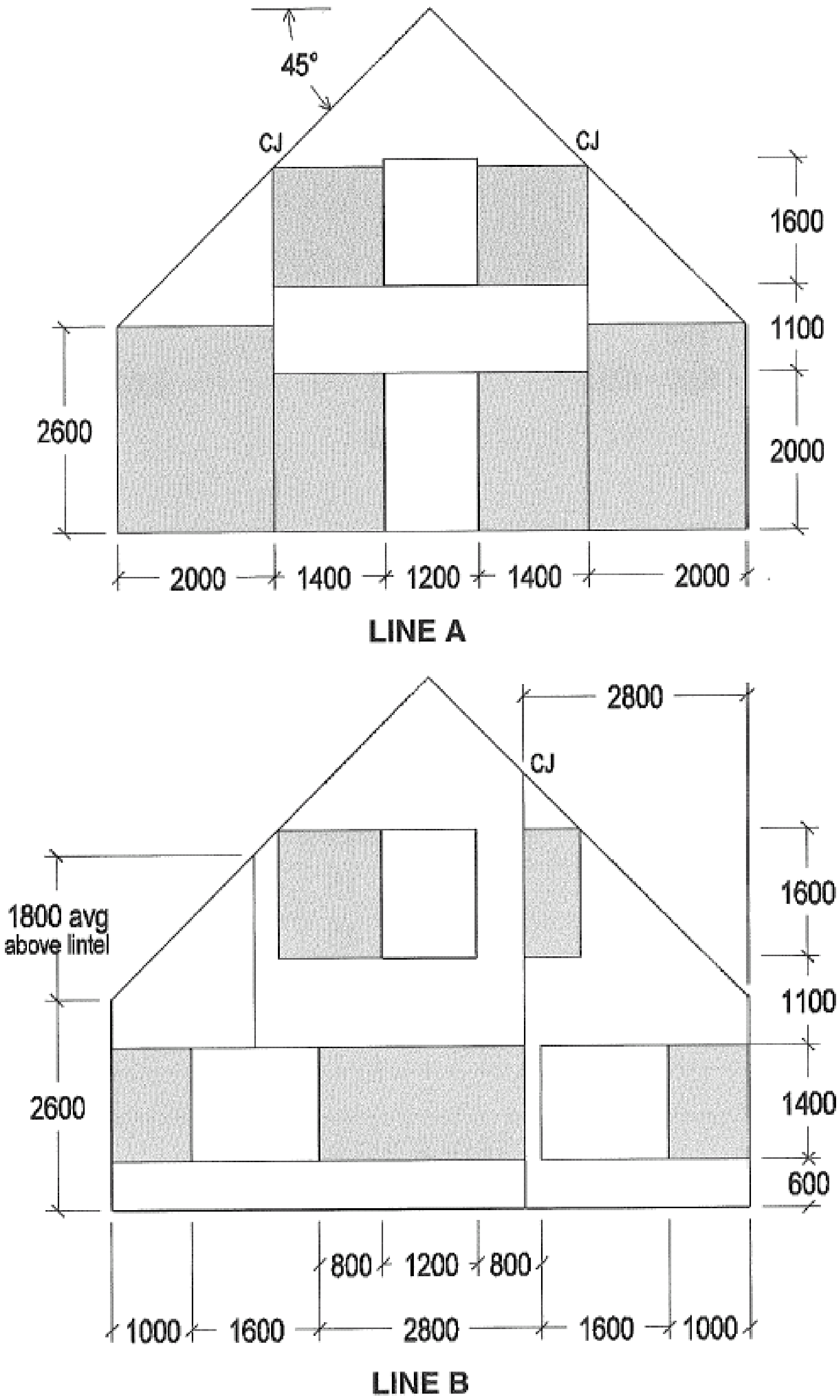


Figure 3.2.3: Bracing Panels for Design Example 2

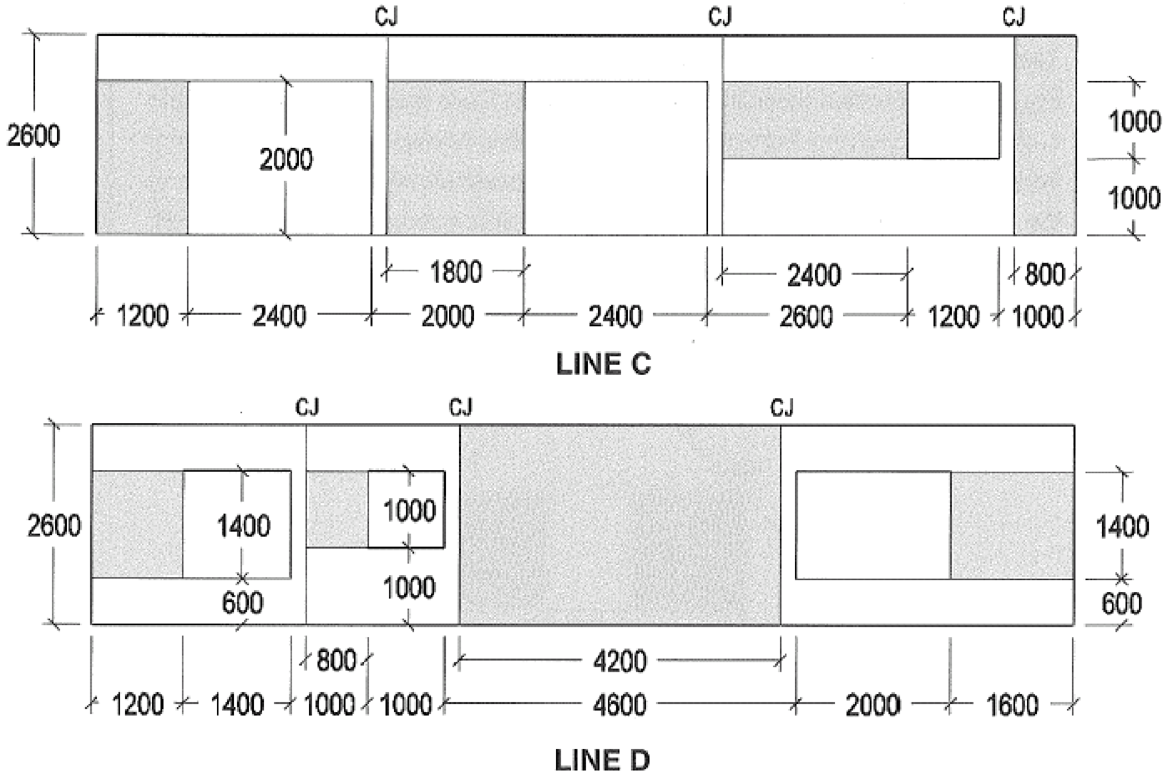


Figure 3.2.3: (Continued) Bracing Line D for Design Example 2

References in NZ 4229	*Design Calculations*	*Design Results*

As detailed in Note 7, page 12, it is next necessary to ensure that the building has sufficient bracing capacity. The capacity of bracing line A is calculated in Table 3.2.2:

Table 5.1

Table 3.2.2: **Bracing Capacity of Bracing Line A**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
A	2.6	2.0	15	P	550
	2.0	1.4	15	P	393
	2.0	1.4	15	P	339
	2.6	2.0	15	P	550
					1886

From Table 3.2.2, the capacity of bracing line A is 1886 BU's, being more than the demand of 1720 BU's given in Table 3.2.1.

→ Bracing Line A OK

Bracing Line A OK

Table 5.1

Table 3.2.3: **Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
B	1.4	1.0	15	P	333
	1.4	2.6	15	P	1448
	1.4	1.0	15	P	333
					2114

References in NZ 4229	Design Calculations	Design Results

From above the capacity of bracing line B (2114 BU's) exceeds the demand (1720 BU's).

→ Bracing Line B OK

Bracing Line B OK

Note that the total capacity of the building in the N-S direction is the sum of 1886 BU's (bracing line A) and 2114 BU's (bracing line B), which exceeds the total demand on the building (2867 BU's) as determined in Step1F.

→ Building has sufficient capacity in the N-S direction.

Now checking bracing lines oriented in the perpendicular direction:

Table 5.1

Table 3.2.4: **Bracing Capacity of Bracing Line C**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
C	2.0	1.2	15	P	305
	2.0	1.8	15	P	580
	1.0	2.4	15	P	1670
	2.6	0.8	15	P	145
					2700

From above, bracing line C has sufficient capacity, exceeding the demand of 1720 BU's.

Bracing Line C OK

→ Bracing Line C OK

Finally, checking the capacity of bracing line D:

Table 5.1

Table 3.2.5: **Bracing Capacity of Bracing Line D**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
D	1.4	1.2	15	P	420
	1.0	0.8	15	P	330
	2.6	4.2	15	P	1615
	1.4	1.6	15	P	650
					3015

Comparing the data in Tables 3.2.1 and 3.2.5, it may be seen that bracing line D has sufficient capacity.

Bracing Line D OK

Step 2E – Conclusion

From Step 1F it was established that the total bracing demand on the house was 2867 BU's, with bracing lines having individual bracing demands of 1720 BU's based upon use of a structural diaphragm (see Step 2C).

From Tables 3.2.2, 3.2.3, 3.2.4 and 3.2.5 it has been shown that each panel has sufficient individual capacity. Given that each panel must support 60% of the total demand on the building, it follows that panel lines A and B have sufficient capacity in the North-South direction, and panel lines C and D have sufficient capacity in the East-West direction.

Bracing capacity OK

→ Building has sufficient bracing capacity.

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 3 – Determine wall reinforcement	
	All walls are partial filled 15 series masonry.	
5.3.1	→ Vertical reinforcement is D12 @ 800 mm	All rebar D12 @ 800 mm Vertical
Table 8.2(a)	Horizontal reinforcement is D16 @ 2500 mm	
	The Standard specifies maximum spacing of horizontal reinforcing of 2800 mm. Note that for wall lines C & D, no horizontal reinforcement is required other than that in the bond beam (at 2500 mm).For the gable ended walls (bracing lines A & B) the lintels above the gable end windows are reinforced with D12 extending to the raking bond beam (Figure 3.2.4), to provide the required horizontal reinforcement.	
	Step 4 – Mid-floor timber diaphragm design	
	As established in Step 2B, a mid-floor timber diaphragm is required. This diaphragm has a length of 12.8 m and a width of 8 m (length less than twice width).	
	Less than 16 m → OK	
9.1.2	Storey height 2.6 m	
9.3.1 9.3.2 Table 9.2 9.3.3	Use plywood floor diaphragm not less than 18 mm thick, nail fixed with 60 mm long, 2.8 mm diameter flat head nails spaced at 150 mm centres, with construction as shown in Figure 9.1 of the Standard. Note that details of the diaphragm to wall connection are shown in Figure 9.5 of the Standard.	Floor diaphragm dimensions OK
Figure 9.1 Figure 9.5	The diaphragm opening has dimensions of 2.0 x 2.8 m. This is outside the allowable size specified in 9.5.1.2 of the Standard. The diaphragm opening therefore requires specific engineering design.	Diaphragm opening requires specific design
9.5.1.2(b)	→ Diaphragm opening requires specific engineering design.	
	Step 5 – Bond Beam Design	
	Structural mid-floor diaphragm system being used (see Step 2B and Step 4).	
	→ Bond beam reinforcement 1-D16, with a bond beam depth of 200 mm	Bond beam depth 200 mm, rebar 1-D16
10.3.1 10.3.2	Note that bracing lines A and B are gable shaped walls (see Note 8).	
10.5.1 10.5.3	For these walls a raking bond beam Type B2 shall be provided to the top of the gable. As explained in Note 8, page 12, an intermediate bond beam Type B3 is not required at the top of bracing lines A and B, immediately beneath the gable shaped wall (see Figure 2.5), because the timber mid-floor diaphragm frames into the wall at the base of the gable. Note that the Standard does not clearly address the design of a steeply pitched gable. For seismic zone A, Table 10.1 indicates a maximum bond beam span of 7.2 m. Summing the length of the raking bond beam we obtain a total length of 11.3 m, indicating that a roof diaphragm should be used with a B1 raking bond beam. However, Section 9.2.2 of the Standard limits the slope of roof diaphragms to 25°. Furthermore, from Figure 3.2.1 it is evident that the roof diaphragm has several opening of significant size. Consequently, specific design of the roof diaphragm would be required. Alternatively, the wall gable may be specifically designed to establish suitable performance as a vertical cantilever.	

References in NZ 4229	Design Calculations	Design Results
	Step 6 – Lintel Design	
	Due to the building configuration, two lintel designs will be considered.	
Figure 6.1	*Step 6A – Determine weight of roof*	
	From Figure 3.2.1 the roof span is 9 m (see Section 1.3) with a lightweight roof	
	→ Contributing weight = 4.5 kN/m	Roof weight 4.5 kN/m
	Note that lintels in bracing lines A and B do not support roof weight.	
	Step 6B – Determine weight of walls above lintel	
	For bracing lines A and B, Figure 3.2.3 shows on average height of masonry gable above the lintel position of 2.4 m (1.6 + 3.2)/2. Excluding self weight (accounted for in tables), the net weight above the lintel is then 1.8 m. Building located in Wanganui (see Note 9).	Upper storey wall weight 61 kN/m
	Assume block density of 2200 kg/m^3 partial filled with grout. Nominal wall thickness 150 mm.	
	→ Contributing weight = 1.8 x 2.5 = 4.5 kN/m	
Figure 6.1	Note that lintels in bracing lines C and D do not support upper storey walls.	
	Step 6C – Determine weight of suspended floor	
	Timber floor with maximum span of 5 m. (Maximum span to internal partition wall).	
	→ Contributing weight = 7.5 kN/m	Floor weight 7.5 kN/m
	Note that lintels in bracing lines A and B do not support the suspended floor.	
Figure 6.2	*Step 6D – Establish total load on lintel*	
	Total weight on lintels in bracing lines A and B = 0 + 4.5 + 0 = 4.5 kN/m.	
	Total weight on lintels in bracing lines C and D = 4.5 + 0 + 7.5 = 12 kN/m.	
	→ Clearly the lintels of bracing lines C and D carry the greater weight and this weight is therefore used to determine lintel design requirements.	
	Step 6E – Lintel reinforcement	
	From Figure 3.2.3 it may be seen that all lintels are 390 mm deep, with the longest lintel span being 2.4 m.	
	→ All lintels 2-D12 with R6 stirrups @ 200 c/c	Total lintel weight 12 kN/m

References in NZ 4229	Design Calculations	Design Results
Table 11.2(a)	Note that as the storey height is 2.6 m, the lintel may not be incorporated with the bond beam without specific design. Figure 3.2.4 shows the reinforcement for bracing line B, including vertical reinforcement at all ends and corners of walls and on either side of shrinkage control joints and wall openings. Figure 3.2.4 also shows horizontal reinforcement below all openings and at less than 2.8 m intervals in the wall gable, complying with the spacing stipulated in Table 8.2(a) of the Standard. Note that when necessary, vertical reinforcement may be positioned in the upper storey wall without requiring reinforcement at the same position in the lower storey wall. Where this is done, the reinforcement in the upper storey wall must be anchored in the bond beam at the base of the upper wall.	Lintel rebar 2-D 12 with R6 @ 200 c/c

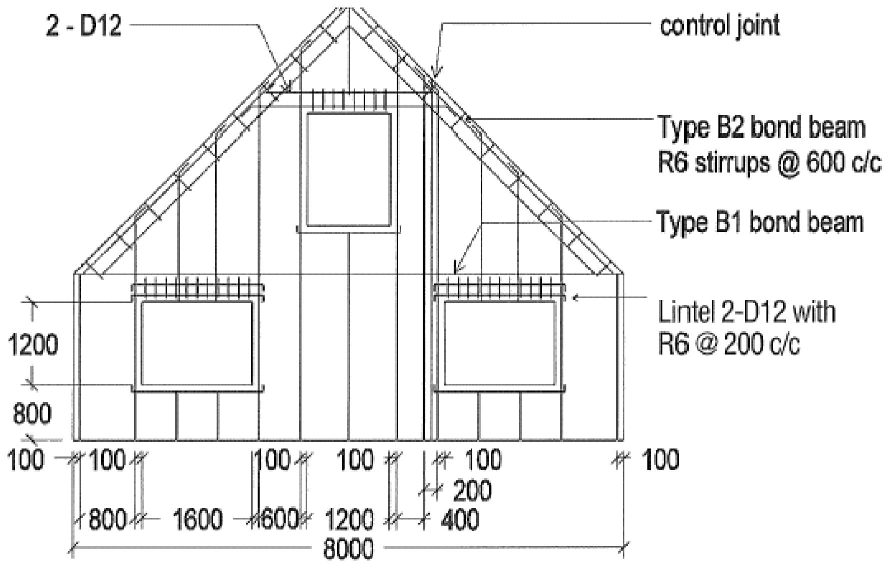


Figure 3.2.4: Bond Beam and Lintel Reinforcement for West Wall (Bracing Line B)

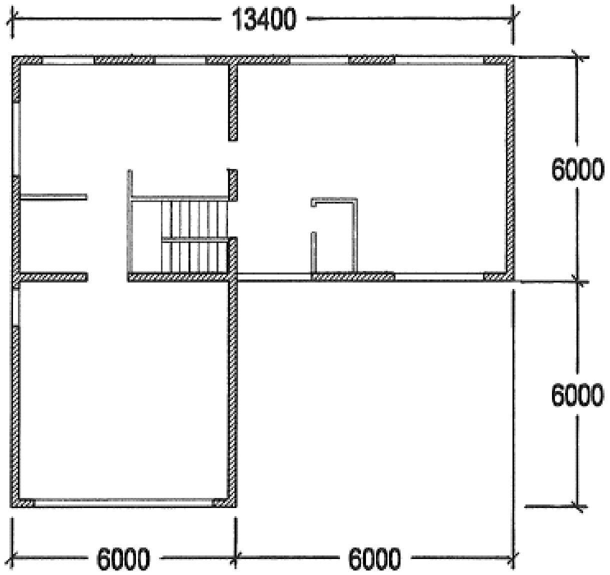
References in NZ 4229	Design Calculations	Design Results
Table 6.1	***Step 7 – Footing Design***	
	Step 7A – Determine wall weights	
	→ Factored unit weight of wall 2.5 kN/m^2	Weight of wall 6.5 kN/m
	From Figure 3.2.1 the wall height is 2.6 m.	
	Contributing weight = 2.6 x 2.5 = 6.5kN/m	
Table 8.1	*Step 7B – Consider wall axial load at foundation*	
Table 8.1	Total weight on footing is weight of roof (step 6A), plus weight of mid-height diaphragm (step 6C), plus weight of lower storey wall (step 7A).	Load on footing 18.5 kN/m
	→ Weight on footing = 4.5 + 7.5 + 6.5 = 18.5 kN/m	
	As in design example one the load on the foundation in wall lines C & D is greater than that of the gable ended walls and is therefore used to determine requirements for footing design.	
	This weight is well below the limiting wall capacity of 68 kN/m reported in Table 8.1 of the Standard.	
	→ Wall load capacity OK	Wall load capacity OK

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 7C - Detail footing	
Table 6.2	Footing { width 300 mm { depth 200 mm { steel 2-D12, with R6 stirrups @ 600 c/c	*Footing:* Width 300 mm depth 200 mm steel 2-D12, with R6 stirrups @ 600 c/c
Figure 6.5	See Figure 6.5 of the Standard for the footing cross-section detail.	

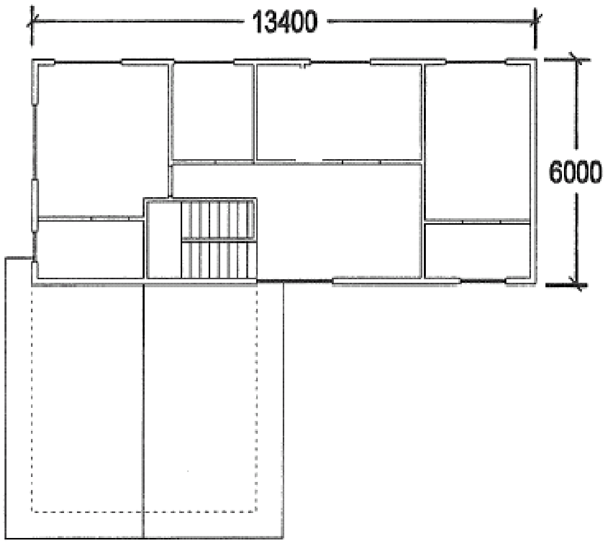
3.3 Design Example 3

References in NZ 4229	*Design Calculations*	*Design Results*

Design example 3 is a two-storey house with a timber suspended floor and a lightweight roof, located in Rotorua on sub-soil Class D. The upper storey is constructed of timber framing with lightweight timber cladding, and the lower storey is constructed of 15 series concrete masonry external walls (see Note 2, page 7) and internal timber partitioning. There is also one internal masonry wall in the lower storey. A two-bay internal garage is located on the lower storey. Details of the house are shown in Figure 3.3.1.



Ground Floor Plan



First Floor Plan

Figure 3.3.1: Design Example 3

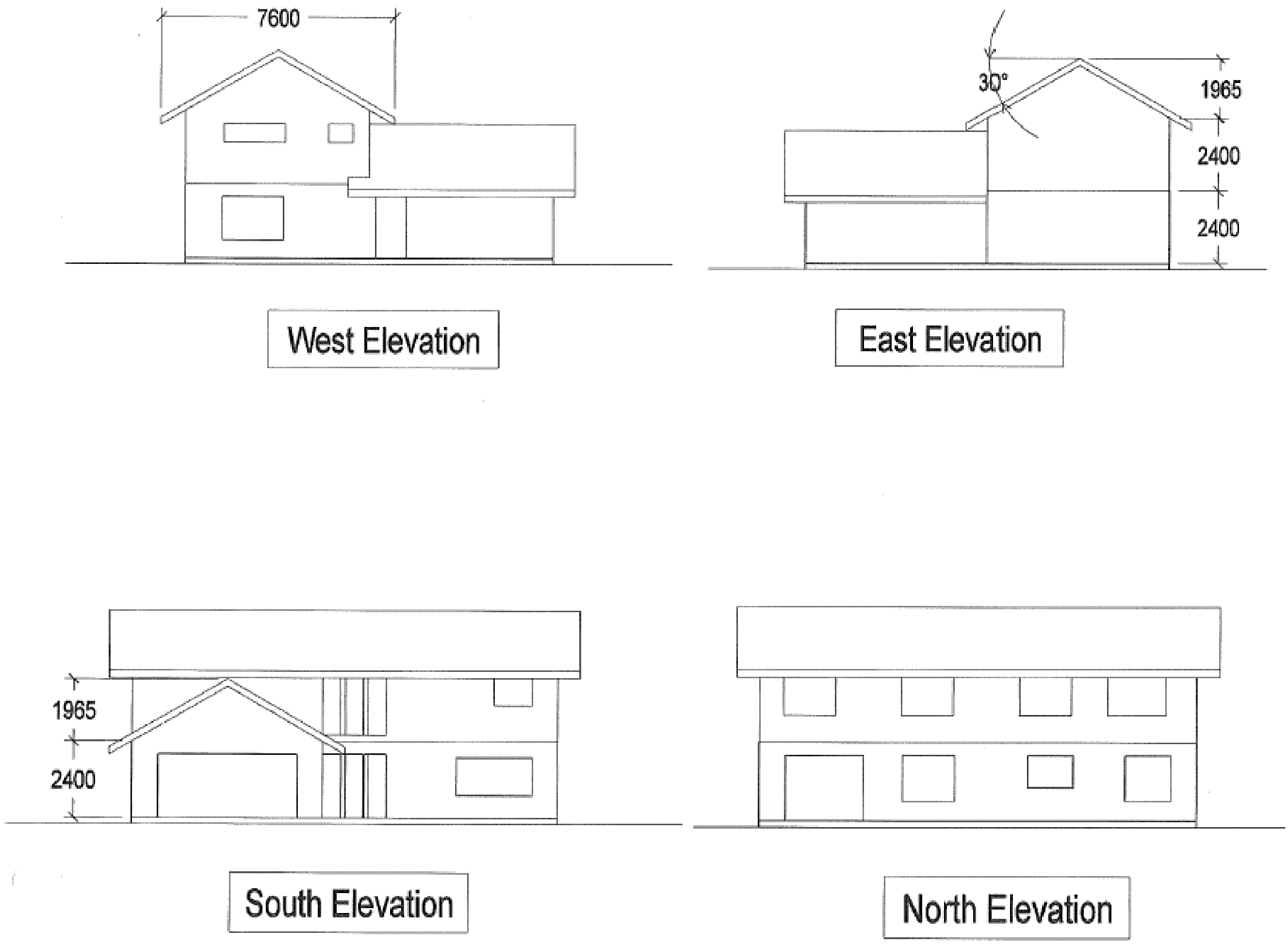


Figure 3.3.2: (Continued) Design Example 3

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 1 – Bracing Demand Evaluation (see Note 3)	
Figure 4.1 or Table 4.1	*Step 1A – Determine Earthquake Zone*	
	House located in Rotorua.	
	→ Earthquake Zone 2	EQ Zone 2
	Step 1B – Determine Bracing Unit demand for earthquake loading (see Note 4)	
	Two-storey building with first storey partially filled masonry, second storey timber with lightweight cladding, having a lightweight roof, and an intermediate timber floor. Building located in earthquake Zone 2.	EQ demand on garage 17 BU/m^2
Table 4.3	→ Demand on lower storey garage = 17 BU/m^2	EQ demand elsewhere 28 BU/m^2
	→ Demand on lower storey excluding garage = 28 BU/m^2	
	No correction for sub-soil Class D.	

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 1C – Determine total bracing demand for earthquake loading	
Figure 1.1	Plan Area = 13.4 x 6 + 6 x 6 = 80.4 + 36 =116.4m^2. The plan area is less than 350 m^2, so that the building falls within the scope of NZS 4229.	
	→ EQ Demand = 80.4 x 28 + 36 x 17 = 2863 BU's	EQ Demand 2863 BU's
	Step 1D – Determine Bracing Unit demand for wind loading (default EH)	
	For single storey garage, single storey height = 2.4 m, roof height = 2.0 m, and height to apex < 10 m.	Single storey wind demand 64 BU/m across 72 BU/m along ridge
Table 4.2	→ Demand across ridge = 64 BU/m	
	→ Demand along ridge = 72 BU/m	
	Two storey structure, storey height less than 3m, roof height 2m, and height to apex 7m.	Two storey wind demand 152 BU/m across 144 BU/m along ridge
Table 4.2	→ Demand across ridge = 152 BU/m	
	→ Demand along ridge = 144 BU/m	
	Step 1E – Determine total bracing demand for wind loading	
	For wind in N-S direction, longest wall is 13.4m across ridge (two storey).	
	→ Total demand = 152 x 13.4 = 2037 BU's	Total wind demand N-S 2037 BU's
	For wind in E-W direction, longest wall along ridge (two storey) is 6m, longest wall across ridge (single storey) is 6m	
	→ Total demand = 144 x 6 + 64 x 6 = 1248 BU's	Total wind demand E-W 1248 BU's
	Step 1F – Determine total bracing demand	
	Earthquake demand = 2863 BU's	Design demand 2863 BU's
	Maximum wind demand = 2037 BU's	
	→ Earthquake loading governs	
Figure 5.1(a)	***Step 2 – Determine Bracing Capacity***	
	Step 2A – Determine bracing lines, location of shrinkage control joints (see Note 5) and bracing panels	
	Bracing lines for the building are shown in Figure 3.3.2. Given that the upper storey is constructed of timber framing with lightweight timber cladding, only the lower storey is considered. Note that for bracing line E it has been necessary to position the shrinkage control joints at greater than 6 m centres. This decision would require consultation with an engineer, and be the subject of specific design.	
	Individual bracing lines are shown in Figure 3.3.3.	

References in NZ 4229	*Design Calculations*	*Design Results*

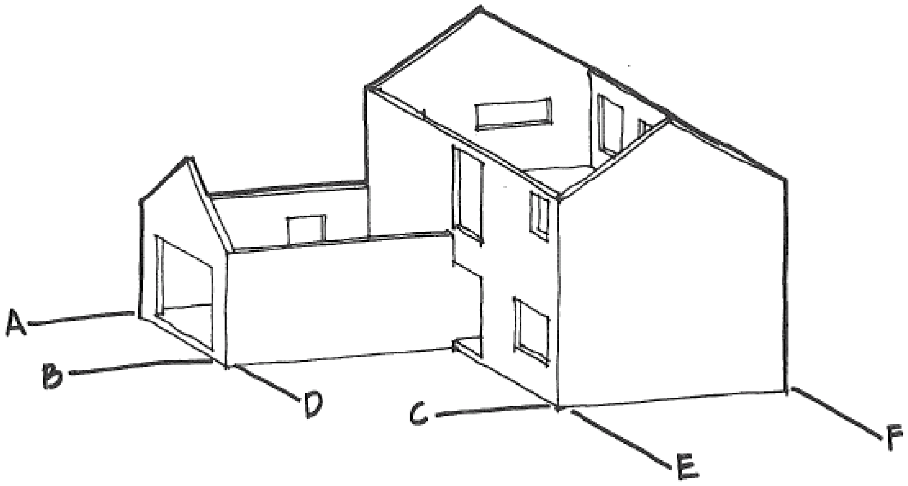


Figure 3.3.2(a): Bracing Lines for Design Example 3

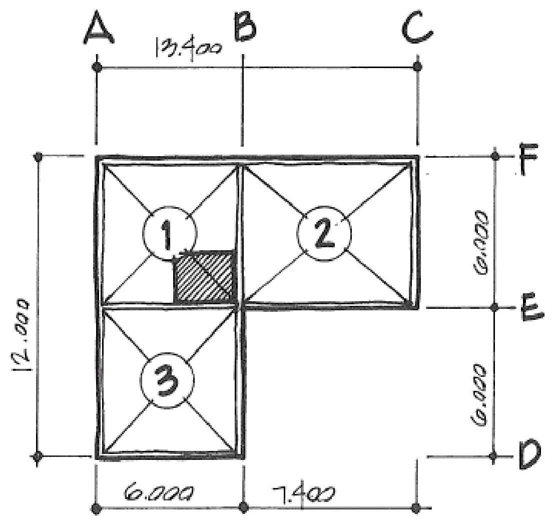


Figure 3.3.2(b): Bracing Lines and Diaphragm Positions

Step 2B – Method of Bracing

Table 10.1 — From Figure 3.3.2 it may be seen that the maximum span length on bracing line E and F is 7.4m. Table 10.1 of the Standard permits a maximum span of 5.6 m for two storey partial filled 15 series masonry in earthquake zone B. On this basis a timber first floor diaphragm will be required (see Note 6).

Design Results: Diaphragm method to be used

Step 2C – Determine required minimum capacity of individual bracing lines

8.8.2
8.8.4 — Having established in Step 2B that a structural diaphragm is required, minimum requirements for individual walls are given in Clauses 8.8.2 and 8.8.4. In general it is required that each individual wall be capable of supporting 60% of the total demand on the adjoining diaphragm, although this may be reduced to 30% if the opposite wall can be shown to support 100% of the total demand. This information is shown in Table 3.3.1 based on the presence of three adjoining diaphragms as shown in Figure 3.3.2. Individual demands are then based on 60% of the earthquake demand (28 BU/m^2 for diaphragm 1 and 2, 17 BU/m^2 for diaphragm 3) and the dimensions of diaphragm 1 (36 m^2), diaphragm 2 (44.4 m^2), and diaphragm 3 (36 m^2).

References in NZ 4229	Design Calculations	Design Results

Table 3.3.1: **Individual Bracing Line Demands for Lower Storey of Design Example 3**

Bracing Line	Adjoining Diaphragm	Individual Demand (BU's)
A	0.6 x (D1 + D3)	972
B	0.6 x (D1 + D2 + D3)	1718
C	0.6 x D2	746
D	0.6 x D3	367
E	0.6 x (D1 + D2 + D3)	1718
F	0.6 x (D1 + D2)	1351

Figure 8.4

Note that for walls having a common diaphragm, as shown in Figure 8.4 of the Standard, further reduction of the demands listed in Table 3.3.1 is possible. The procedure is demonstrated for the lower storey of design example 4.

Step 2D – Determine bracing capacity of each bracing line

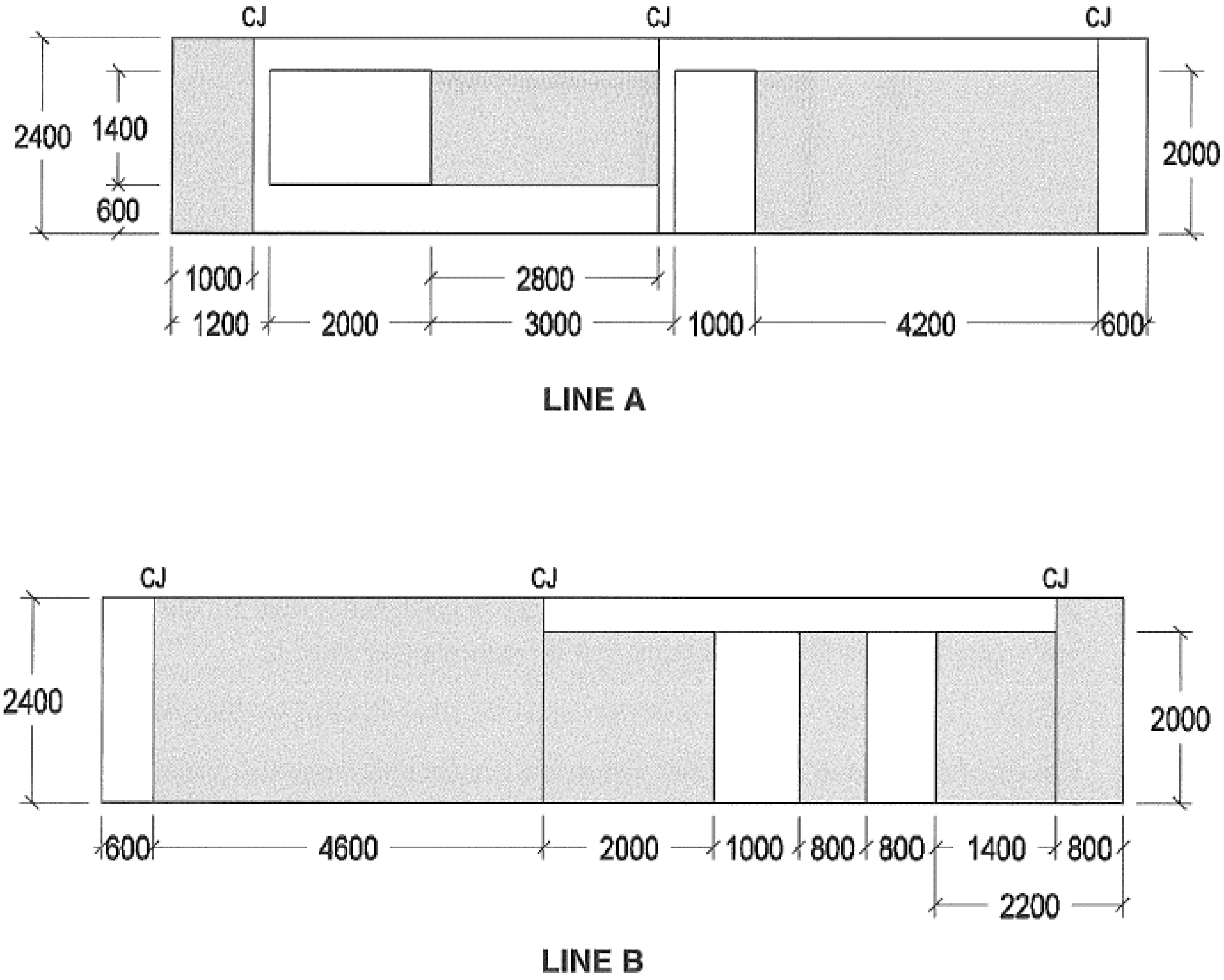


Figure 3.3.3: Bracing Panels for Design Example 3

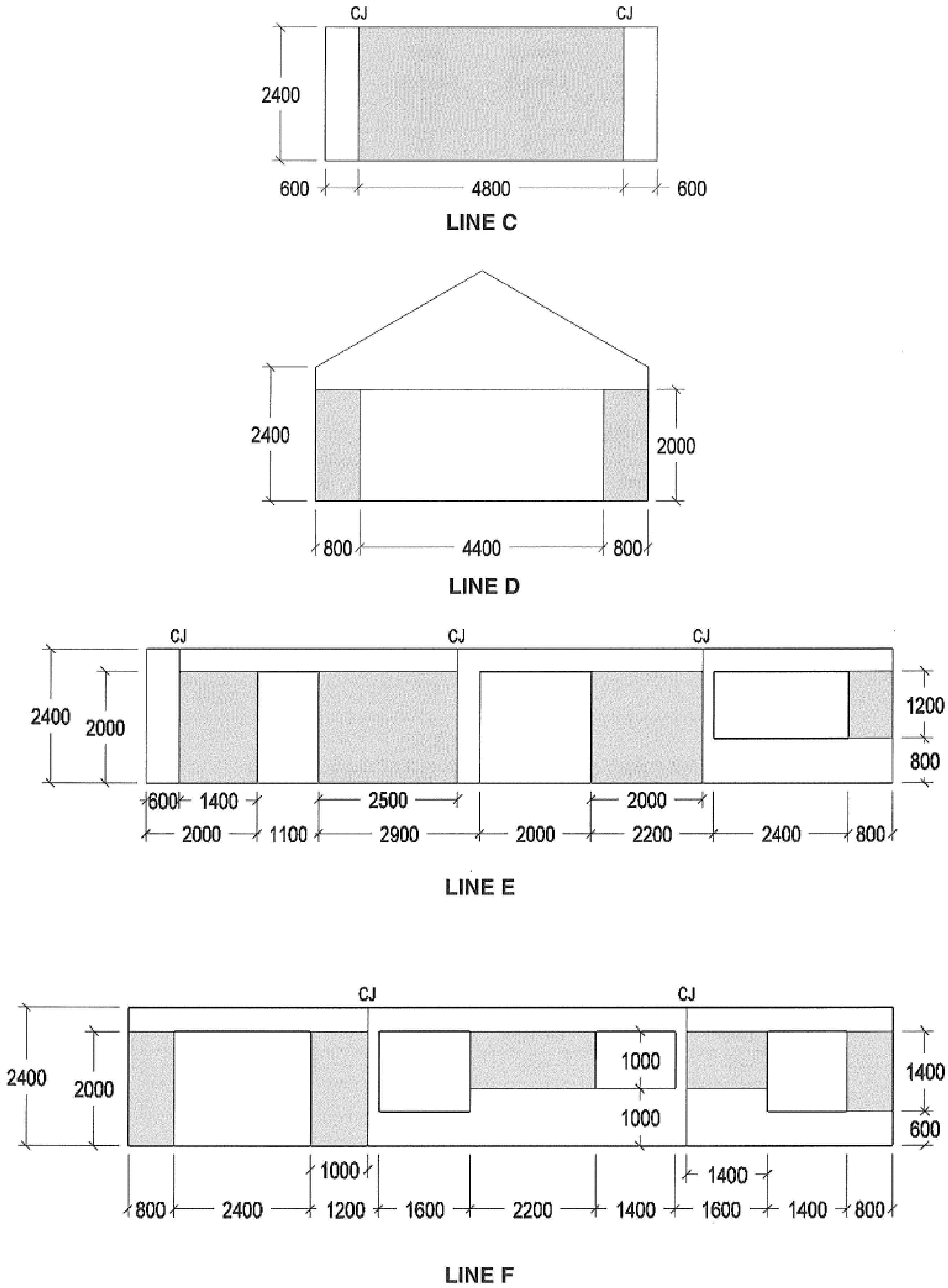


Figure 3.3.3: (Continued) Bracing Panels for Design Example 3

References in NZ 4229	Design Calculations	Design Results

Consider the bracing capacity of bracing line A:

Table 5.1

Table 3.3.2: **Bracing Capacity of Bracing Line A**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
A	2.4	1.0	15	P	206
	1.4	2.8	15	P	1635
	2.0	4.2	15	P	1985
					3828

The total capacity of bracing line A is 3828 BU's, which exceeds the required capacity for this bracing line of 972 BU's.

→ Bracing Line A OK

Bracing Line A OK

Next consider the bracing capacity of bracing line B:

Table 5.1

Table 3.3.3: **Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
B	2.4	4.6	15	P	2023
	2.0	2.0	15	P	680
	2.0	0.8	15	P	180
	2.0	1.4	15	P	393
	2.4	0.8	15	P	155
					3431

From Table 3.3.3 it has been established that the total bracing capacity of bracing line B is 3431 BU's. The required minimum capacity for bracing line B was found in Table 3.3.1 to be 1718 BU's.

→ Bracing Line B OK

Bracing Line B OK

Next check Bracing Line C:

Table 5.1

Table 3.3.4: **Bracing Capacity of Bracing Line C**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
C	2.4	4.8	15	P	2185
					2185

From Table 3.3.4, bracing line C has a capacity of 2185 BU's, while Table 3.3.1 indicates that bracing line C has a demand of 746 BU's.

→ Bracing Line C OK

Bracing Line C OK

Now checking panels aligned in the opposite direction:

References in NZ 4229 | *Design Calculations* | *Design Results*

Table 5.1

Table 3.3.5: **Bracing Capacity of Bracing Line D**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
D	2.0	0.8	15	P	180
	2.0	0.8	15	P	180
					360

From Table 3.3.1 bracing line D requires 367 BU's. However, from Step 1B the earthquake demand is 17 BU/m^2 and from Figure 3.3.2 the size of diaphragm 3 is 36 m^2.

8.8.4 Consequently, bracing line D has a capacity of: $\frac{360}{(17 \times 36)} = 59\%$ of the total demand on diaphragm 3. This is permissible if the opposite bracing line (bracing line E) can support 100% of the demand from diaphragm 3. The demand on bracing line E then becomes

Demand = $0.6 \times (28\,\text{BU/m}^2 \times 13.4\,\text{m} \times 6\,\text{m}) + (17\,\text{BU/m}^2 \times 6\,\text{m} \times 6\,\text{m})$ = 1718 BU's

Now checking the capacity of bracing line E:

Table 5.1

Table 3.3.6: **Bracing Capacity of Bracing Line E**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
E	2.0	1.4	15	P	393
	2.0	2.5	15	P	995
	2.0	2.0	15	P	680
	1.2	0.8	15	P	275
					2343

Clearly both bracing lines D and E have sufficient capacity, as the capacity of bracing line E (2343 BU's) is in excess of the re-evaluated demand (1718 BU's).

→ Bracing lines D and E OK

Bracing lines D and E OK

Now checking the final bracing line:

Table 5.1

Table 3.3.3: **Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
F	2.0	0.8	15	P	180
	2.0	1.0	15	P	243
	1.0	2.2	15	P	1450
	1.0	1.4	15	P	713
	1.4	0.8	15	P	245
					2831

From above bracing line F has a total capacity of 2831 BU's, while Table 3.3.1 indicates a demand of 1351 BU's.

→ Bracing line F OK

Bracing Line F OK

References in NZ 4229	Design Calculations	Design Results
	Step 2E – Conclusion	
	In Step 2D it has been shown that in all cases, bracing capacity exceeds bracing demand. Note also that as the capacity of each wall must exceed60% of the total demand, the sum of all wall capacities exceeds the total demand on the structure.	
	→ Building has sufficient bracing capacity.	Bracing capacity OK
	Step 3 – Determine wall reinforcement	
5.3.1	All walls have partial filled 15 series masonry.	All wall rebar D12 @ 800 Vertical
Table 8.2	→ Vertical reinforcement is D12 @ 800 mm	
	Horizontal reinforcement is D16 @ 2300 mm	
	Note that the Standard specifies maximum spacing of horizontal reinforcing of 2800 mm. However, as the lower storey is only 2.4 m high, no horizontal reinforcement is required other than that for lintels and bond beams.	
	Step 4 – Mid floor timber diaphragms design	
	As shown in Figure 3.3.2, diaphragm 1 has dimensions of 6 m x 6 m, diaphragm 2 has dimensions of 7.4 m x 6 m, and diaphragm 3 has dimensions of 6 m x 6 m. Therefore all diaphragms have a length less than 16 m, and less than twice their width.	
	→ Diaphragm dimensions OK	Floor diaphragm dimensions OK
9.3.2 Table 9.2 9.3.3 Figure 9.1 Figure 9.5	Use plywood floor diaphragm not less than 18 mm thick, nail fixed with 60 mm long, 2.8 mm diameter flat head nails spaced at 150 mm centres, with construction as shown in Figure 9.1 of the Standard. Note that details of the diaphragm to wall connection are shown in Figure 9.5 of the Standard.	
	Step 4A – Check diaphragm opening	
9.5.1.2(b)	Diaphragm 1 has an opening with dimensions of 2.0 m x 2.6 m. This is outside the allowable size specified in 9.5.1.2 of the Standard. The diaphragm opening therefore requires specific engineering design.	Diaphragm opening requires specific design
	→ Diaphragm opening requires specific engineering design.	
	Step 5 – Bond Beam Design	
	Structural mid-floor diaphragm system being used (see Step 2B and Step 4).	Bond beam depth 200 mm, rebar 1-D16
10.3.1 10.3.2	→ Bond beam reinforcement 1-D16, with a bond beam depth of 200 mm	
	Step 6 – Lintel Design	
	Step 6A – Determine weight of roof	
Figure 6.1	From Figure 3.3.1 the roof span is 7.6m,with a lightweight roof	Roof weight 3.8 kN/m
	→ Contributing weight = 3.8 kN/m	

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 6B – Determine weight of walls above lintel	
Table 6.1	Upper storey is timber frame plus lightweight timber cladding. Upper storey height is 2.4 m	Upper storey wall weight 1.4 kN/m
	→ Contributing weight = 2.4 x 0.6 = 1.44 kN/m	
	Step 6C – Determine weight of suspended floor	Floor weight 8 kN/m
Figure 6.1	Timber floor with maximum span of 6 m.	
	→ Contributing weight = 8 kN/m	
	Step 6D – Establish total load on lintel	
	Total weight on lintel in two storey structure = 3.8 + 1.4 + 8 = 3.2 kN/m	Total lintel weight 13.2 kN/m
	Total weight on lintel in single storey structure (garage) = 3.8 kN/m (same roof span as 2 storey structure)	
	Step 6E – Lintel reinforcement	
	From Figure 3.3.2 it may be seen that all lintels are 390 mm deep, with the longest lintel span below the two storey structure being 2.4m and the longest lintel span in the single storey structure being 4.4 m. Note that as roof trusses in the lower storey structure run parallel with the 4.4 m lintel, this lintel is effectively not loaded, other than by its own self weight.	
Table 11.2(a)	→ All lintels 2-D12 with R6 stirrups @ 200 c/c	Lintel rebar 2-D12 with R6 @ 200 c/c
	Note however that lintels may make advantage of the 1-D16 from the bond beam as shown previously for design example 1 in Figure 3.1.4.	
	Step 7 – Footing Design	
	Step 7A – Determine wall weights	
Table 6.1	Building located in Rotorua (see Note 9).	
	Assume block density of 1850 kg/m^3 partial filled with grout. Nominal wall thickness 150 mm.	
	→ Factored unit weight of wall 1.9kN/m	Weight of wall 4.6 kN/m
	Wall height 2.4 m	
	→ Weight of wall 1.9 x 2.4 = 4.6 kN/m	

References in NZ 4229	Design Calculations	Design Results
	Step 7B – Consider wall axial load at foundation	Load on footing 17.8 kN/m
	Total weight on footing is weight of roof, plus weight of upper-storey wall, plus weight of mid-height diaphragm, plus weight of lower storey wall.	
	→ Weight on footing = 3.8 + 1.4 + 8 + 4.6 = 17.8 kN/m	
	As in design example one the load on the foundation in wall lines E & F is greater than that of the gable ended walls and is therefore used to determine requirements for footing design.	
	This weight is well below the limiting wall capacity of 68kN/m reported in Table 8.1 of the Standard.	Wall axial load capacity OK
	→ Wall axial load capacity OK	
Table 6.2	*Step 7C – Detail footing*	
	Weight on footing is 17.8 kN/m.	
	Therefore:	Footing width 300 mm depth 200 mm steel 2-D12, with R6 stirrups @ 600
Figure 6.5	Footing { width 300 mm { depth 200 mm { steel 2-D12, with R6 stirrups @ 600 c/c	
	See Figure 6.5 of the Standard for the footing cross-section detail.	

3.4 Design Example 4

References in NZ 4229	Design Calculations	Design Results

Design example 4 is a two-storey house with a concrete first floor and a lightweight roof, located in Invercargill on sub-soil Class D. The upper and lower storeys are constructed of 15 series concrete masonry external walls (see Note 2, page 7) and internal timber partitioning. There is also one internal masonry wall in both upper and lower storeys. A two-bay internal garage is located on the lower storey. Details of the house are shown in Figure 3.4.1.

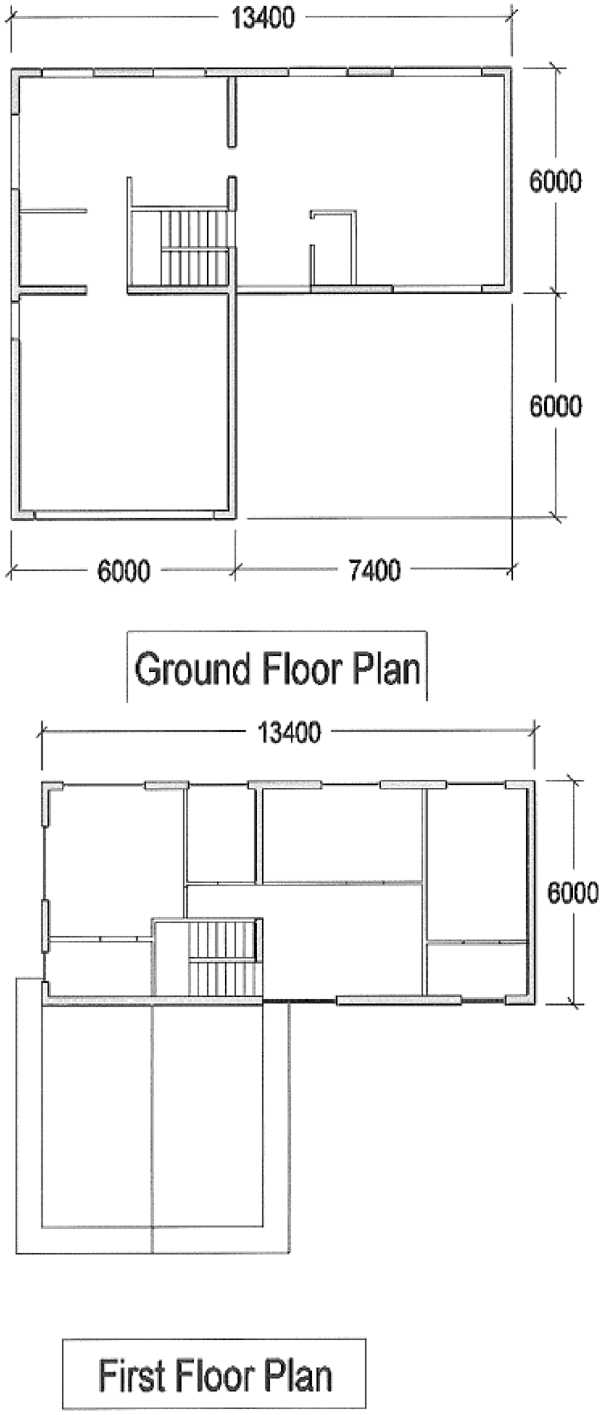


Figure 3.4.1: Design Example 4

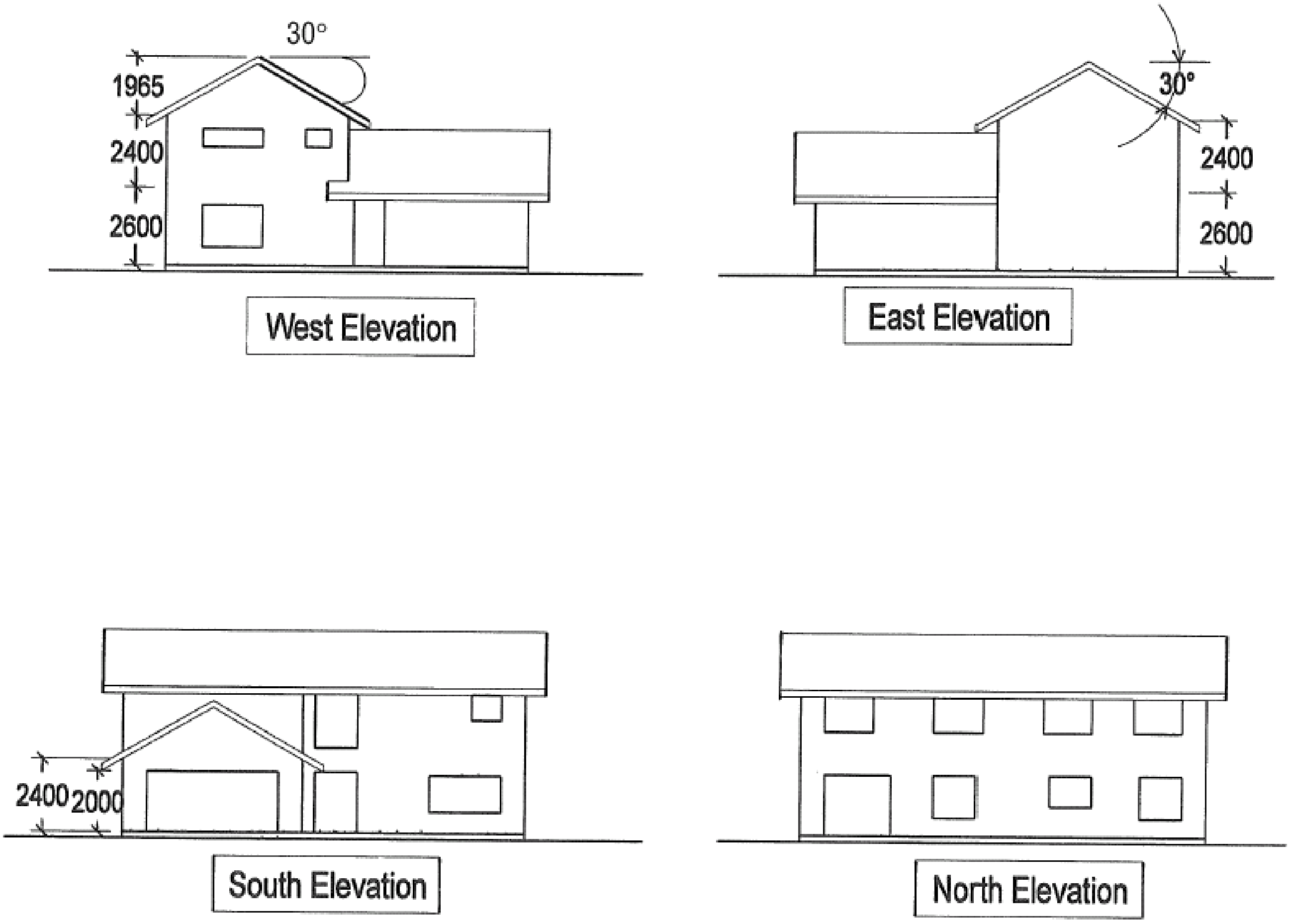


Figure 3.4.1: (Continued) Design Example 4

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 1 – Bracing Demand Evaluation (see Note 3)	
	Step 1A – Determine Earthquake Zone	
Figure 4.1 or Table 4.1	House located in Invercargill.	
	→ Earthquake Zone 1	EQ Zone 1
	Step 1B - Determine Bracing Unit demand for earthquake loading (see Note 4)	
	Two-storey building with 15 series partially filled masonry for both storeys, having a lightweight roof and an intermediate concrete floor. Building located in earthquake Zone 1 on sub-soil D.	
Table 4.3	→ Demand on upper storey and lower storey garage = 11 BU/m^2	EQ demand on upper storey 11 BU/m^2
	→ Demand on lower storey (excluding garage) = 51 BU/m^2	EQ demand on lower storey 51 BU/m^2
	No correction factor required for sub-soil Class D.	

References in NZ 4229	Design Calculations	Design Results
	Step 1C – Determine total bracing demand for earthquake loading	
	Plan area of upper storey = 13.4 x 6 = 80.4 m^2	
	Plan area of lower storey = (13.4 x 6) + (6 x 6) = 80.4 + 36 = 116.4 m^2	
Figure 1.1	As the plan area is less than 250 m^2, the building falls within the scope of NZS 4229.	EQ demand on upper storey 884 BU's
	→ EQ demand on upper storey = 80.4 x 11 = 884 BU's	EQ demand on lower storey 4496 BU's
	→ EQ demand on lower storey = (80.4 x 51) + (36 x 11) = 4100 + 396 = 4496 BU's	
Table 4.2	*Step 1D – Determine Bracing Unit demand for wind loading (default EH)*	
	For single storey garage and upper storey, storey height = 2.6 m, roof height = 2 m, and height to apex ≤10 m. Use H5/h2.	Single storey wind demand 80 BU/m across 88 BU/m along ridge
	→ Demand across ridge = 80 BU/m	
	→ Demand along ridge = 88 BU/m	
Table 4.2	Two storey structure, storey height less than 3 m, roof height 2 m, and height to apex 7 m. Use H7/h2.	Two storey wind demand 152 BU/m across 160 BU/m along ridge
	→ Demand across ridge =152 BU/m	
	→ Demand along ridge = 160 BU/m	
	Step 1E – Determine total bracing demand for wind loading	
	For wind in N-S direction, longest wall is 13.4 m across ridge (two storey)	
	→ Total demand on upper storey only = 80 x 13.4 = 1072 BU's	Total wind demand N-S 2144 BU's
	→ Total demand on lower storey = 160 x 13.4 = 2144 BU's	
	For wind in E-W direction, longest wall along ridge (two storey) is 6 m, longest wall across ridge (single storey) is 6 m	
	→ Total demand on upper storey only = 88 x 6 = 528 BU's	Total wind demand E-W 1440 BU's
	→ Total demand on lower storey = 160 x 6 + 80 x 6 = 1440 BU's	
	Step 1F – Determine total bracing demand	
	For upper storey:	
	Earthquake demand = 884 BU's	Design demand for upper storey 1072 BU's
	Maximum wind demand = 1072 BU's	
	→ Wind loading governs for upper storey	
	Where wind loading dominates significantly, then reference to detailed wind loading in NZS 3604 is recommended.	

References in NZ 4229	*Design Calculations*	*Design Results*

For lower storey:

Earthquake demand = 4496 BU's

Maximum wind demand = 2144 BU's

→ Earthquake loading governs

Design demand for lower storey 4496 BU's

Step 2 – Determine Bracing Capacity of Upper Storey

Step 2A – Determine bracing lines, location of shrinkage control joints (see Note 5) and bracing panels for upper storey

Bracing lines for the upper storey are shown in Figure 3.4.2, with individual bracing panels identified in Figure 3.4.3. Of particular note when considering the bracing panel geometry in the upper storey of bracing line C is that the wall is defined as terminating at the top of the bond beam, rather than the full height of the gabled wall.

As detailed in Note 5, page 11, vertical control joints should be placed at any change of wall height exceeding600mm.This requirement specifically demanded the placement of control joints at the changes in wall heights on bracing lines A and B.

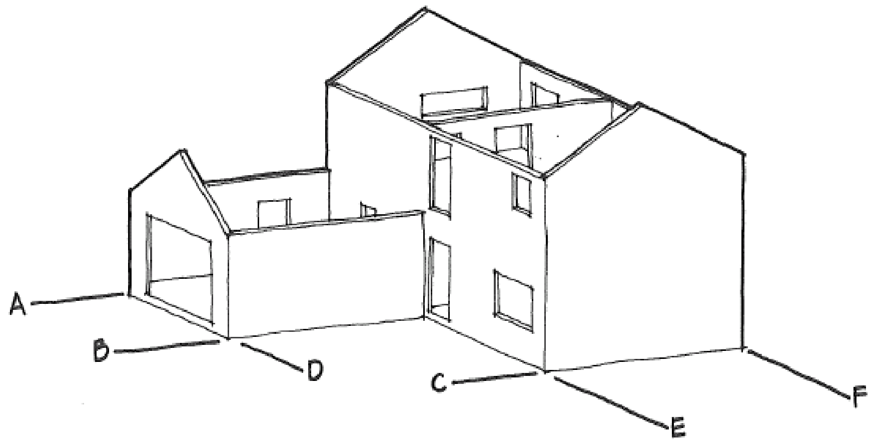


Figure 3.4.2(a): Bracing Lines for Design Example 4

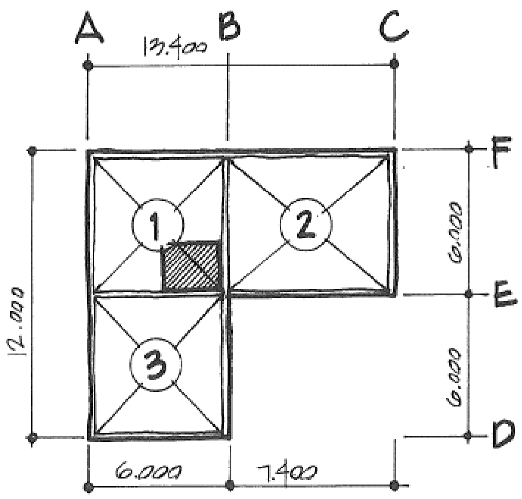


Figure 3.4.2(b): Bracing Lines Diaphragm Positions

References in NZ 4229	Design Calculations	Design Results

Table 10.1 — *Step 2B – Method of Bracing*

Table 8.3 — From Figure 3.4.2 it may be seen that the longest bond beam span in the upper storey is 8.0 m, on bracing lines E and E From Table 10.1 of the Standard the longest permissible bond beam span for a single (upper) storey 15 series partially filled wall in earthquake Zone 1 is 8.0 m. This information is similarly detailed in Table 8.3.

→ Bracing line method may be used

Design Result: Bracing line method to be used

8.6.1, 8.7.4 — *Step 2C – Determine required minimum capacity of individual bracing lines (see Note 10)*

As detailed in Note 11, page 12, when using the bracing line method individual bracing lines must be able to support a minimum tributary area. The required capacity is calculated below in Table 3.4.1 based on an earthquake demand of 11 BU/m^2 (see Step1B), and wind loading of 80 BU/m across ridge and 88 BU/m along ridge (see Step 1D).

Table 3.4.1: **Individual Bracing Line Demands for Upper Storey of Design Example 4**

Bracing Line	Wall Length	Perpendicular Wall Length	EQ Demand	Wind Demand	Design Demand
A	6 m ext.	3	198	240	240
B	6 m int.	6.7	442	536	536
C	6 m ext.	3.7	244	296	296
D	13.4 m ext.	3	442	264	442
E	13.4 m ext.	3	442	264	442

Step 2D – Determine bracing capacity of each bracing line

Consider the upper-storey bracing capacity of bracing line A (see Figure 3.4.3)

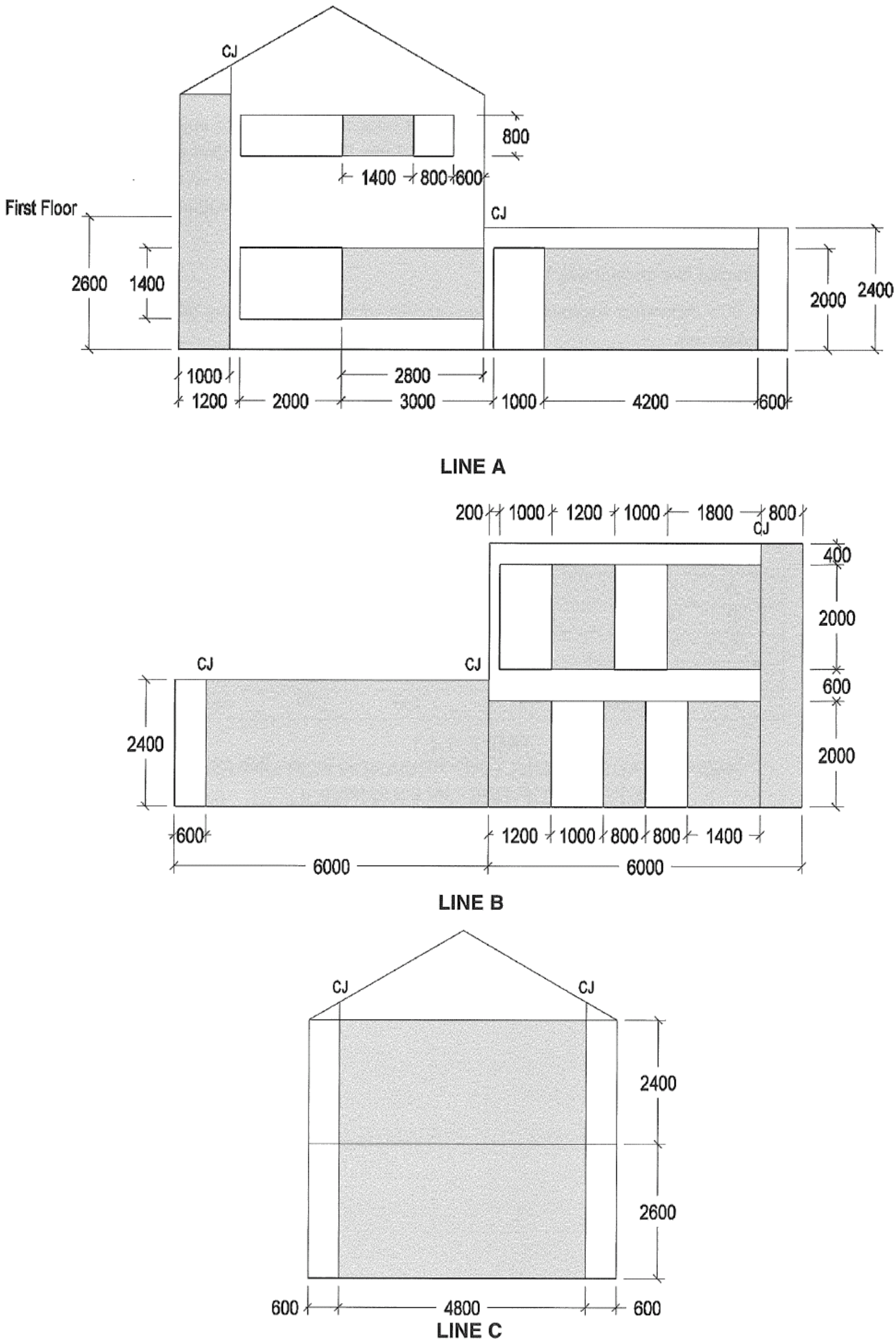


Figure 3.4.3: Bracing Panels for Design Example 4

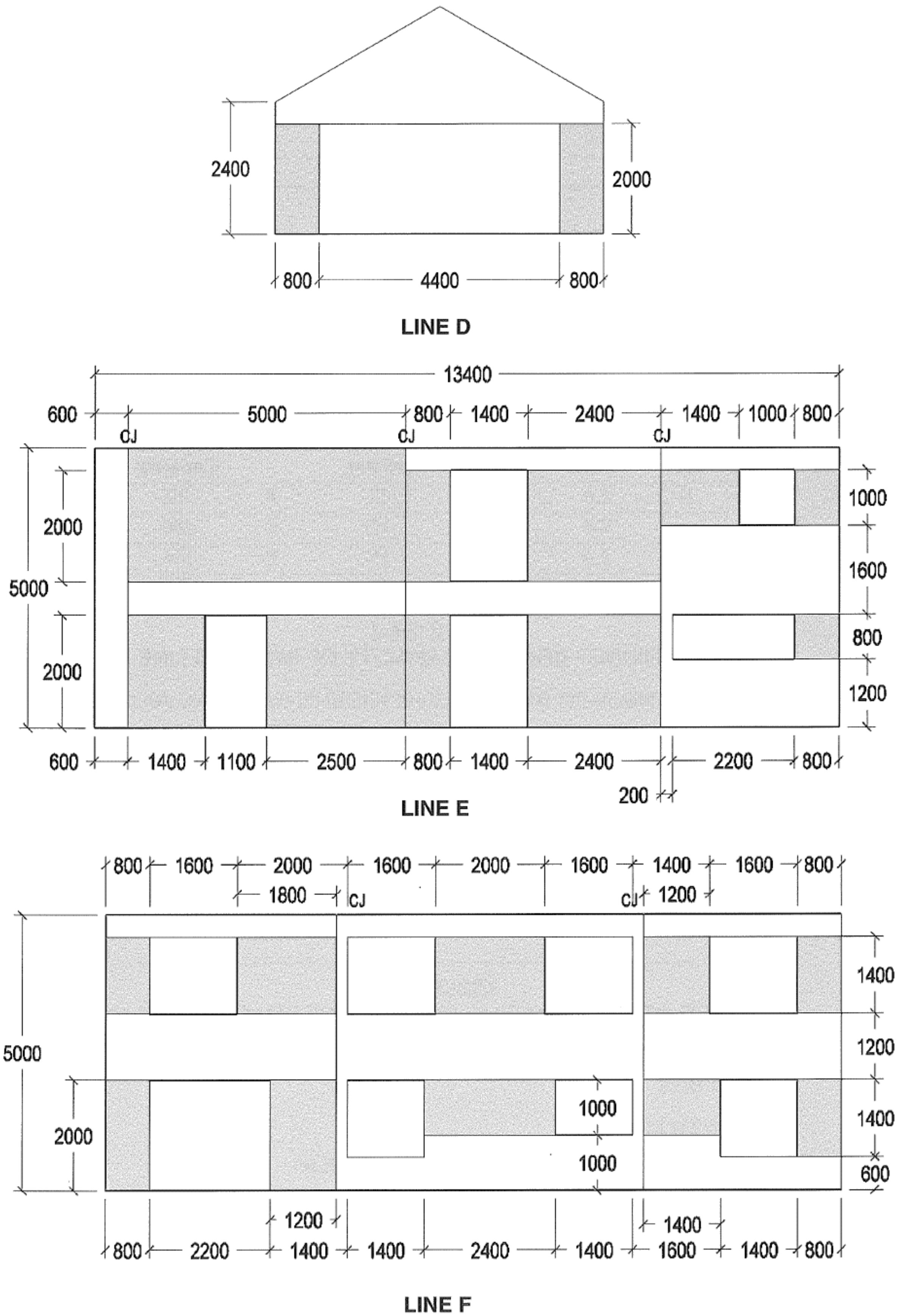


Figure 3.4.3: (Continued) Bracing Panels for Design Example 4

References in NZ 4229	Design Calculations	Design Results

Table 5.1

Table 3.4.2: **Upper Storey Bracing Capacity of Bracing Line A**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
A	2.4	1.0	15	P	208
	0.8	1.4	15	P	828
					1036

From above, bracing line A has a capacity of 1036 BU's, exceeding the demand of 240 BU's.

→ Bracing Line A OK

Bracing Line A OK

Next considering the upper-storey bracing capacity of bracing line B:

Table 5.1

Table 3.4.3: **Upper Storey Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
B	2.0	1.2	15	P	305
	2.0	1.8	15	P	580
	2.4	0.8	15	P	155
					1040

From above, bracing line B has a capacity of 1040 BU's, exceeding the demand of 563 BU's.

→ Bracing Line B OK

Bracing Line B OK

Next considering the upper-storey bracing capacity of bracing line C:

Table 5.1

Table 3.4.4: **Upper Storey Bracing Capacity of Bracing Line C**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
C	2.4	4.8	15	P	2185
					2185

From above, bracing line C has a capacity of 2185 BU's, exceeding the demand of 296 BU's.

→ Bracing Line C OK

Bracing Line C OK

The upper-storey bracing capacity of bracing line E is:

References in NZ 4229	Design Calculations	Design Results

Table 5.1

Table 3.4.5: **Upper Storey Bracing Capacity of Bracing Line E**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
E	2.4	5.0	15	P	2730
	2.0	0.8	15	P	180
	2.0	2.4	15	P	925
	1.0	1.4	15	P	713
	1.0	0.8	15	P	330
					4878

From above, bracing line E has a capacity of 4878 BU's, exceeding the demand of 442 BU's.

→ Bracing Line E OK

Bracing Line E OK

Finally, the upper-storey capacity of bracing line F is:

Table 5.1

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
F	1.4	0.8	15	P	245
	1.4	1.8	15	P	790
	1.4	2.0	15	P	930
	1.4	1.2	15	P	420
	1.4	0.8	15	P	245
					2630

From above, bracing line F has a capacity of 2630 BU's, exceeding the demand of 442 BU's.

→ Bracing Line F OK

Bracing Line F OK

Step 2E – Conclusion

In Step 2D it has been shown that in all cases, bracing capacity exceeds bracing demand. The sum of bracing lines A, B and C (oriented in the N-S direction) is 4006 BU's, and the sum of bracing lines E and F (oriented in the E-W direction) is 7508 BU's. In both cases, the summed capacities exceed the total demand of 1072 BU's (see Step1F).

→ Upper storey has sufficient bracing capacity.

Upper storey bracing capacity OK

Step 3 – Determine wall reinforcement for upper storey

5.3.1 All walls have partial filled 15 series masonry.

→ Vertical reinforcement is D12 @ 800 mm

Table 8.2(a) Horizontal reinforcement is D16 @ 2300 mm

All wall rebar D12 @ 800 Vertical

Note that the Standard specifies maximum spacing of horizontal reinforcing of 2800 mm. However as the upper storey is only 2 m high, no horizontal reinforcement is required other than that for lintels and bond beams.

References in NZ 4229	Design Calculations	Design Results
	Step 4 – Upper storey ceiling diaphragm design	
	As the bracing line method is being used for the upper storey, an upper storey ceiling diaphragm is not required.	Ceiling diaphragm not required
	Step 5 – Bond Beam Design for upper storey	Bond beam depth 390 mm, rebar 2-D16 (bracing lines B, E and F)
	Bracing line method is used for upper storey. Building in earthquake Zone 1 with maximum spacing between bracing lines of 8.0 m.	
	→ Bond beam reinforcement 2-D16 with a bond beam depth of 390 mm B2	
10.2.1	Note that bracing lines A and C are gable shaped walls (see Note 8 page 12).	B2 bond beam at top of gable shaped walls B3 bond beam at top of wall (bracing lines A and C)
Table 10.1	For these walls a racking bond beam Type B2 shall be provided at the top of the gable, having a span of 6.9 m. As explained in Note 8, an intermediate bond beam Type B3 is required at the top of bracing lines A and C, immediately beneath the gable shaped wall (see Figure 2.5).	
	Step 6 – Lintel Design for upper storey	
	Step 6A – Determine weight of roof	Roof weight 3.8 kN/m
	From Figure 3.4.1 the roof span is 7.8 m, with a lightweight roof	
Figure 6.1	→ Contributing weight – 3.8 kN/m	
	Step 6B – Lintel reinforcement	
	From Figure 3.4.1 it may be seen that all lintels are 390 mm deep, with the longest lintel span in the upper storey between 2.0 m.	
Table 11.2(a)	→ All lintels 2-D12 with R6 stirrups @ 200 c/c	Lintel rebar 2-D12 with R6 @ 200 c/c
	Note however that lintels may make advantage of the 2-D16 from the bond beam.	
	The R6 stirrup centres are closed from 600 mm centres to 200 mm for the length of each lintel.	
	Step 7 – Determine Bracing Capacity of Lower Storey Walls	
	Step 7A – Determine bracing lines, location of shrinkage control joints (see Note 5) and bracing panels for lower storey	
	Most bracing lines for the lower storey correspond with those of design example 3, with the exceptions being bracing lines B, E and F. The bracing panels within these lower storey bracing lines of design example 4 are shown in Figure 3.4.3.	
	Step 7B – Method of Bracing	
Table 10.1	From Figure 3.4.2 it may be seen that the longest bond beam span in the lower storey is 7.4 m, on bracing lines E and F. Table 10.1 of the Standard permits a maximum span of 7.0 m for two storey partial filled 15 series masonry in earthquake Zone 1. Therefore the diaphragm method must be used.	Diaphragm method to be used

References in NZ 4229	Design Calculations	Design Results

Step 7C – Determine required minimum capacity of individual bracing lines

8.8.2
8.8.4
Figure 8.4

Having established in step 7B that a structural diaphragm is required, minimum requirements for individual walls are given in Clauses 8.8.2 and 8.8.4. In general it is required that each individual wall be capable of supporting 60% of the total demand on the adjoining diaphragm, although this may be reduced to 30% if the opposite wall can be shown to support 100% of the total demand. Note however that when two diaphragms are connected to a common wall, as shown in Figure 8.4 of the Standard, then the maximum bracing value of that wall shall not be less than 40% of the sum of the requirements of both diaphragms. This applies here to bracing line B and bracing line E. This information is shown in Table 3.4.6 based on the presence of three adjoining diaphragms as shown in Figure 3.3.2. Individual demands are then based on 60% of the earthquake demand for the lower storey (51 BU/m^2) and the dimensions of diaphragm 1 (36 m^2) and diaphragm 2 (44.4 m^2) and the earthquake demand for the lower storey garage (11 BU/m^2) and the dimensions of diaphragm 3 (36 m^2).

Table 3.4.6: **Individual Bracing Line Demands for Lower Storey of Design Example 4**

Bracing Line	Adjoining Diaphragm	Individual Demand (BU's)
A	0.6 (D1 + D3)	1339
B	0.4 (D1 + D2) + 0.6 (D3)	1878
C	0.6 (D2)	1342
D	0.6 (D3)	238
E	0.4 (D1 + D3) + 0.6 (D2)	2251
F	0.6 (D1 + D2)	2460

Step 7D – Determine bracing capacity of each bracing line

Recognising that bracing lines A, C and D of design example 4 correspond with those of design example 3, Table 3.3.2 shows that bracing line A has a capacity of 3828 BU's, exceeding the demand listed in Table 3.4.6 of 1339 BU's.

→ Bracing Line A OK

Bracing Line A OK

Similarly, from Table 3.3.4 bracing line C has a capacity of 2185 BU's exceeding the demand of 1342 BU's and from Table 3.3.5 bracing line D has a capacity of 360 BU's exceeding the demand of 238 BU's.

→ Bracing Lines C and D OK

Bracing Lines C and D OK

Now checking the capacity of bracing line B:

Table 5.1

Table 3.4.7: **Lower Storey Bracing Capacity of Bracing Line B**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
B	2.4	5.4	15	P	2718
	2.0	1.2	15	P	305
	2.0	0.8	15	P	180
	2.0	1.4	15	P	393
	2.4	0.8	15	P	155
					3751

References in NZ 4229	Design Calculations	Design Results

From Table 3.4.7, the capacity of bracing line B (3751 BU's) exceeds the demand (1878 BU's).

→ Bracing Line B OK

Bracing Line B OK

Next checking the capacity of bracing line E:

Table 5.1

Table 3.4.8: **Lower Storey Bracing Capacity of Bracing Line E**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
E	2.0	1.4	15	P	393
	2.0	2.5	15	P	995
	2.0	0.8	15	P	180
	2.0	2.4	15	P	925
	1.2	0.8	15	P	275
					2768

From above, the capacity of bracing line E (2768 BU's) exceeds the demand (2251 BU's).

→ Bracing Line E OK

Bracing Line E OK

Finally, checking the capacity of bracing line F:

Table 5.1

Table 3.4.9: **Lower Storey Bracing Capacity of Bracing Line F**

Bracing Line	Panel Height	Panel Length	Size (series)	Fill	Bracing Capacity
F	2.0	0.8	15	P	180
	2.0	1.2	15	P	305
	1.0	2.4	15	P	1670
	1.0	1.4	15	P	713
	1.4	0.8	15	P	245
					3113

From Table 3.4.9, the capacity of bracing line F is 3113 BU's, which exceeds the demand of 2460 BU's.

→ Bracing Line F OK

Bracing Line F OK

Step 7E – Conclusion

From Step 1F the total bracing demand in the lower storey was calculated to be 5408 BU's. In Step 7D it has been shown that each bracing line has sufficient capacity. Furthermore, the sum of the capacities of bracing line A, B and C (oriented in the N-S direction) is 9764 BU's and the sum of the capacities of bracing lines D, E and F (oriented in the E-W direction) is 6241 BU's.

→ Building has sufficient bracing capacity

Bracing capacity OK

References in NZ 4229	*Design Calculations*	*Design Results*
	Step 8 – Determine wall reinforcement for lower storey	
	All walls have partial filled 15 series masonry.	
	→ Vertical reinforcement is D12 @ 800 mm	
	Horizontal reinforcement is D16 @ 2300 mm	All wall rebar D12 @ 800 Vertical
Table 8.2(a)	Note that the Standard specifies maximum spacing of horizontal reinforcing of 2800 however, as the lower storey is only 2.4 m high, no horizontal reinforcement is required other than that for lintels and bond beams.	
	Step 9 – Lower storey ceiling diaphragm design	
	Step 9A – Concrete mid floor diaphragm design	
9.4.1 Figure 9.6	Diaphragms 1 and 2 (see Figure 3.4.2) both have lengths less than three times their width. These diaphragms shall include a minimum amount of steel reinforcement of 665 mesh, and shall be connected to the supporting bracing wall by D16 bars @ 800 mm, as shown in Figure 9.6 of the Standard.	Concrete diaphragm to have 665 mesh and connect to wall using D16 @ 800 c/c
	The design of the concrete floor for strength and serviceability is not covered by NZS 4229:1999.	
9.4.2	The opening in diaphragm 1 will be the subject of specific engineering design.	
	Step 9B – Timber ceiling diaphragm	
9.5.2	Diaphragm 3 is a horizontal timber ceiling diaphragm having dimensions of 6 m x 6 m.	
9.1.2 9.2.2	Use plywood ceiling diaphragm not less than 6 mm three ply, nail fixed with 30 mm long 2.5 mm diameter flat head nails spaced at 150 mm centres into framing member at sheet edges, with construction as shown in Figure 9.1 of the Standard.	Concrete diaphragm opening requires specific design
Figure 9.1	**Step 10 – Bond Beam Design for lower storey**	
	Structural ceiling diaphragm system being used (see Step 7B and Step 9).	Bond beam depth 200 mm rebar 1-D16
	→ Bond beam reinforcement 1-D16, with a bond beam depth of 200 mm.	
	Step 11 – Lintel Design for lower storey	
10.3.1 10.3.2	From Step 6 the weight of the roof is 3.8 kN/m.	
	Step 11A – Lintel reinforcement	
	Upper storey is 15 series partially grouted concrete masonry. Building located in Invercargill (see Note 9, page 12). Upper storey height is 2.4 m.	
Table 6.1	→ Contributing weight = 2.4 x 2.5 = 6.0 kN/m	Upper storey wall weight 6 kN/m
	Step 11B – Determine weight of suspended floor	
	Concrete suspended floor with maximum span of 6 m. Assume lightweight composite steel and concrete floor.	
Figure 6.8	→ Contributing weight = 20k N/m	

References in NZ 4229	Design Calculations	Design Results

Step 11C – Establish total load on lintel

Total weight on lintel in two storey structure = 3.8 + 6 + 20 = 29.8 kN/m

Total weight on lintel in single storey structure = 3.8 kN/m

Design result: Total lintel weight 29.8 kN/m

Step 11D – Lintel reinforcement for lower storey

From Figure 3.4.3 it may be seen that all lintels are 390 mm deep, with the longest lintel span below the two storey structure being 2.2 m and the longest lintel span in the single storey structure being 4.4 m.

For the lintels in the lower storey of the two-storey structure the following reinforcement requirements apply.

Table 11.2(a)

Table 3.4.10: **Lintel Reinforcement for Design Example 4**

Span	Horizontal Bars	Stirrups
2.2 m	2-D16	R6 @ 50 c/c
2.0 m	2-D16	R6 @ 50 c/c
1.4 m	2-D12	R6 @ 150 c/c
1.1 m	2-D12	R6 @ 150 c/c
1 m or less	2-D12	R6 @ 200 c/c

For the lintel with 4.4 m span located on bracing line D (garage opening), note that the roof trusses run parallel with the 4.4m lintel, this lintel is effectively not loaded other than by its own self weight.

Lintel reinforcement 2-D12 R6 stirrups @ 200 c/c

Note however that lintels may make advantage of the 1-D16 from the bond beam acting as the top bar of the lintel.

Step 12 – Footing Design

Step 12A – Consider wall axial load at foundation

Total weight on footing is weight of roof, plus weight of upper storey wall, plus weight of mid-height concrete diaphragm, plus weight of lower storey wall.

→ Weight on footing= 3.8 + 6 + 20 + 6 = 35.8 kN/m

Design result: Load on footing 35.8 kN/m

Table 8.1

This weight is well below the limiting wall capacity of 68 kN/m reported in Table 8.1 of the Standard.

→ Wall axial load capacity OK

Step 12B – Detail footing

Weight on footing is 36 kN/m. Therefore:

Table 6.2

Footing { width 300 mm
{ depth 200 mm
{ steel 2-D12, with R6 stirrups @ 600 c/c

Figure 6.5

See Figure 6.5 of the Standard for the footing cross-section detail.

Design result: Footing: width 300 mm depth 200 mm steel 2-D12, with R6 stirrups @ 600

4.3 Standard Specifications for Concrete Masonry Buildings Not Requiring Specific Design

Contents

Introduction

The development of a Standard Specification is inevitably an attempt at consensus on the various specifications in current use. The assistance of several consultants who provided copies of their specifications is acknowledged. JASMAX were engaged to prepare an amalgamated specification which was subsequently circulated. In response to comments, the specification was amended by the CCANZ, who also provided the commentary clauses, drawn essentially from their Information Bulletin IB 61.

1.0 Preliminary

1.1 NZS 4229:2013 Concrete Masonry Buildings Not Requiring Specific Design

The specification sets out the requirements for the construction of buildings not requiring specific design where the foundations and some or all the walls in any storey are constructed in masonry and the floors, ceilings and roofs are constructed in light timber frame construction, with the exception that ground floors can be slab-on-grade.

COMMENTARY: *A considerable amount of masonry construction does not have each element specially designed in detail. This is known as the Non Specific Design approach. Tables and rules are set out to enable a building to be detailed without the need for structural calculations. NZS 4229 sets out these rules for a limited set of buildings. These are typically shown in Figure 1.*

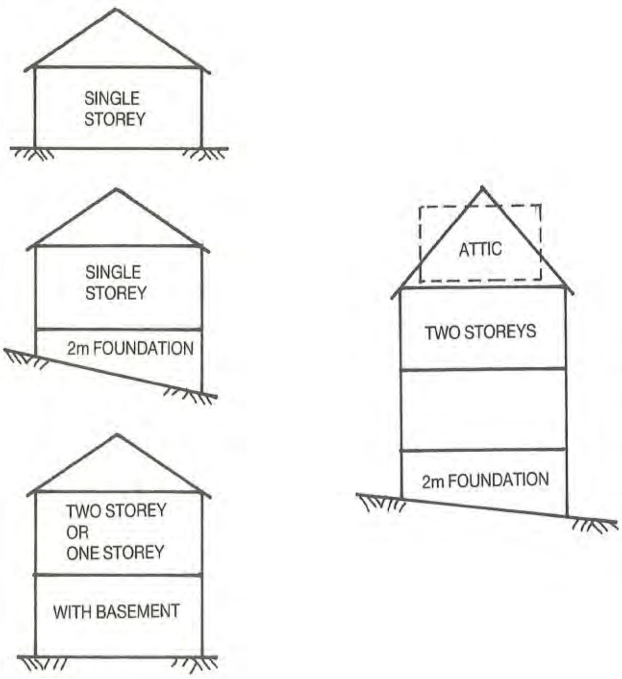


Figure 1: *Examples of designing to NZS 4229 are shown in Section 4.2 'Reinforced Masonry'*

Mixed construction is included in masonry lower walls with timber wall construction above. Timber floors are used throughout except a concrete slab-on-ground can be used.

1.2 Conditions of Contract

The work carried out by the contractor or sub-contractors under his control under this specification shall be subject to all the applicable provisions of the General and Special Conditions of Contract, the Preliminary and General sections of the Contract Documents.

COMMENTARY: *In non-specific design work there may be no professional adviser. In these cases, a client may consider using New Zealand Standard NZS 3902:2004 "Housing, alterations and small buildings contract". Where a professional advisor is used, then normally NZS 3910:2003 "Conditions of contract for building and civil engineering* *construction" provides a standard form of conditions of contract.*

1.3 Scope

(a) Work covered by this specification and contract documents include:

(b) Supply, delivery and placing of all structural masonry.

(c) Supply, delivery, cutting, bending and placing of steel reinforcing to cavities and cells as shown on the drawings or as specified in NZS 4229.

(d) Grouting of walls to be as shown on the drawings or as specified in NZS 4229.

(e) Building in all the necessary fittings and fixings as required.

COMMENTARY: *Because NZS 4229 contains a significant number of building details, it is possible that contract drawings may reference these details without full repeating the information. The mason will need, therefore, to hold a copy or have access to NZS 4229.*

1.4 Co-operation

All work shall be carried out with full co-operation between the Contractor and all other sub-contractors.

Any allowance for co-ordination shall be included in the tender or any negotiated agreements.

COMMENTARY: *Often the masonry will be built under a Sub-contract arrangement. This means that the positioning of starter reinforcement for walls is the General Contractor's responsibility. Co-operation with the General Contractor in assisting with starter bar positioning is highly desirable. A CCANZ technical bulletin IB 47 goes someway to explaining how to set out the starter bars. It should be noted that locations of windows/doorways affect the position of starter bars.*

1.5 Standards

Unless otherwise specified or otherwise shown on the drawings all masonry shall be constructed in strict accordance with:

- NZS 4229:2013 Concrete masonry buildings not requiring specific design.
- NZS 4210:2001 Masonry Buildings: materials and workmanship.
- AS/NZS 4455.1:2008 Masonry units, pavers, flags and segmental retaining wall units - Masonry units.
- NZS 3109:1997 Concrete Construction.
- AS/NZS 4671:2001 Steel reinforcing materials.
- AS 1478.1-2000 Chemical admixtures for concrete, mortar and grout - Admixtures for concrete.
- NZS 3122:2009 Specification for Portland and blended cements (General and special purpose).
- NZS 3604:2011 Timber-framed buildings.

The documents above, with current amendments, and all related documents cited in those documents, are deemed to form part of this specification. However this specification (which includes contract drawings) takes precedence in the event of it being at variance with the above documents.

COMMENTARY: *The mason should hold up to date copies of NZS 4210 and NZS 4229. Reference to the other documents will be required on a more limited basis.*

Generally masonry work built within the requirements of NZS 3604 will be under a Sub-contract situation and the General Contractor should have available a copy of this document.

1.6 Workmanship and Inspection

Construction of the blockwork shall comply with NZS 4210 and be under the control of a Licensed Building Practitioner – Blocklayer.

The licensed mason shall check the work at critical stages, such as set out, reinforcing, prior and during grouting and ensure that the work is in accordance with NZS 4210, NZS 4229 and all other relevant documents.

COMMENTARY: *For the construction of buildings designed using non-specific design methods, there will usually be no intermediate formal inspections of the work by the Designer although there may be site visits by the Client.*

Inspection requirements for the Local Authority Building Inspector are required. The mason should check these requirements before commencing work and ensure adequate notice for inspection is given.

The responsibility for ensuring that masonry work complies with the drawings and specification lies *with the mason himself. For this reason it is important that competency and knowledge of the building codes is shown by the mason. Hence the requirement that a licensed mason be used.*

The Contractor shall give 24 hours notice to the inspecting mason of the typical critical stages and those the inspection mason will nominate prior to the commencement of the contract.

Masonry incorporating faulty masonry units, faulty mortar, faulty filling concrete, inadequate preparations, incorrect steel (placing and type), unapproved details or work practices, or any other aspects not complying with this specification as disclosed by testing or otherwise shall be rejected.

Any element of the construction that is rejected shall be made good at the Contractor's expense.

The Client, on completion of the work or later, may find a fault which shows a breach in interpretation of drawings, code or specification. In this case there will be redress and a requirement to correct the error. Corrections at this stage can be very costly.

1.7 Contractor's Responsibilities

The Contractor is responsible for ensuring that he can build the structure in accordance with the documents.

Should the Contractor find omissions or errors in the contract documents, then the Contractor shall immediately inform the Client/Designer of these findings. The Contractor will then receive instructions on the course of action to be taken to remedy the situation.

The Contractor shall not proceed with any alternative or remedial details of construction until he has received written instructions from the Client/Designer.

COMMENTARY: *As the mason is often a Sub Contractor, the timing of construction work and its feasibility of construction within the general contract needs to be carefully studied.*

1.8 Site Requirements

Chapter 3 of NZS 4229 shall be followed in every respect. If the Contractor finds that the site does not comply with NZS 4229 or as described in the contract documents then the Contractor shall immediately bring this situation to the Client/Designer's attention.

The Client/Designer will then issue written instructions to clarify that situation.

COMMENTARY: *Chapter 3 of NZS 4229 deals with the aspects of safe bearing pressure, soil types, minimum depth of footing below ground, site clearance and permitted slopes.*

The minimum safe bearing pressure, for example, is 100 kPa.

1.9 Quality Records

The mason shall keep accurate records relating to strength and quality of materials including cleanliness of sands use in the construction, and make available to the Client/Designer or Inspectors when requested.

Records to include:

(a) Proportions of materials used for mortar.

(b) Supplier of grout and control test certificates

(c) Mix proportions for site mixed grout.

(d) Daily spread of grout test values.

(e) Expansive agent type and dosage if used.

COMMENTARY: *Specific project testing of materials is not required. However the materials MUST comply with the minimum requirements set down in the documents and codes.*

It is recommended that the mason, as a competent tradesman, should have compressive strength tests of mortar carried out at regular intervals, particularly if the source of supply of sand is changed. In addition to being a check on the suitability of the mortar mix, such tests serve as a check on the cleanliness of the sand.

2.0 Materials

2.1 Concrete Blocks

All concrete blocks shall comply with AS/NZS 4455 Part 1 and shall be dense, hard, sound and true to size and shape. Any necessary cutting shall be by saw, neat and true to lines.

All blocks shall be delivered to site, stored off the ground and kept clean and dry suitable for their end use.

COMMENTARY: *The concrete blocks should be within ±3 mm height tolerance and ±2 mm for length and width. Minor chipping is acceptable but where the work is to be fair face the blocks supplied should be substantially free of defects. Discuss with the* *block manufacturer **before** laying starts if there are any doubts on the quality of surface finish.*

2.2 Other Masonry

Where other types of masonry unit are to be used as shown on the drawings or as specified in NZS 4229, then these units and their handling shall comply with NZS 4210 in all respects.

COMMENTARY: *If natural sedimentary stone is used then a check is often needed on the bedding direction for laying. The stone needs laying with its natural bedding plane horizontal.*

The reuse of masonry, particularly clay bricks, needs also to be considered with care since weathering and bonding characteristics may be unsatisfactory.

2.3 Cement

Cement shall comply with NZS 3122.

COMMENTARY: *Cement should be stored off the ground and kept protected from the weather. To ensure it is free flowing it should not be stacked more than 1.5 m high and used on a first in first used basis as much as possible.*

2.4 Lime

Building lime shall comply with BS 890.

2.5 Sand

Sand for mortar shall comply with NZS 3103. Sand for grout shall comply with NZS 3121.

2.6 Water

Water shall be fresh clean water and drinking quality for use in concrete, mortar, grout, cleaning out, wetting, working materials and curing.

2.7 Aggregates for Concrete Grout

Fine and coarse aggregates for concrete grout shall comply with NZS 3121.

COMMENTARY: *Note that mortar sands are not generally considered suitable for substitution as a sand (fine aggregate) used for grout infill.*

2.8 Reinforcing Steel

Reinforcing steel shall be grade 300 complying with AS/NZS 4671. All reinforcing shall be of the deformed type, except that ties and stirrups may be of the round bar.

Where noted on the drawings, Grade 500 reinforcing steel will be required.

Steel reinforcement shall be clean, rust scale-free and undamaged.

Steel reinforcement shall be cut, bent and placed for construction in accordance with NZS 4229 or as shown on the drawings.

COMMENTARY: *The identification of steel is illustrated by the photograph below.*

Figure 2: *High strength (500) steel identified by the blank space in the deformations.*

2.9 Admixtures

2.9.1 General

The inclusion of chemical admixtures to assist in construction of the masonry structure shall comply with AS 1478.1.

COMMENTARY: *Traditionally, a small quantity of hydrated lime was used to improve the consistency of the mix. Today, chemical admixtures may sometimes be more convenient to use than lime. It is very important to ensure that you are measuring correct dosages for your mortar mix. Failure to comply with the manufacturers instructions can lead to problems of low strength in the mortar.*

Some workability aids entrain air and unless there are careful adjustments to the mortar a significant loss in strength can occur. Excessive mixing time can cause more air entrainment and loss of strength.

Final comments:

1. *Make absolutely sure you are using the correct dosage rate.*
2. *Do not mix admixtures without receiving clearance from the manufacturers.*
3. *Make some mortar cylinders from time to time to check strength levels.*

2.9.2 Expansive Agent for Concrete Grout

Proprietary aluminium powder-based agents for the grouting method as detailed in Section 12.1 of this specification shall be used in strict accordance with NZS 4210 and the manufacturer's instructions.

COMMENTARY: *It is important to use the gas forming expansive admixtures which cause an expansion of the grout in its plastic state. Expansion agents that cause expansion of the hardened grout are unsatisfactory for use in masonry.*

Also, if the grout is to be strength tested using standard steel moulds, the grout sample must be taken BEFORE the expansive admixture is added.

3.0 Mortar

Mortar shall comply with NZS 4210.

COMMENTARY:

Mix Proportions

Satisfactory mortars commonly fall within the following cement to sand proportions:

	Parts by Volume	
	Cement	***Sand***
For all reinforced masonry and all work below ground	*1*	*3-4.5*

To produce mortar of consistently the right quality, the volumes of materials should be measured using buckets or gauge boxes and not shovelled direct from the stock pile or cement bag into the mixer.

There is a difficulty in volume measuring sand because sand "bulks" up in volume depending upon the moisture content. Bulking of 15% is often used as a general guide. This really means that if you are batching to 1:3 cement/sand by volume you need 3.5 volumes of damp sand to each volume of cement.

Always keep a dry bucket reserved for measuring cement quantities and make sure you are measuring the correct amounts.

Materials should be thoroughly mixed in a mechanical mixer for a minimum of five minutes. Hand mixing of say three to four builder's buckets is permitted.

If a batch of mortar, not containing a special retarding admixture, has not been used with 1½ hours of adding the cement, it should be discarded.

Retempering mortar within this time may be done by adding water to a basin shape formed in the mortar and thoroughly remixing by hand or returning the mortar to the mixer for retempering, water adjustment and remixing.

Retempering by just dashing water over the mortar is not permitted.

Workability

The water must be added carefully from a measured container and not directly from the end of a hose pipe. For a mortar to be satisfactory, it must have the correct workability which must be produced consistently by uniform batching and mixing.

In wet weather, the amount of water in the sand may be significantly higher, consequently you will need less added water to bring the mortar to the correct consistency. Where colour matching of mortar is extremely important, particularly for example where pigments have been used, it is recommended that the sand stockpile be covered to reduce moisture variations.

Each mason will have a way of telling whether the mortar is of the right consistency. One test is - scoop up mortar on the trowel, flick the trowel turning it over to see if the mortar will remain on the trowel face – see photo. If the mortar falls off, it is too wet or it may be too dry.

For speed of laying, avoiding significant extrusion of wet joints and therefore risk of marking faces and increased cleaning of cells, it pays to get the mortar absolutely right. The man on the mixer has a very important job.

Coloured Pigments

Coloured mortars can be produced using mineral oxide pigments. Dosage of pigments must not exceed 6% by weight of cement.

It has been reported that additions of over 3% with some pigments may cause a reduction in bond strength.

Careful batching and uniform curing are essential if uniformity in colour between batches is to be *achieved. Premixing of pigment and cement helps with uniformity.*

3.1 Mortar Strength

Mortar shall have a minimum 28 day compressive strength of 12.5 MPa.

COMMENTARY: *Details of the testing method Specific Design Specification.*

4.0 Concrete Grout

Coarse Grout (in terms of NZS 4210).

COMMENTARY: *For most structural masonry grout infills, "coarse" grout will be used.*

4.1 Blockwork Cavities

Grout for filling blockwork cavities shall comply in all respects with NZS 4210.

4.2 Compressive Strength

Specified minimum 28 day compressive strength equals 17.5 MPa or 20 MPa or 25 MPa.

COMMENTARY*: The specified strength of grout may be 17.5, 2.0 or 25 MPa. The specified strength depends upon the durability zone the building is in:*

Zone B:	*17.5 MPa*
Zone C:	*20 MPa*
Zone D:	*25 MPa*

Only 17.5 MPa can be produced on the construction site. 20 and 25 MPa must be supplied by ready mixed concrete plants working to NZS 3104.

If grout is to be sampled to test for compressive strength, the sample must be taken BEFORE any expansive admixture is added.

4.3 Spread

Spread Value = 450-530 mm

COMMENTARY: *Grout must have good flowing characteristics in order to fill the cells. This flowing requirement makes it different to wet concrete. A simple test is used to determine the flowing characteristics and is called a spread test.*

NZS 4210 requires a spread test each day that grout filling is taking place.

An inverted slump cone is used, being filled without rodding and the lifted 50 mm allowing the grout to flow out. The diameter of the spread of grout is measured at two positions at right angles to one another. The average of these measurements is the spread value.

For concrete masonry the spread should be within the range 450 to 530 mm. It is important to be within the range, neither below it nor above. Grouts should be checked for this characteristic before commencing placement in the wall.

CCANZ Bulletin IB 50 explains in more detail the spread test.

4.4 Maximum Size of Aggregate/Sand

Maximum aggregate size = 13.2 mm

COMMENTARY: *When grout spaces are narrow, less than 60 mm, then a grout consisting of concreting sand and cement must be used. Normally this would only apply to the 100 series hollow masonry or when a cavity less than 60 mm is to be filled between two masonry skins.*

An admixture may also be used.

Coarse grouts contain a proportion of aggregate sizes, usually in the range 4.57 mm to 9.5 mm, but on occasion sizes up to 13.2 mm can be used although this would be uncommon. Segregation of the larger particle sizes becomes a significant risk when placing the grout material.

4.5 Expansive Admixture

Expansion prior to initial set = 2-4%

COMMENTARY: *The structural integrity of any grouted walls is improved by using a gas forming expansive agent as specified in NZS 4210.*

The expansive agent must be dosed at site immediately prior to placing, to ensure that the expansion takes place within the masonry cells. The dosage is specified by the manufacturer to give between 2-4% increase in volume before the grout sets.

Do not use expansion agents that cause hardened concrete to expand.

The principal advantages of using an expansive agent within the grouting method are:

(i) the reduction in delay times resulting from consolidation in grout lifts of 1.2 m;

(ii) better compaction and expulsion of free water; and

(iii) better face shell bonding

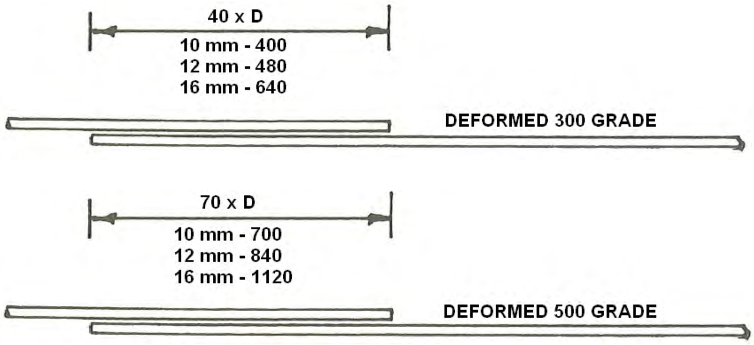


Figure 3: *Lapping details*

4.6 Delivery and Manufacture

4.6.1 Off the Site (Ready Mixed)

Grout supplied ready mixed from off the site shall comply with the relevant provisions of NZS 3104.

4.6.2 At the Site

17.5 MPa grout can be manufactured at the site providing it complies fully with NZS 4210.

COMMENTARY: *While normal supplies would probably be ordered from a ready mixed concrete supplier, it is possible to site mix 17.5 MPa grout. 20 and 25 MPa site mixing is not permitted.*

Note: *You must use a concreting sand not the builders' sand you are using to produce the mortar.*

A typical mix for a 50 litre 17.5 MPa mix is:

1½ buckets of cement
4 buckets of concreting sand
2 buckets of 4.75 mm to 9.5 mm coarse aggregate.

The added water will vary with the moisture contents of the sand/aggregate, but should be adjusted to bring the grout within the specified spread range of 450-530 mm for concrete masonry.

Mixing time should be not less than 1½ minutes. Larger mixes will require a longer time for mixing. Generally all grouts should be placed within 1½ hours of mixing unless retarding admixtures have been used.

5.0 Reinforcement Details

Unless shown on the drawings reinforcement details shall be as required by NZS 4229 and NZS 4210. Any reinforcement details or requirements not specifically covered by NZS 4229 shall comply with those of NZS 3109.

5.1 Laps in Reinforcement

All lapping reinforcement, including starter bars, shall be securely fixed together. See Figure 3.

5.2 Placement and Cover to Reinforcement

All reinforcement, including starter bars, shall be carefully set out to suit masonry modules and required locations within cells and cavities, in accordance with the drawings or details as specified in NZS 4229.

The minimum cover to reinforcement shall be:

Zone B	45 mm
Zone C	50 mm
Zone D	60 mm

Reinforcement, including starter bars, shall be set out to the tolerances specified in NZS 4210.

COMMENTARY:

Position Tolerances

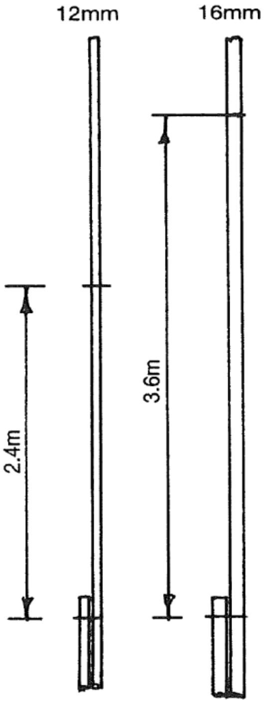
While 6 mm is the minimum dimension to a face shell, the overall cover from the outer face of the shell will vary depending on durability zone, i.e. 45 mm for Zone B, 50 mm for Zone C and 60 mm for Zone D.

Generally the requirements are that reinforcing steel should be prefixed with the block layer using open ended units as appropriate, e.g. typically Series 20.05 for partial fill walls and 20.16 for solid fill walls supplemented by other units as appropriate. It is recognised that on occasions it may be necessary in special circumstances to fix vertical steel after blocklaying. However, access to the starter bar is still mandatory since the vertical steel must be tied to the starter bar.

In order to ensure that the reinforcement remains in place during the grouting operation that follows, the steel must be adequately tied. First, it must be tied to the starter bars, and then it must be adequately and securely fixed as shown in Figure 4.

Note that when 12 mm bars were used no intermediate tie or spacer is required for the usual wall heights.

When the Client/Designer has not specified the position of the vertical bar, it is assumed that it is to be placed in the centre of the cell itself.



Horizontal steel should also be positively located by tying to stirrups or to vertical bars and must be at least 25 mm above or below the adjoining mortar joint.

The other important matter concerning steel reinforcement is that it must have sufficient concrete cover to ensure long term durability.

5.3 Starter Bars

The Client/Designer shall immediately be notified of any misalignment of cast-in starters or any other type of cast-in reinforcing. The Client/Designer will give written instructions to remedy the misalignment. Reinforcement shall not be bent on site to correct alignment.

COMMENTARY: *The starter bars (Figure 6) should be checked for position BEFORE laying the first course. If they are incorrect, then it will be necessary to cut off bars, drill holes to a significant depth and grout in new steel with epoxy mortar.*

Double cranking of steel is totally unacceptable, see diagram. ***Note that starter bar positions are influenced by the windows in the wall.***

More details are found in CCANZ Bulletin IB 45.

6.0 Concrete Base

6.1 Initial Preparation

The concrete base (footing, foundation, wall, beam, slab, etc.) that is intended to support a masonry structure shall be built in accordance with NZS 4210 and NZS 4229.

COMMENTARY: *The first step requires that the foundations be checked for line, level and dimension and that starter bars are in the correct position.*

Tolerances for acceptance are governed by NZS 4210 and override NZS 3109 tolerances for construction.

Any discrepancies which would cause the bed joint to exceed 20 mm in thickness must be corrected. It should be noted that the minimum thickness is laid down as 7 mm for structural masonry or 4 mm for veneer construction.

Any concrete surface lying outside the tolerance limits shown in Figure 5 (page 9) needs correction before commencing the work of laying.

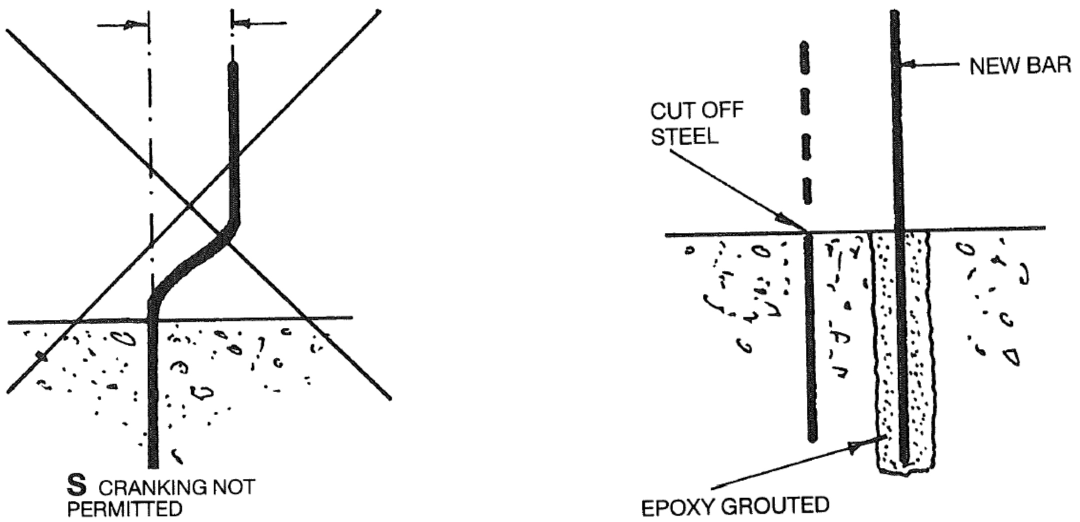


Figure 4: *Starter Bars*

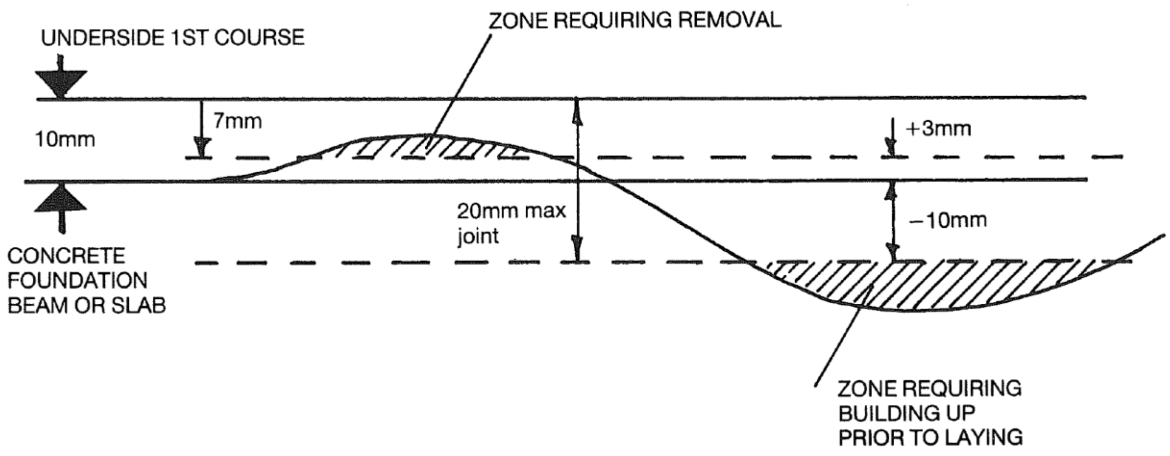


Figure 5: *Concrete base – initial preparation*

6.2 Foundation Surface

The Contractor shall ensure that the surface on which masonry is to be laid is free of all laitance, loose aggregate and any material that could reduce the bond between the masonry elements and the concrete base.

COMMENTARY: *The image below depicts the appearance of a cleaned horizontal foundation concrete ready to receive the first course of masonry.*

Note: *This texture is achieved by washing and brushing the surface of the concrete approximately 12 hours after placing.*

7.0 LAYING THE UNITS

7.1 Tolerances

Laying of masonry units shall be in accordance with NZS 4210. All masonry units shall be laid in mortar in courses, true to line, plumb, and level to the tolerances of NZS 4210, unless otherwise specified.

COMMENTARY: *The tolerances from NZS 4210 follow below.*

The concrete masonry units themselves should receive a visual inspection for defects prior to laying and should be laid air dry. However, some dampening of the surfaces to be bonded is often desirable.

Table of Recommended Tolerances for Masonry Construction from NZS 4210

Item	***Tolerances***
Deviation from the position shown on plan for a building more than one storey in height	*15 mm*
Deviation from vertical within a storey	*10 mm per 3 m of height*
Deviation from vertical in total height of building	*20 mm*
Relative displacement between load-bearing walls in adjacent storeys intended to be in vertical alignment	*5 mm*
Deviation from line in plan:	
(i) in any length up to 10 m	*5 mm*
(ii) in any length over 10 m	*10 mm total*
Deviation of bed joint from horizontal:	
(i) in any length up to 10 m	*5 mm*
(ii) in any length over 10 m	*10 mm total*
Average thickness of bed joint, cross joint, or perpend	*±3 mm on thickness specified*

7.2 Laying of Blocks

Blocks shall be laid in straight uniform courses with running bond unless specified otherwise on the drawings. Lay units with a consistent joint thickness throughout.

The moisture state of masonry units at the time of laying shall be as set out in NZS 4210. Upon completion of each day's work, exposed tops of blockwalls shall be protected from the weather.

COMMENTARY: *Concrete blocks should not be laid saturated, since the subsequent drying out will result in a considerable amount of shrinkage movement leading to cracks in joints and, possibly, the units themselves.*

Hollow masonry units should have a mortar bed for the face shells and should have all head joints filled to the same depth. It is also important to realise that when concrete masonry is only to be partially filled, certain cross webs to close a cell must be mortar jointed during the laying process, if loss of grout is to be avoided.

Solid masonry should have all joints completely filled and light furrowing of the bed joint is permitted to assist bedding.

7.3 Flashings

When flashings are specified these shall be built in to masonry in such a manner as not to weaken the structure. Joints around the flashings shall be made good as necessary to ensure waterproofness.

COMMENTARY: *Grooves to receive flashings generally should not be deeper than 6-10 mm. Once the flashing is placed the groove will need to be caulked. The jointing material may be elastomeric type or rigid type and this should be specified on the drawings.*

7.4 Weep Holes

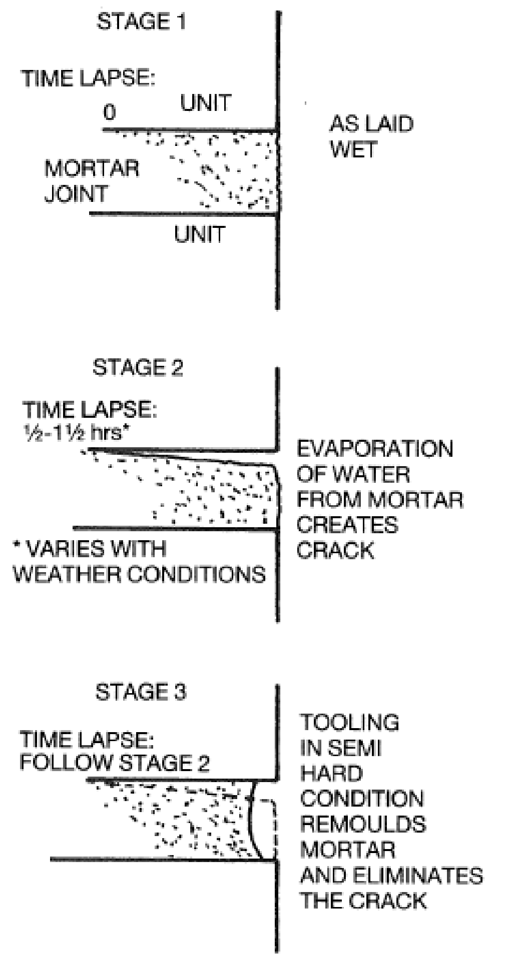
Weep holes in ungrouted hollow masonry walls (above grade) shall be provided at the bottom of cells as necessary to drain moisture to the outside air.

COMMENTARY: *In partially grout filled masonry walls it is desirable to provide a drainage outlet at the bottom of the unfilled cell in the form of a 5 mm hole through the bed joint into the cell cavity.*

7.5 Mortar Joints

Mortar joints shall be burnished smooth and concave to a depth not exceeding 6 mm after the initial stiffening has occurred, unless otherwise specified.

COMMENTARY: *Tooling of mortar joints after initial hardening is essential to achieve a* ***weatherproof wall****. Tooling to a depth of 6 mm is permitted. There consolidation of the front of the joint after initial water loss is seen as being vital to ensure a water tight joint. See diagram below.*



Often tooling is carried out too early. The mortar should be "thumb nail hard" before tooling.

After tooling and when mortar is sufficiently set the work should be lightly brushed down with a soft bristle brush to remove any particles of mortar sticking to the block face. Rubbing the face of blockwork with a piece of masonry will also remove any "dags" of mortar.

8.0 Clean-out

Clean out details and requirements shall comply fully with NZS 4210.

8.1 Clean-out Openings

Temporary clean-out openings shall be provided and shall be of an adequate size and number to ensure satisfactory cleaning out and inspection as required by the specification. When constructed, clean-out openings shall be located at starter bar positions along the length of the first courses in all walls.
Face shells shall be braced and used to close the temporary opening when a fair face finish is required.

COMMENTARY: *In the first course it is necessary to provide clean out positions of at least 100 x 75 mm dimensions at each reinforcement position. For solid filled walls the use of an inverted 20.16 unit in the first course facilitates the subsequent cleaning out immediately prior to grouting. To reduce the risk of mortar droppings adhering to the base concrete, a layer 10-15 mm thick of sand is laid after placing the first course. This together with droppings is subsequently cleaned out.*

In fair face work the sawn face shells can either be braced in position to resist the grout pressure or a recess created by a piece of timber formwork suitably braced and the face shells mortared into position after the grout filling.

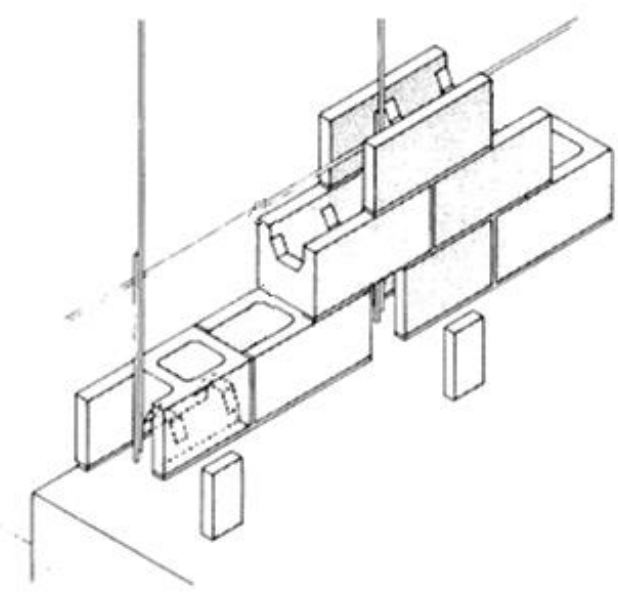


Figure 6: *First course clean-out pockets*

8.2 Cleaning Out

Grout spaces shall be cleaned out before any grout is poured.

COMMENTARY: *After laying up the wall, it is necessary to clean out the bottom of cells to be grouted through the clean-out openings.*

This can be done using a compressed air/water jet or by physically raking with a steel bar followed by washing out. If a sand bed was used it will be relatively easy to remove any debris with a water jet.

Once the base surface is clean, the clean out openings can be sealed by a timber shutter or by mortaring in a piece of face shell. However, in either case adequate bracing will be needed to prevent a blow out under grout pressure.

9.0 Temporary Bracing

The Mason shall be responsible for providing adequate temporary bracing to all masonry to resist any lateral loads until such time that the structure can support the masonry without any distress to either element.

COMMENTARY: *A point which can often be overlooked is the need for temporarily bracing the concrete masonry during construction.*

An ungrouted wall is very susceptible to failure from strong winds. Typically, a wall over one metre in height is at significant risk. See diagram below.

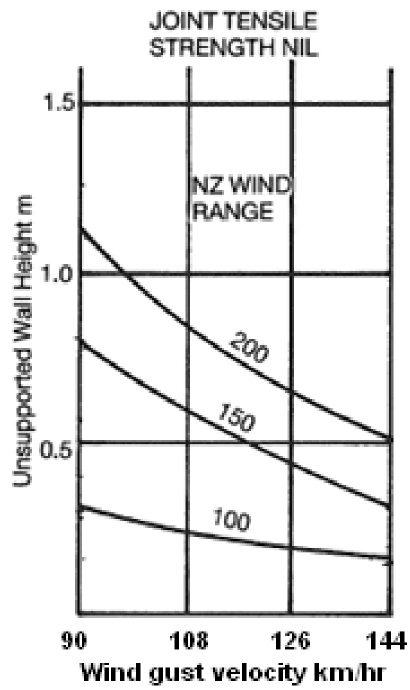


Figure 7: *Maximum unsupported height of ungrouted masonry during construction*

It is important to take some measures to brace the wall in order to prevent its premature failure. Typically bracing at 3 metre centres is recommended in line on both sides of the wall.

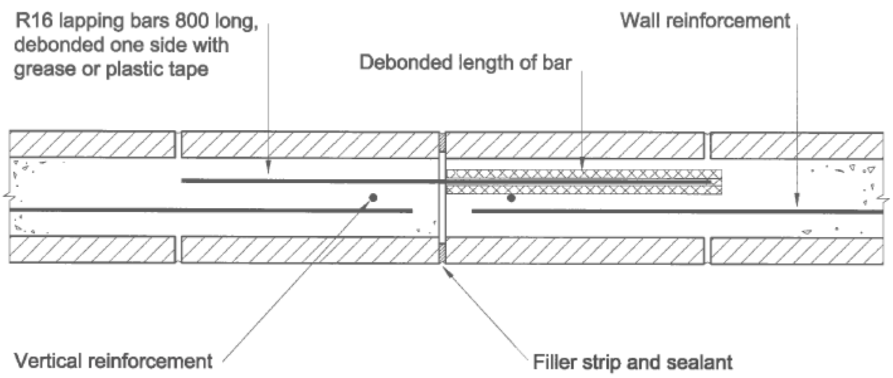


Figure 8: *Control Joint Detail*

10.0 Control Joints

10.1 Spacing

Spacing of vertical control joints shall not exceed 6 metres. Control joints shall be constructed in every respect to the details and requirements of NZS 4229 and NZS 4210, unless otherwise specified on the drawings.

COMMENTARY: *Where there has been no specific design - control joints should be provided at spacings not exceeding 6 m but the final position is influenced by:*

(i) Major changes in wall height.
(ii) Changes in wall thickness.
(iii) Position of control joints in foundations, etc.
(iv) Chases or recesses for services.
(v) Wall intersections.
(vi) Rear return angles in L, T and U shaped situations;
(vii) At one or both sides of openings.

Ideally, no reinforcement should pass through the joint, but it is impracticable in New Zealand, due to earthquake forces. Accordingly, it is usual to de-bond any reinforcement passing through the joints for a certain distance on either side of the control joint. See Figure 8.

Concrete masonry in common with other building materials, is subject to some movements caused by changes in moisture content and temperature.

Since masonry generally will have a high moisture content initially, it loses moisture with time and consequently shrinks. It is usual to primarily design concrete masonry for shrinkage control joints.

10.2 Water and Fireproofing

The Contractor is responsible for ensuring that the control joints are waterproofed and fireproofed, as necessary, with appropriate fillers and sealers.

COMMENTARY: *Control joints will always be undergoing some movement and it is necessary on exterior walls to form a suitable detail to allow for the sealing of the joint against weather penetration. This often takes the form of a 10 mm wide raked back vertical joint. Note that a sealant bond breaker should be used at the back of the joint.*

11.0 Inspection Prior to Grouting

The Mason shall inspect the wall prior to grouting as a final check that the works comply with the drawings/specifications and with special reference to:

(a) Cleanliness of cell for grouting.

(b) Correct positioning and tying of reinforcement.

(c) Sealing of all clean out holes and positions that would cause the loss of grout.

COMMENTARY: *There is often no formal requirement for inspection by the Designer prior to grouting with buildings using the non-specific design code.*

There may be requirements by the Territorial Authority, see Clause 1.6, and following a check by the mason the Territorial Authority may require pre-notification of the intention to grout the wall.

12.0 Grouting of Cells and Cavities

12.1 Grouting Methods

Grouting shall be in accordance with NZS 4210 and the following methods are acceptable:

(a) High lift grouting with expansive admixture.
(b) High lift grouting with reduced compaction.
(c) Low lift grouting.

__COMMENTARY:__ No structural masonry wall in New Zealand is complete without the grouting of reinforcement into the wall either by individual cell or the whole wall.

As discussed earlier, all cells should be cleaned out and clean out ports sealed before grouting commences. There are a number of different grouting methods available depending upon whether the construction work is to be inspected by the Designer. In unsupervised construction as defined in NZS 4230 as design grade B the following methods are acceptable:

(i) High lift grouting with expansive admixture.
(ii) High lift grouting with reduced compaction.
(iii) Low lift grouting.

Note that all work based on the requirements of NZS 4229 is design grade B. For convenience the methods are presented in illustrated form in Information Bulletin IB66 which is part of the Masonry Manual. For convenience method (i) is shown on page 14.

A licensed building practitioner must be able to grout to any of the three methods. On occasions the mason may be able to choose an appropriate method, on others the method may well be specified.

Note that mechanical vibration is a requirement of Method (ii) and an option in Method (i).

A fourth method which is “High Lift Grouting Without Expansive Admixture” and is used only on Specific Design projects, requires mechanical vibration for reconsolidation of grout.

12.2 Extent of Grouting

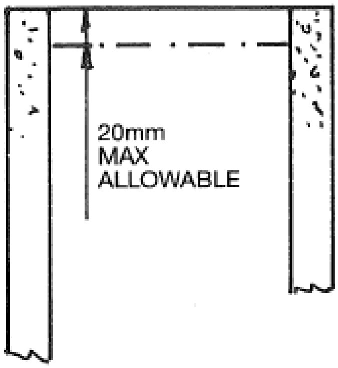
The extent of grouting, whether partial filling or solid filling, shall be as shown on the drawings. Where specific details and localities of grouting are not provided then the grouting shall follow NZS 4229 in all respects.

13.0 Construction Joints

13.1 Horizontal Construction Joints

Horizontal construction joints, when required, shall be formed at the top of the uppermost masonry units. In no case shall they be formed more than 20 mm below the top of the units.

__COMMENTARY:__ Construction joints may be required between different masonry wall lifts. A horizontal construction joint will occur on the top of the uppermost masonry unit. The level of the construction joint, however, should not be lower than 20 mm from the top of this unit. See below.

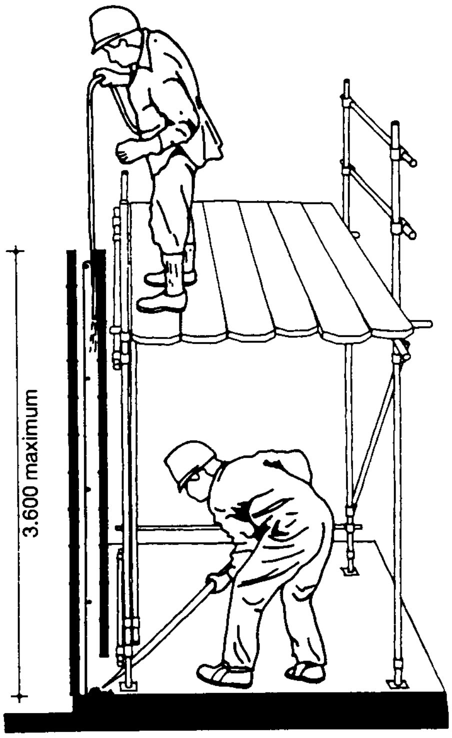


13.1 Preparation of Joints

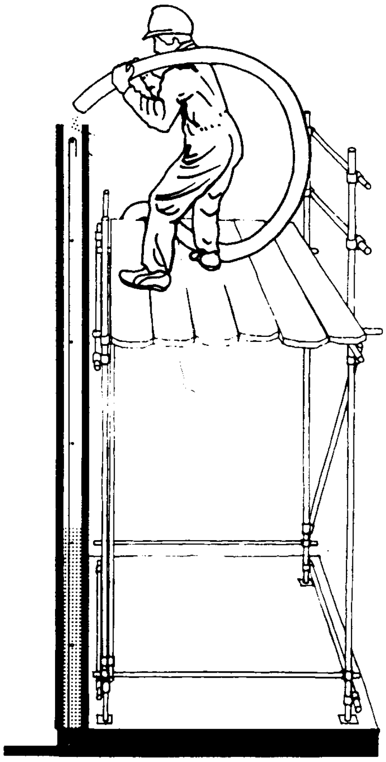
Horizontal construction joints shall have all laitance, loose and foreign matter removed, surface dampened down, prior to the next grouting operation.

__COMMENTARY:__ The horizontal construction joint should be roughened to remove laitence and any loose matter lying on the surface of the hardened grout. This is a similar process to the preparation of the construction joint between the masonry wall and its supporting concrete beam or foundation. It is generally easy to wash and brush the joint a few hours after the grout has hardened to provide a clean surface ready for the next lift. This provides a "sand paper" type of finish to the grout surface. See below.

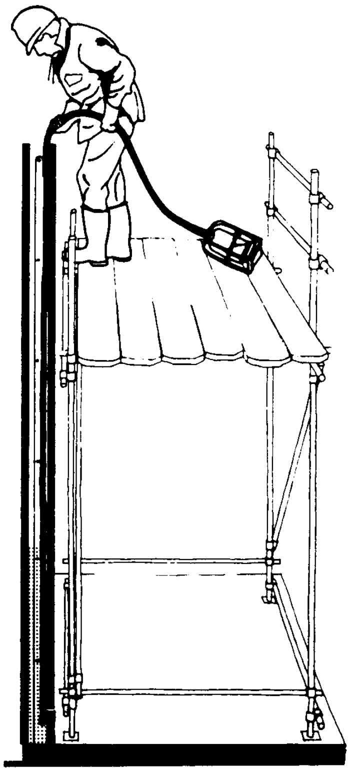
At the position of this intermediate horizontal construction joint it will, of course, be necessary to form clean out ports, as was required for starting off at the ground level. A sand covering of the cleaned surface will keep droppings from sticking to the surface. Cleaning out of the horizontal joint is required before placing the next lift. The surface should be wetted down immediately prior to grouting, being surface damp but no puddles of water present on the surface.



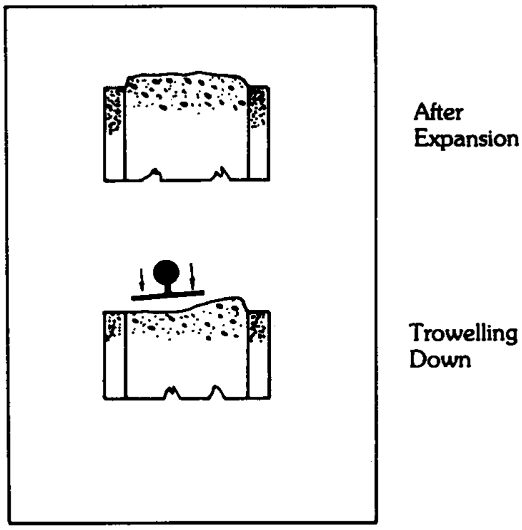
Step 1: Clean out grout space and remove all debris and loose material from construction joint.



Step 2: Grout the wall in a semi continuous operation to the top.



Step 3: Consolidate with vibrator or by rodding as work of filling proceeds to the top.



Step 4: After waiting for expansion, trowel down and recompact the top surface of the expanded grout. An alternative method is to place a weighted board on top of the wall to contain the expansion.

Figure 9: *High lift grouting with expansive admixture. (Follow Steps 1-4 as illustrated).*

14.0 Consideration of Weather

NZS 4210 specifies general and special requirements for construction of masonry in a variety of weather conditions. These requirements shall be followed in all respects.

14.1 Cold Weather Construction

Generally, masonry construction should not be carried out when the air temperature drops below about 4°C, unless precautions are carried out.

COMMENTARY: *The precautions for cold weather construction are shown:*

(i) *Water used for mixing mortar shall be heated.*

(ii) *Masonry shall be protected for not less than 24 hours after laying by covers, blankets, heated enclosures, or the like to ensure that the mortar can gain strength without freezing or harmful effects from cold winds.*

(iii) *No frozen materials nor materials containing ice shall be used.*

14.2 Hot Weather Construction

When masonry Construction is carried out at an air temperature of more than 25 C the following precautions need to be taken.

(i) Masonry units may be lightly dampened before laying.

(ii) Mortar shall be kept moist and shall not be spread on the wall more than two unit lengths ahead of the units being placed.

(iii) The mortar shall be prevented from drying so rapidly that it cannot cure properly; this may be done by applying a very light fog spray several times during the first 24 hours after laying or by other protective measures over the same period.

(iv) Grout shall be protected from too rapid drying.

CCANZ tests showed significant improvement in bond test results by lightly dampening down the surfaces to be bonded.

Having constructed the wall, some effort should be made to try and prevent the mortar from drying out too quickly by the use of some light spray. In addition, the tops of the walls will be very prone to drying out and this will affect the masonry grout. However, to overcome the problem, the tops of the walls should be temporarily covered to reduce the effects of the wind or rain.

15.0 Protection and Cleaning of the Works

Keep fair faced masonry clean of all mortar droppings, grout splashes or stains of any kind as the work proceeds.

Keep the work site clean, tidy, orderly and in a safe state.

At the completion of the works, clean down all masonry and adjoining surfaces, floors, etc and remove all waste material and equipment from these areas.

4.4 Design Publications

Introduction

Please find attached to this section, two papers on reinforced concrete masonry walls:

1. *Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls*, written by K. C. Voon and J. M. Ingham, published in the March 2006 Journal of Structural Engineering.

2. *Experimental In-Plane Strength Investigation of Reinforced Concrete Masonry Walls with Openings*, written by K. C. Voon and J. M. Ingham, published in the May 2008 Journal of Structural Engineering.

Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls

K. C. Voon[1] and J. M. Ingham[2]

Abstract: This paper presents test results of ten single-story reinforced concrete masonry shear walls. Test results are summarized and compared with design formulae specified by the New Zealand masonry design standard NZS 4230:1990 and by the National Earthquake Hazards Reduction Program. It was determined that the test walls exhibited shear strength significantly exceeding the NZS 4230:1990 maximum permissible shear stress, confirming that NZS 4230:1990 was overly conservative in accounting for masonry shear strength. It was also confirmed from the test results that masonry shear strength increases with the magnitude of applied axial compressive stress and the amount of shear reinforcement, but that the shear strength decreases inversely in relation to an increase in wall aspect ratio. In addition, it was shown that the postcracking performance of shear dominated walls was substantially improved when uniformly distributing the shear reinforcement up the height of the walls.

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CE Database subject headings: Concrete masonry; Walls; Shear strength; Grouting; Reinforcement; Axial loads.

Introduction

Masonry has been used as a common construction material worldwide for many centuries. However, the vulnerability of unreinforced masonry systems was highlighted during past and recent earthquakes. Consequently, reinforcement was introduced to masonry shear walls to resist lateral forces generated in regions of high seismic activity. These walls are usually subjected to simultaneous gravity and lateral loads, resulting in overturning moments during seismic excitation. Depending on the load condition, the amount of longitudinal and shear reinforcement, and the aspect ratio, two types of failure mechanisms can be identified in masonry shear wall panels subjected to in-plane loading. One is a flexural type of failure, which is characterized by the tensile yielding of vertical reinforcement or crushing of masonry at critical wall sections. This is generally the preferred mode, as tensile failure is ductile and is effective in dissipating energy in conjunction with reinforcement yielding. The second type is a shear failure, which is characterized by diagonal tensile cracking. Wall panels that fail predominantly in a shear mode exhibit brittle behavior, characterized by rapid strength degradation soon after the maximum strength is developed. In order to prevent such catastrophic failure, New Zealand was amongst the first countries to develop reinforced masonry seismic design procedures based on the principle of capacity design (Priestley 1980) that required the dependable shear strength to exceed the maximum lateral loading necessary to develop the wall flexural overstrength.

At the time the New Zealand masonry design standard NZS 4230:1990 was released, it was recorded in the associated commentary that the shear strength provisions in this standard were overly conservative. However, the absence at that time of experimental data related to the shear strength of masonry walls when subjected to in-plane seismic forces prevented the preparation of more accurate criteria. A comprehensive literature review conducted by the authors indicated that since the release of NZS 4230:1990, many experimental studies have been carried out worldwide to investigate the in-plane shear capacity of reinforced masonry shear walls. It was demonstrated that the shear resistance of reinforced masonry walls comes from several mechanisms, such as tension of horizontal reinforcement, dowel action of vertical reinforcement, applied axial stress, and aggregate interlocking. Although some researchers observed that the shear strength of a wall panel is not directly proportional to the amount of horizontal reinforcement present (Mayes et al. 1976; Chen et al. 1978; Hidalgo et al. 1979; Hiraishi 1985; Sveinsson et al. 1985), the ductility of shear-dominated wall panels was improved by increasing the amount of horizontal reinforcement (Matsumura 1988; Shing et al. 1990). Furthermore, it was demonstrated that horizontal reinforcement can effectively inhibit the opening of diagonal cracks (Priestley 1977).

The New Zealand masonry design standard was recently revised, providing an opportunity to update the masonry shear strength criteria. Therefore, ten single-story concrete masonry wall panels were tested at the University of Auckland in order to examine the in-plane shear strength of concrete masonry walls constructed using New Zealand masonry units utilizing pumice aggregate and assembled using common local construction techniques. The main variables considered in this experimental program included the amount and distribution of shear reinforcement, level of axial compression stress, type of grouting, and wall aspect ratios. This experimental program supplemented the experimental data already available by specifically investigating the

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Note. Associate Editor: Rob Y. H. Chai. Discussion open until August 1, 2006. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on August 16, 2004; approved on July 26, 2005. This paper is part of the ***Journal of Structural Engineering***, Vol. 132, No. 3, March 1, 2006. ©ASCE, ISSN 0733-9445/2006/3-400–408/$25.00.

Table 1. Masonry Shear Wall Specimens

Wall	H (mm)	L (mm)	Effective width (mm)	H_e (mm)	H_e/L	Reinforcement		ρ_h (%)	Grouting	Axial stress (MPa)
						Vertical	Horizontal			
1	1,800	1,800	140	1,800	1.0	5-D20	5×R6	0.05	Full	—
2	1,800	1,800	140	1,800	1.0	5-D20	1-R6	0.01	Full	—
3	1,800	1,800	140	1,800	1.0	5-D20	5-D10	0.14	Full	—
4	1,800	1,800	140	1,800	1.0	5-D20	2-D10	0.06	Full	—
5	1,800	1,800	60	1,800	1.0	5-D20	—	—	Partial	—
6	1,800	1,800	60	1,800	1.0	3-D20	—	—	Partial	—
7	1,800	1,800	140	1,800	1.0	5-D20	5-R6	0.05	Full	0.50
8	1,800	1,800	140	1,800	1.0	5-D20	5-R6	0.05	Full	0.25
9	3,600	1,800	140	3,600	2.0	5-DH25	9-R6	0.05	Full	0.25
10	1,800	3,000	140	1,800	0.6	8-D20	5-R6	0.05	Full	0.25

shear strength of walls subjected to low axial compression stresses ($\sigma_n \leqslant 0.50$ MPa) and low shear reinforcement ratios ($\rho_h \leqslant 0.062\%$). Experimental studies conducted in both the United States and Japan involved masonry walls that were subjected to σ_n and ρ_h of up to 5.9 MPa and 0.67%, respectively.

Experimental Program

Test Specimens

A total of ten concrete masonry cantilever walls were tested. The dimensions and reinforcement details for the ten walls are summarized in Table 1. All walls (except Walls 5 and 6) were fully grouted and had vertical reinforcing steel spaced at 400 mm c/c, but varied in the quantity and positioning of horizontal shear reinforcement. The horizontal shear reinforcing steel was uniformly distributed evenly up the height of the walls and was hooked (180° bend) around the outermost wall vertical reinforcing bars. Walls 5 and 6 were partially grout filled with no horizontal shear reinforcement, and their vertical reinforcement was spaced at 400 mm and 800 mm, respectively. For the two partially grout-filled walls, only cells containing reinforcing bars were filled with grout. Walls 1 to 6 were tested without externally applied vertical axial compression load. Walls 7 and 8 were duplicates of Wall 1, but with externally applied axial compression stress of 0.5 MPa and 0.25 MPa, respectively. Walls 9 and 10, which were constructed to H_e/L ratios of 2.0 and 0.6, were both subjected to axial compression stress of 0.25 MPa. All test walls (except Wall 3) were designed to have a dominant shear type of failure. Wall 3 was designed to fail in flexure.

All walls were constructed by experienced masons under supervision, and employed a running bond pattern of standard production 15 series (140 mm wide) precast concrete masonry units (CMUs). This width was adopted for the tests to ensure that high shear stresses could be applied within the maximum load restrictions of the hydraulic actuators available at the University of Auckland. Open-end bond beam CMUs having a depressed web were used throughout the wall height to allow the horizontal shear reinforcement to be positioned at all levels and to enhance the continuity of grout. Dricon Trade Mortar—a bagged 1:4 portion of Portland cement and sand by volume—was used throughout. High slump ready-mix grout using small aggregate (7 mm) was employed for filling the cavities within the test walls and a common commercially used expansive chemical additive was added to the grout to avoid formation of voids caused by high shrinkage of the grout. Additional detailed descriptions of the wall panels are reported in Voon and Ingham (2003).

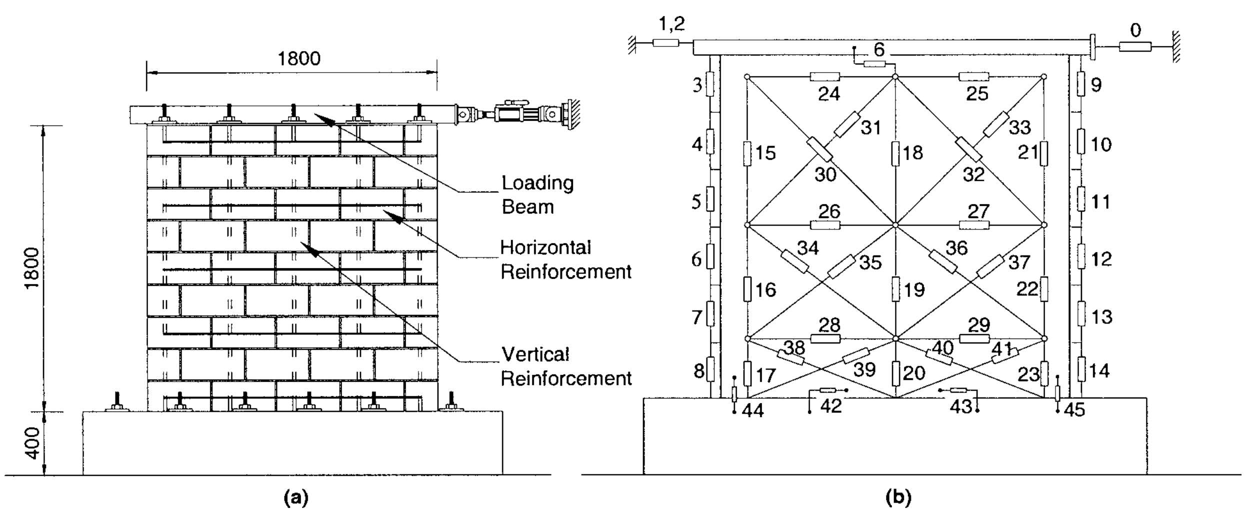


Fig. 1. Test setup and instrumentation

Table 2. Summary of Test Results

Wall	f'_m (MPa)	Prediction F_n (kN)	Prediction V_n (kN)	Prediction $\frac{V_n}{F_n}$	Test result V_{max} (kN)	Test result d_{vmax} (mm)	Test result $\frac{V_{max}}{V_n}$	Test result d_u (mm)	Failure mode
1	17.6	229	219	0.95	215	10	0.98	12	Flexure/ Shear
2	17.6	229	195	0.85	195	6	1.00	8	Shear
3	17.0	229	250	1.09	215	8	0.86	10	Flexure/ Sliding
4	17.0	229	219	0.95	223	8	1.02	10	Shear
5	18.5	229	91	0.40	143	8	1.57	10	Shear
6	18.5	142	91	0.64	93	8	1.02	14	Shear
7	18.8	282	256	0.91	263	6	1.03	8	Shear
8	18.8	256	240	0.93	244	6	1.02	6	Shear
9	24.3	272	268	0.99	207	20	0.77	24	Shear
10	24.3	672	568	0.85	598	4	1.05	4	Shear

Test Setup and Instrumentation

The test setup is shown in Fig. 1(a), primarily consisting of a reinforced concrete footing and a horizontally mounted hydraulic actuator providing a horizontal shear force to the top of the wall through a 150×75 steel channel section (herein called the loading beam). The wall was stabilized from moving in its out-of-plane direction by two parallel horizontal struts that were positioned perpendicular to the wall and hinged to the channel and a reaction frame. Fig. 1(b) schematically shows typical wall instrumentation. Lateral load was measured by a load cell positioned in series with the hydraulic actuator, denoted as instrument 0 in Fig. 1(b). The lateral displacement at the top of the wall was measured by instruments 1 and 2 (portal type LVDT). Wall flexural and shear deformations were monitored by instruments 3–14 and 15–41, respectively. The base uplift was monitored by instruments 44 and 45, installed very close to the base joint while, relative sliding displacement between the wall and footing was measured by instruments 42 and 43.

Material Properties

Samples were taken from steel reinforcement used in wall construction. These samples were subjected to tensile testing, with the average yield strengths (f_y) for the R6, D10, and D20 reinforcing steel being 325, 320, and 318 MPa, respectively. Also, masonry compression strength, f'_m, was determined by material testing of masonry prisms. These prisms were built of three concrete masonry units staked on top of each other, using the same construction technique as was used for the wall. The prisms were grout filled at the same time as the wall and were then subjected to the same curing condition as the wall panels. The prisms were tested using an Avery universal testing machine. This type of test specimen provided the most accurate estimate of f'_m, with values summarized in Table 2.

Testing Procedure

The cyclic loading sequence adopted for all tests is shown in Fig. 2, and consisted of a series of displacement-controlled components. Each stage of loading consisted of two cycles to the selected displacement. This experimental program defined failure as the point on the loading curve where the wall strength had reduced to 80% of the maximum strength previously recorded, with d_u as the corresponding ultimate displacement (see Fig. 3).

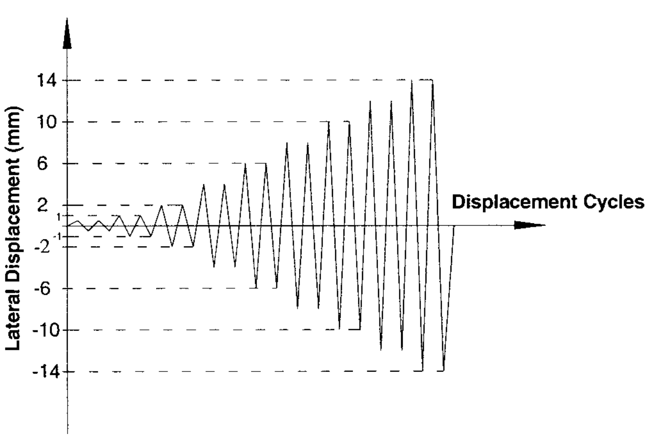


Fig. 2. Imposed displacement history

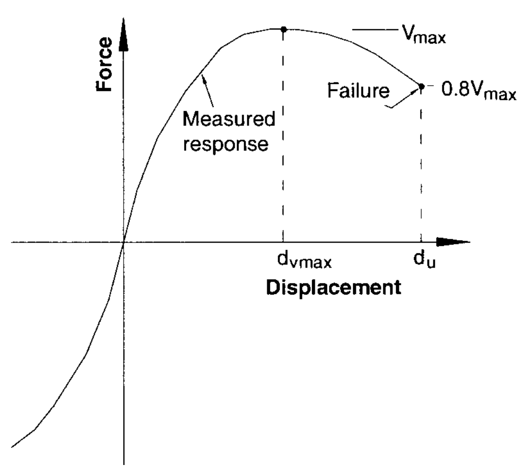
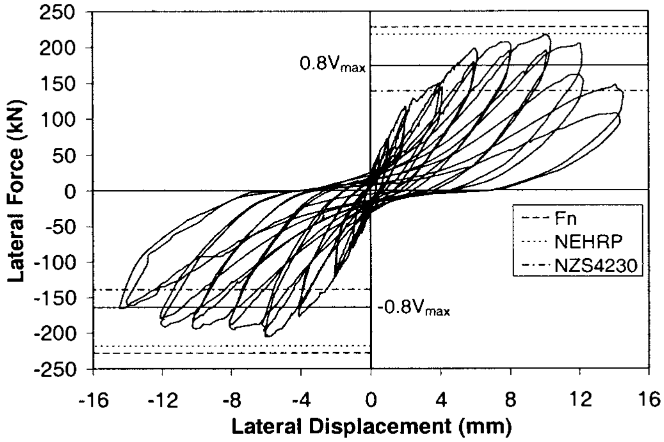
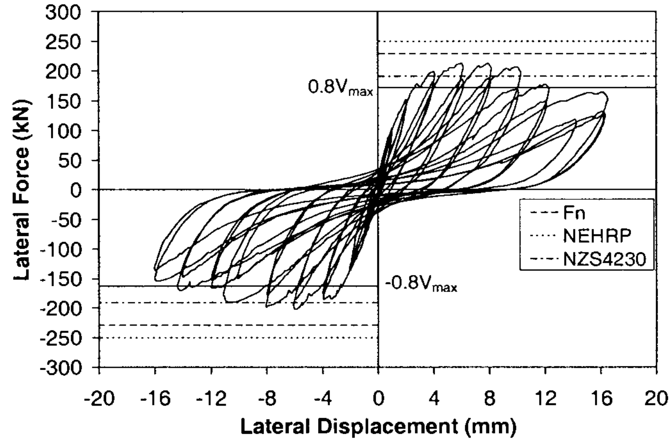


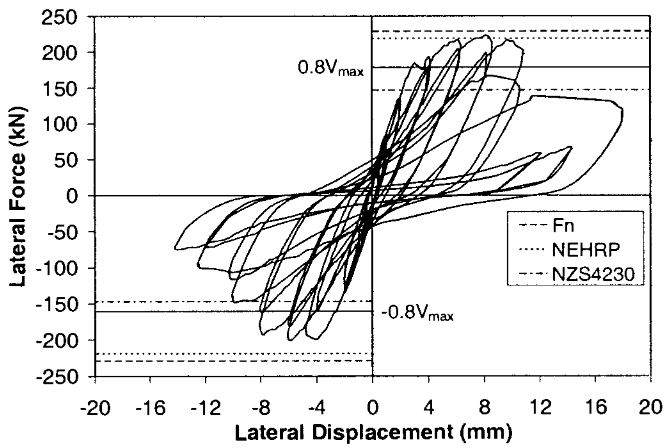
Fig. 3. Theoretical point of failure



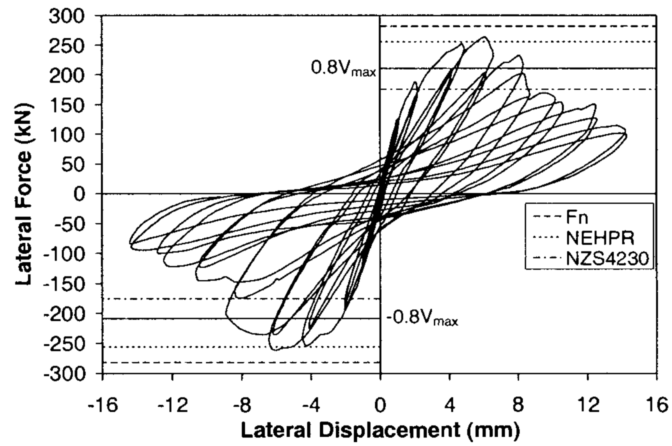
(a) Wall 1



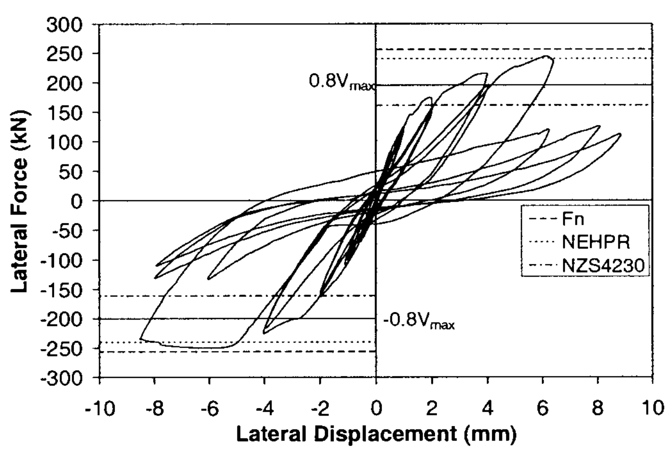
(b) Wall 3



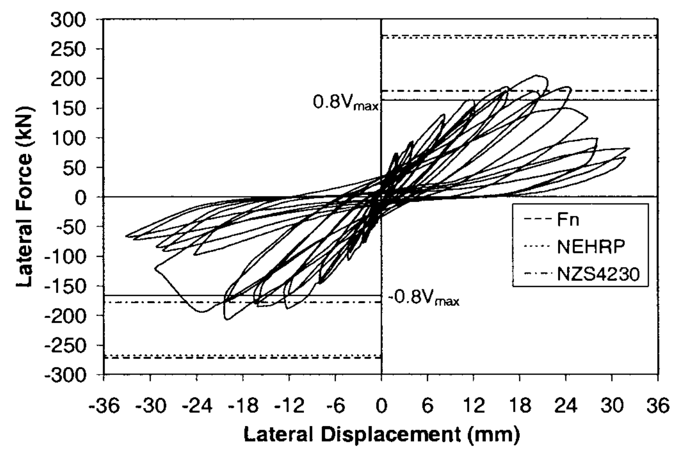
(c) Wall 4



(d) Wall 7



(e) Wall 8



(f) Wall 9

Fig. 4. Force-displacement histories

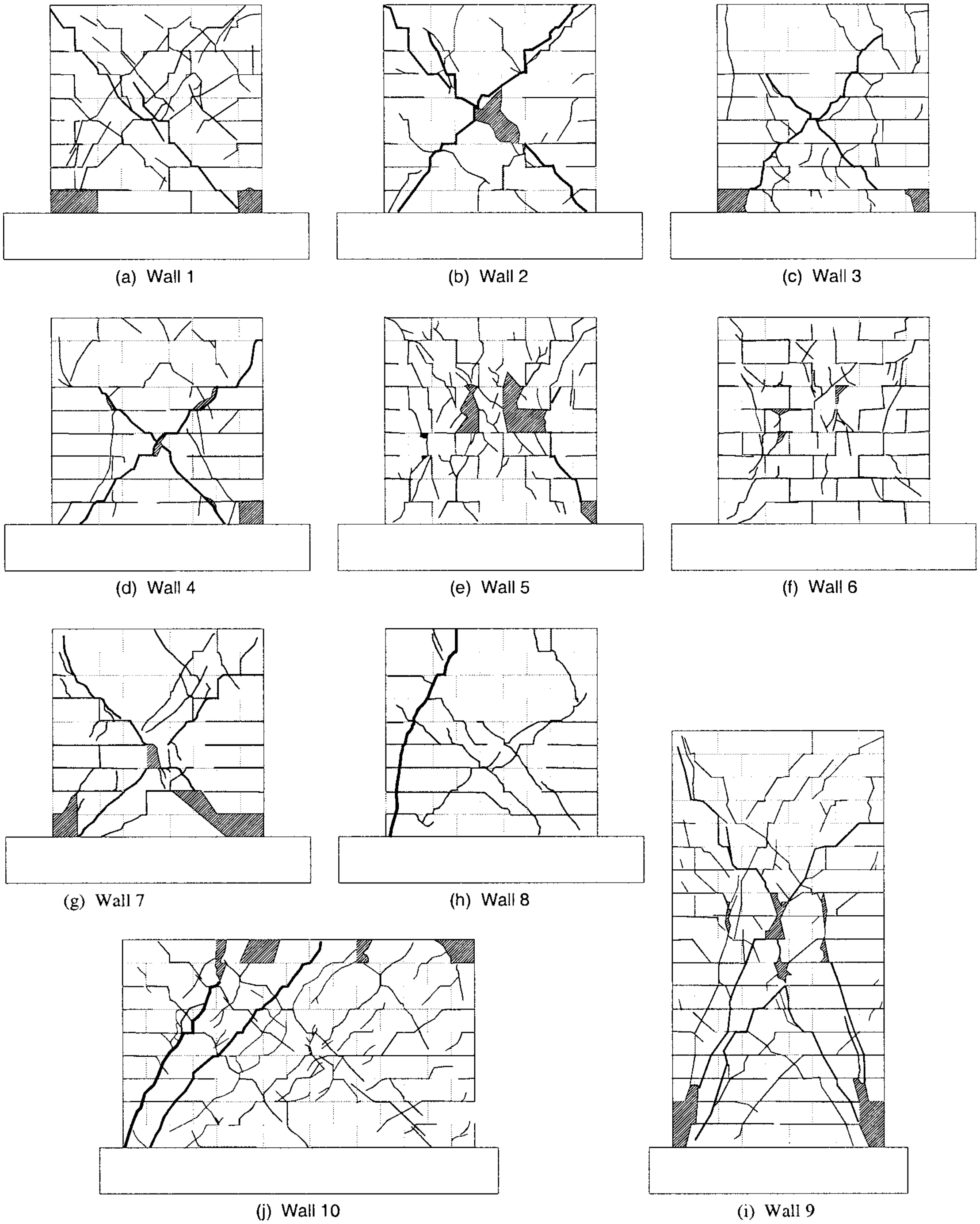


Fig. 5. Masonry wall cracking patterns

The predicted masonry shear strength, V_n, was calculated according to the formulae presented in NZS 4230:1990

$$V_n = 0.8 v_m A_n + 0.8 \rho_h A_n f_{yh} \tag{1}$$

where v_m is the greater of $v_m = 0.30$ MPa or $v_m = 0.03 f'_m + 0.3\sigma_n \leqslant 0.72$ MPa, with $f'_m \leqslant 16$ MPa; and also according to NEHRP (1997)

$$V_n = 0.083\left(4 - 1.75\frac{H_e}{L}\right)A_n\sqrt{f'_m} + 0.5\rho_h A_n f_{yh} + 0.25\sigma_n A_n \tag{2}$$

Experimental Results

General wall behavior is summarized in Table 2, where V_{max}=maximum lateral force recorded and d_{vmax}=corresponding displacement (see Fig. 3). The predicted masonry shear strength, V_n, listed in Table 2, was based on Eq. (2), being most recently developed and having the ability to predict masonry shear strength with accuracy superior to that of Eq. (1) (Voon and Ingham 2001). The shear strengths of the two partially grouted walls listed in Table 2 were calculated using a common New Zealand approach where the effective section width for shear was assumed to be the net thickness of the face shells only. This limitation was

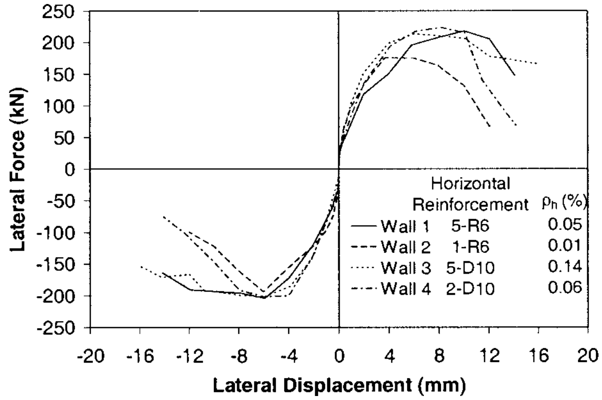


Fig. 6. Effect of shear reinforcement on masonry shear strength

to satisfy the requirements of continuity of shear flow and to avoid the possibility of vertical shear failure up a continuous ungrouted flue. This resulted in an effective width of 60 mm for the partially grout-filled concrete masonry walls. The F_n value included in Table 2 was the lateral force required to develop the nominal flexural strength of the wall and was evaluated based on a rectangular masonry compression stress block using measured rather than specified material strengths.

Fig. 4 depicts a selection of experimentally obtained force-displacement curves. The force corresponding to the wall nominal flexural strength is identified by the symbol F_n. Also shown on these plots are lines corresponding to the predicted shear strengths derived from Eq. (1) (denoted NZS4230) and Eq. (2) [denoted National Earthquake Hazards Reduction Program (NEHRP)]. Table 2 and Fig. 4 both show that for all walls (except Wall 3), the predicted shear strength was less than the force corresponding to the predicted flexural strength.

All walls, except for Walls 1 and 3, exhibited shear dominated response. This type of response was characterized by the development of early horizontal flexural cracks that were then superceded by wide-open diagonal cracks that extended throughout the masonry walls. These diagonal cracks were initiated by tension splitting of masonry in the compression strut that formed in the walls. Rapid strength degradation was observed for walls that failed in the shear dominated mode [see Figs. 4(c and d)], attributed to the widening of diagonal cracks and crushing of masonry.

Although Wall 1 was expected to fail in shear, it did not exhibit sudden strength degradation after reaching maximum wall strength, therefore indicating possible yielding of vertical reinforcement during testing. Consequently, it was classified as having a flexure/shear type of failure. This type of failure mode was possible due to the absence of axial load and the adoption of closely distributed shear reinforcement using small size reinforcing bars (i.e., R6). Accordingly, the initial diagonal cracks did not widen significantly under increasing lateral force, but instead new sets of diagonal cracks formed and gradually spread over the wall diagonals, accompanied by higher energy dissipation and ductile behavior. It is also shown in Fig. 4(b) that Wall 3 failed to reach F_n. This was due to significant sliding at the wall base. Consequently, a portion of the shear force was transferred by dowel action of vertical reinforcement. This in turn led to a reduction in wall lateral strength. Fig. 5 illustrates wall cracking patterns at the end of testing. The shaded areas shown in Fig. 5 indicate masonry crushing.

The data of Fig. 4 confirm that the NZS 4230:1990 shear expression significantly underpredicted the in-plane shear strength of masonry walls. It is also observed from the same figure that shear prediction using Eq. (2) provided a better match for walls that had $H_e/L \leq 1.0$. However, Fig. 4(f) illustrates that Eq. (2) overpredicted the shear strength of Wall 9 by about 23%. This overprediction of shear strength was due to the substantial increase in diagonal shear cracking (compared to other walls) that developed before the maximum wall strength was reached, and also due to the fact that Eq. (2) does not address the reduction in masonry shear strength within potential plastic hinge regions. Experimental results indicated that a displacement ductility level of 2.9 was recorded when Wall 9 developed its maximum strength.

Discussion

This section discusses how design parameters, such as the amount of shear reinforcement, distribution of shear reinforcement, magnitude of axial compression load, wall H_e/L ratio, and type of grouting affect the shear strength of masonry walls. The figures in this section are limited to force-displacement envelopes, arranged in groups to show the effect of a particular parameter.

Effect of Shear Reinforcement

In this study both the amount of shear reinforcement and the distribution of reinforcing bars throughout the height of the wall were varied. It was observed that a change in the quantity of shear reinforcement had a direct influence on V_n. This is clearly illustrated in the experimental results shown in Fig. 6, where the maximum shear strength increased from 195 kN for Wall 2 to 215 kN for Wall 1, resulting in a strength increase of 10% when the shear reinforcement increased from 1 R6 to 5 R6 reinforcing bars.

Fig. 7 shows the effectiveness of horizontal reinforcement in enhancing the post-cracking performance of masonry walls. It can be seen that the deformability of walls improved when the amount of shear reinforcement increased from 1 R6 to 5 R6 reinforcing bars for the case of Walls 2 and 1, and when the horizontal reinforcement increased from 2 D10 to 5 D10 reinforcing bars for the case of Walls 4 and 3. The advantage of distributing shear reinforcement (using a greater number of reinforcing bars of smaller diameter) up the height of the wall can be clearly observed by comparing the force-displacement envelopes of Walls 1 and 4. The two walls contained approximately the same total cross-sectional area of shear reinforcement, but the shear reinforcement was distributed differently. Fig. 7 shows that Wall 4

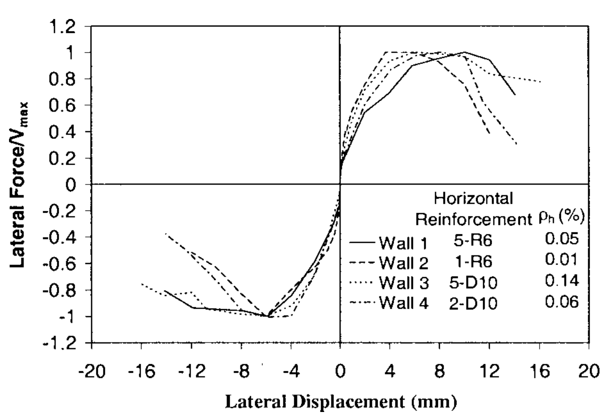


Fig. 7. Force-displacement envelopes normalized with V_{max}

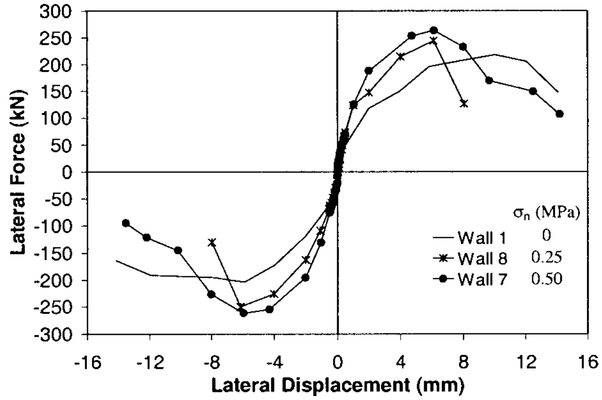


Fig. 8. Effect of axial compression stress on masonry shear strength

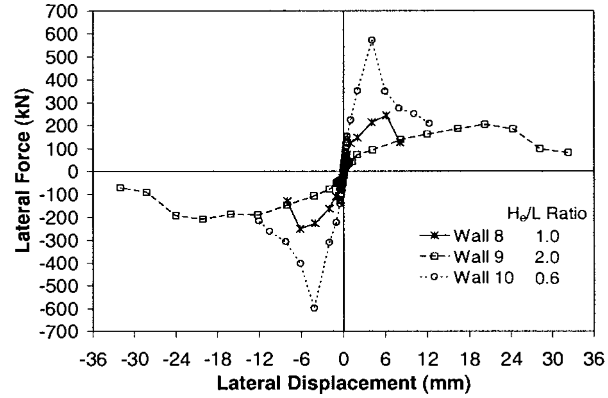


Fig. 10. Effect of H_e/L on masonry shear strength

exhibited abrupt strength degradation after the peak wall strength was attained, whereas Wall 1 exhibited a more gradual strength degradation. This type of failure was made possible for Wall 1 due to the adoption of 400-mm spaced horizontal reinforcement. The closely spaced shear reinforcement enabled the distribution of stresses throughout the wall diagonals after the initiation of shear cracking. Accordingly the initial diagonal cracks did not widen significantly under increasing lateral displacements; instead, new sets of diagonal cracks formed and gradually spread over the wall diagonals, accompanied by higher energy dissipation and more ductile behavior. It was therefore possible to classify Wall 1 as having a flexure/shear type of failure or "ductile shear failure." Conversely, Wall 4 exhibited a "brittle shear failure." This type of shear failure was expected since Wall 4 was constructed without the closely distributed shear reinforcement. This prevented the tensile stress due to applied shear force from being adequately transferred across the diagonal cracks. Hence, the cracks opened extensively, resulting in a major x-shaped diagonal crack pair [see Fig. 5(d) for wall crack pattern], leading to a relatively sudden and destructive failure.

Effect of Axial Compression Stresses

The positive influence of axial compression stress on masonry shear strength is illustrated in Fig. 8. This figure shows the performance of the three masonry walls that had the same dimensions and reinforcement details, but were subjected to varying levels of axial compression stress. The maximum shear strength increased from 215 kN (Wall 1), to 244 kN (Wall 8) and 263 kN (Wall 7) when the axial compression stress was increased from zero to 0.25 MPa and 0.50 MPa, resulting in an increase of about 13% and 22%, respectively. Also observed from the same figure is reduction of the postcracking deformation capacity of Walls 7 and 8. This was because the failure type became more brittle as the axial compression stress increased.

It was also noted from observations made during the experimental process that an increase in axial compression stress delayed the initiation of cracking until larger lateral force was applied. This can be explained from principal stresses: a larger lateral force is required to exceed the compressive field resulting from the larger axial load. This compressive field must first be overcome before cracking can initiate.

Effect of Grouting

Experimental results presented in Table 2 illustrate the significant reduction of shear strength when the masonry walls were partially grouted. For the partially grouted masonry walls, it was also shown that Wall 5, with five grouted flues, had about 50% higher strength than the corresponding Wall 6, which had only three grouted flues. However, the effect of grouting method becomes less significant when the net shear stress is calculated by considering the cross-sectional area of both the CMUs and the grouted cells. This is shown in Fig. 9, where force-displacement envelopes of both fully (without axial load) and partially grouted walls (Walls 5 and 6) are presented.

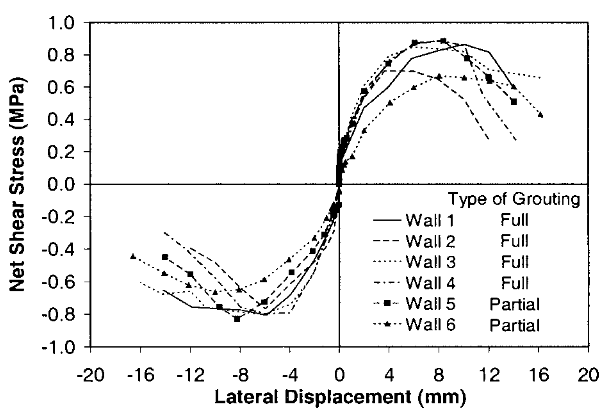


Fig. 9. Effect of grouting on masonry

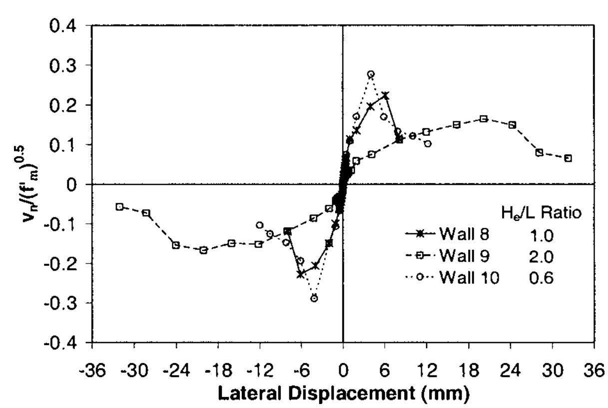


Fig. 11. Relationship of $v_n/\sqrt{f_m'}$ versus H_e/L

Effect of Wall H_e/L Ratio

Fig. 10 illustrates that the masonry shear strength decreased from a maximum of 244 kN for Wall 8 ($H_e/L=1.0$) to 207 kN when the H_e/L ratio was increased to 2.0 in the case of Wall 9, resulting in a decrease of 15%. Fig. 10 also shows a significant increase in masonry shear strength for Wall 10 that was constructed to a reduced H_e/L ratio. This indicates that masonry shear strength increases as the H_e/L ratio decreases, while it decreases inversely in relation to an increase in the H_e/L ratio. It is realized, however, that the different shear strengths shown in Fig. 10 could have been due to the variations in the net area A_n and in f'_m. In order to meaningfully observe the relation between masonry shear strength and H_e/L ratio, the influence of A_n and f'_m must be excluded from the test results. Consequently, a plot of $v_n/\sqrt{f'_m}$ is presented in Fig. 11. As anticipated, tendency similar to that observed in Fig. 10 is evident.

Conclusions

The effects of shear reinforcement, axial compression load, type of grouting, and wall aspect ratio on masonry shear strength were investigated in this study. It was established that axial compression load had a significant influence on the in-plane shear performance of masonry shear walls, mainly because it suppressed the tensile field in a material inherently weak in tension. Consequently, as the axial compression load increased, so did the ability of the walls to provide shear resistance. However, the postcracking deformation capacities were observed to reduce with increasing axial load. This was because of the increasing brittleness of this failure type as the axial compression stress increased.

It was observed that shear reinforcement not only provided additional shear resistance, but also improved the postcracking performance of the masonry walls when shear reinforcement was uniformly distributed up the height of the walls. The provision of closely spaced shear reinforcement enabled the distribution of stresses throughout the wall diagonals after the initiation of shear cracking. Accordingly, the initial diagonal cracks did not widen significantly under increasing lateral displacements, but instead new sets of diagonal cracks formed and gradually spread over the wall diagonals, accompanied by higher energy dissipation and more ductile behavior. The test results also demonstrated that partial grouting significantly reduced masonry shear strength. However, the effect of grouting became less significant when net shear stress was calculated accounting for the cross-sectional area of both the masonry units and grouted cells. In addition, the test results indicated that masonry shear strength decreased inversely in relation to the H_e/L ratio.

Finally, the test results demonstrated that NZS 4230:1990 was conservative in its treatment of masonry shear strength. This was shown by the constant underprediction of shear strength for the masonry walls investigated in this study. The test results of this study also demonstrated that the NEHRP masonry shear expressions provided reasonably accurate prediction for walls that had $H_e/L \leqslant 1.0$, but overpredicted the shear strength of Wall 9 that had $H_e/L=2.0$.

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The opinions and conclusions presented herein are those of the authors, and do not necessarily reflect those of the University of Auckland or any of the sponsoring parties to this project.

Notation

The following symbols are used in this paper:

A_n = net cross-sectional area of wall;
d_u = lateral displacement at wall failure;
d_{vmax} = displacement at maximum recorded lateral force;
F_n = lateral force to generate calculated wall nominal flexural strength;
f'_m = masonry compressive strength;
f_{yh} = yield strength of horizontal shear reinforcement;
H = wall height;
H_e = effective wall height;
L = wall length;
V_{max} = experimentally recorded maximum lateral force;
V_n = nominal shear strength of masonry wall;
v_m = permissible shear stress;
ρ_h = shear reinforcement ratio; and
σ_n = axial compressive stress.

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Experimental In-Plane Strength Investigation of Reinforced Concrete Masonry Walls with Openings

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Abstract: This paper presents test results of eight partially grout-filled perforated concrete masonry walls that were subjected to cyclic lateral loading. Test results obtained from this research indicated that the size of openings and the length of trimming reinforcement significantly affected the lateral strength of perforated masonry walls. It was shown that the current New Zealand nonspecific masonry design standard NZS 4229 unsafely overpredicts the strength capacity of concrete masonry walls with small openings, and an amendment is proposed to rectify this matter. It was also shown that NZS 4229 is increasingly conservative as the height of openings increased. Diagonal cracking patterns that formed during testing were observed to align well with the load paths by which lateral shear force was assumed to be transferred to the foundation when using strut-and-tie analysis. This observation supports the use of the strut-and-tie technique as a viable tool to evaluate the flexural strength of walls of this type.

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CE Database subject headings: Concrete masonry; Walls; Grouting; Openings; Reinforcement; Cyclic load.

Introduction

For many decades, masonry has been used as a common structural material in a large proportion of New Zealand building projects. However, the poor performance of unreinforced masonry in the magnitude 7.8 1931 Hawke's Bay earthquake (Dowrick 1998; Scott 1999) subsequently led to a ban on all unreinforced masonry in New Zealand, and the associated development of conservative reinforced concrete masonry design provisions. Consequently, a typical detail in fully-grouted concrete masonry walls was the use of ∅ 12 mm deformed reinforcing bars having a characteristic yield strength of 300 MPa and spaced at 400 mm centers, both vertically and horizontally. The recent promulgation of alternative construction forms such as tilt-up precast concrete wall systems has resulted in the perception within New Zealand that reinforced concrete masonry is an expensive form of construction when compared with competing products and systems. Consequently, a decision was made by the New Zealand concrete masonry industry to develop a nonspecific design standard NZS 4229 (NZS 1999) which, while retaining suitable conservatism, was more realistic in its treatment of measured experimental response. In particular, it was permitted to use partially grout-filled nominally reinforced concrete masonry in the most seismically active regions of New Zealand. Furthermore, efforts were made to simplify use of the standard so that the design of single and double story masonry structures, not containing crowds and not dedicated to the preservation of human life, could be effectively conducted by architects and architectural draftspersons with limited, if any, input from consulting structural engineers.

The in-plane lateral strength of a masonry panel is specified in NZS 4229 through determination of its "bracing capacity," with the bracing capacity values being derived from wall tests conducted at the University of Auckland by Brammer (1995) and Davidson (1996), of which only two considered the performance of walls with openings. However, it was subsequently identified that an important lintel reinforcement detail adopted in testing of these two walls differed from that specified in NZS 4229. Hence, a third wall, having an opening and with reinforcement detailing complying with NZS 4229 was tested (Ingham et al. 2001), and it was observed that this wall did not achieve the bracing capacity prescribed in NZS 4229. Further assessment indicated that the existing design standard may be nonconservative in its treatment of walls with openings. Consequently, an experimental study was conducted at the University of Auckland to investigate the influence of openings in partially grout-filled nominally reinforced concrete masonry walls.

This paper describes the results from structural testing of eight single story-height concrete masonry walls that were recently reported in comprehensive form in Voon and Ingham (2006). The primary objective of the study was to validate the adequacy of NZS 4229 in addressing the bracing capacity of masonry walls containing openings. The eight walls of the study had variations in lintel reinforcement detailing, including detailing that complied with NZS 4229 and a range of penetration geometries.

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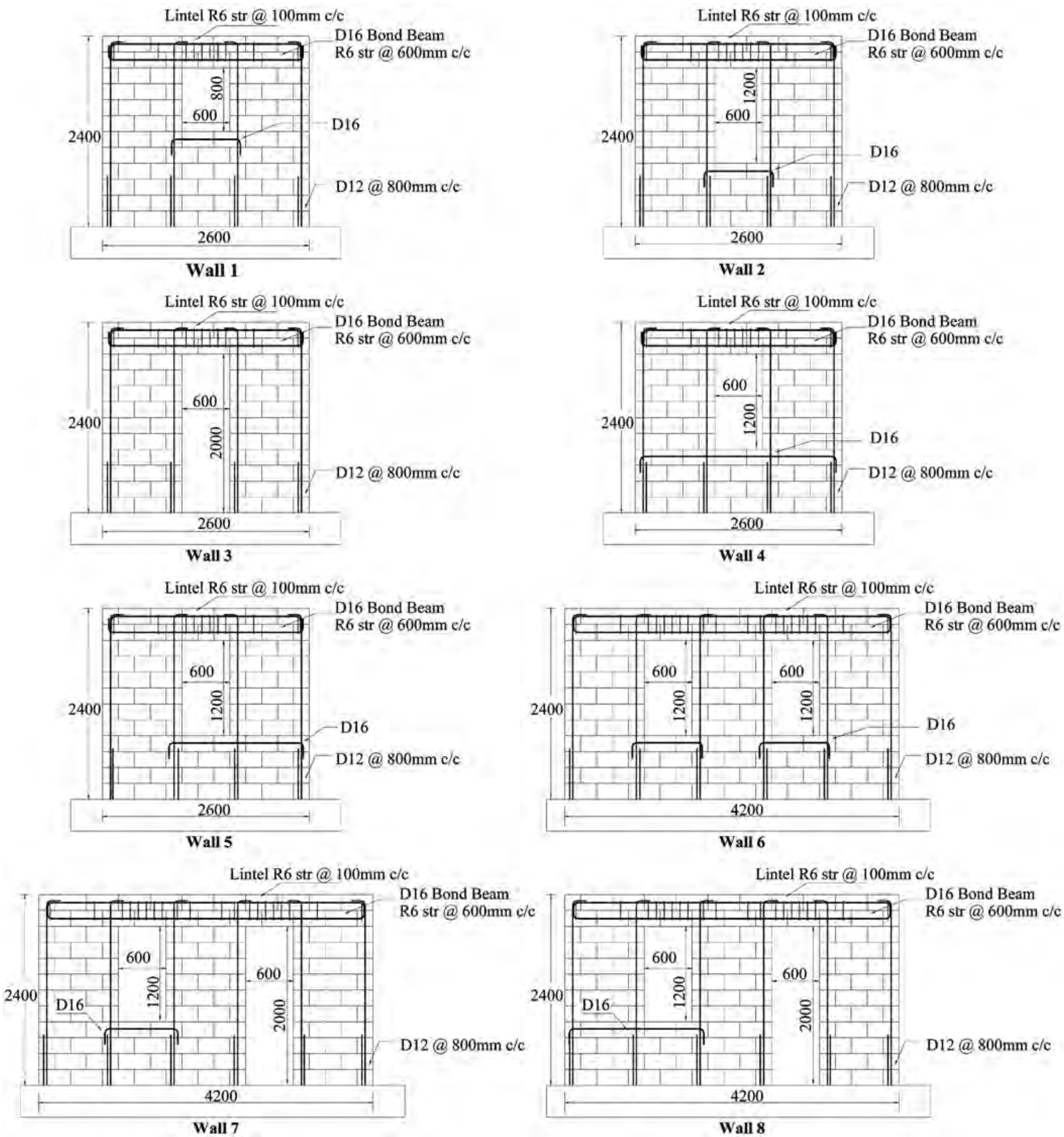

Fig. 1. Wall geometries and reinforcing details

Experimental Program

Test Specimens

The geometries and reinforcement details of the eight single-story masonry walls are shown in Fig. 1. All eight walls were partially grout filled, where only those cells containing reinforcement were grouted, and were constructed to a common height of 2,400 mm. None of the eight masonry walls had applied axial compression load. The vertical reinforcement of the partially grout-filled walls shown in Fig. 1 was spaced at 800 mm centers, and the horizontal reinforcement in all walls consisted of two D16 reinforcing bars placed in a solid grout-filled bond beam within the top two block courses and a D16 trimming reinforcing bar placed below all window openings. All wall openings had a length of 600 mm.

All walls were constructed by experienced masons under supervision, and employed a running bond pattern of standard production 15 series (140 mm wide) precast concrete masonry units (CMUs). The mortar used throughout the study was a preblended bagged 1:4 mix of portland cement to sand by volume. High

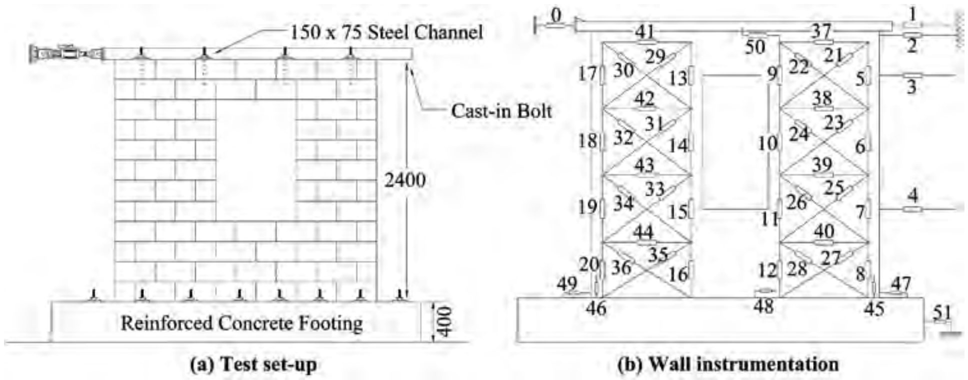

Fig. 2. Typical test setup and wall instrumentation

slump ready-mix grout using small aggregate (7 mm) was employed for filling the cavities within the test walls and a common commercially used expansive chemical additive was added to the grout to avoid formation of voids caused by high shrinkage of the grout.

Test Setup and Instrumentation

The testing of specimens reported herein was conducted according to the setup shown in Fig. 2(a). The test setup and method of loading adopted in this experimental program were designed to simulate the response that a masonry shear wall would experience during seismic excitation. Although a single-story wall does not have the complexity of a multistory structure, it is advantageous to consider due to the ease of data interpretation. Horizontal cyclic loading was applied to the top of the wall via a 150×75 steel channel as shown in Fig. 2(a), which was fastened to the top of the bond beam by cast-in bolts. The jack was fastened to the strong wall, and the tested wall was stabilized from moving in its out-of-plane direction by two parallel horizontal struts, which were positioned perpendicular to the wall and hinged to the channel and a reaction frame. Fig. 2(b) schematically shows typical wall instrumentation. A load cell to measure the magnitude of the lateral force was placed between the actuator and the steel channel, denoted as device 0 in Fig. 2(b). Portal displacement transducers, denoted as 1 and 2, measured lateral displacement at the top of the wall. Displacements at the window levels were measured by instruments 3 and 4. Portal displacement transducers 47–49 were used to measure sliding of the wall relative to the concrete footing, and transducers 45 and 46 measured the uplift at wall toe positions. Any slip in the steel channel and the concrete footing were measured by transducers 50 and 51, respectively. Further transducers were placed according to the configuration shown in Fig. 2(b) to attain the shear and flexural components of deformation.

Material Properties

Samples were taken from steel reinforcement used in wall construction. These samples were subjected to tensile testing, with the average yield strengths (f_y) for the D12 and D16 reinforcing steel being 305 and 315 MPa, respectively. In addition, masonry compression strength f'_m was determined by material testing of masonry prisms. These prisms were built of three concrete masonry units stacked on top of each other, using the same construction technique as was used for the wall. The prisms were grout filled at the same time as the wall, and were then subjected to the same curing condition as the wall panels. The prisms were tested using an Avery universal testing machine, with f'_m values summarized in Table 1.

Wall Strength Prediction

NZS 4229 Codification of Wall Capacity

NZS 4229 is primarily intended for use by architects and draftspersons, rather than structural engineers. Consequently, a simplified procedure was adopted for the assessment of bracing capacity. The strategy employed in NZS 4229 for proportioning bracing capacity is primarily dependent on wall geometry. The assumption was that the bracing capacity of a masonry wall having penetrations could be determined based on the geometry of individual bracing panels, as demonstrated by the shaded areas shown in Fig. 3, where the geometry of each bracing panel is based upon the vertical dimension of the smallest adjacent opening. The total bracing capacity is then assumed to be the sum of the capacities provided by the individual bracing panels of the wall. The evaluated wall strengths using the NZS 4229 procedure are identified as F_{code} in Table 1. From Table 2, it is evident that for a given panel, the bracing capacity increases as the panel length increases, but diminishes as the panel height increases. This prompted some observers to comment on the influence that a small wall opening would have, as this would effectively generate two bracing panels with a small height and located at either side of the opening, rather than the wall being considered as a single panel that is taller and longer. It was then conceivable that the addition of a small opening to the wall might result in the illogical result that the evaluated capacity of the wall increased.

Prior to testing, the flexural strengths of the masonry walls were evaluated using the bracing capacity values specified in NZS 4229, with the relevant information reproduced in Table 2 for partially grouted 15 series (140 mm thick) concrete masonry. These bracing values were derived by assuming a reinforcement lower characteristic yield stress of $f_y = 300$ MPa and a masonry characteristic compression strength of $f'_m = 8$ MPa, and by treating the panels as vertical flexural cantilevers with a height measured to the center of the fully grouted bond beam. It is emphasized that the conservatism of these NZS 4229 bracing capacities with respect to the experimental results (Brammer 1995; Davidson 1996) was primarily attributed to the actual material strengths being significantly greater than specified, the adoption of a flexural strength reduction factor of $\phi = 0.8$, and a further reduction to

Table 1. Summary of Wall Behavior

Wall number	L_w	h_{op}/h_w	f'_m	F_{code}	$F_{n,st0}$	$F_{n,st1}$	F_{max}	Δ_y	μ_{max}	μ_{ev}	$\frac{F_{max}}{F_{n,st0}}$	$\frac{F_{max}}{F_{n,st1}}$	$\frac{F_{max}}{F_{code}}$	$\frac{F_{max}}{F^*_{code}}$
1	2,600	0.33	16.2	51.8	44.7	46.4	+50.2 −49.0	0.82	+6 −4	>6.0	1.12 1.10	1.08 1.06	0.97 0.95	1.21 1.18
2	2,600	0.5	12.9	37.3	35.9	38.4	+41.2 −38.7	0.57	±4	>6.0	1.15 1.08	1.07 1.00	1.11 1.04	1.23 1.15
3	2,600	0.83	14.4	24.3	28.0	30.8	+33.3 −34.4	1.07	±4	>6.0	1.19 1.23	1.08 1.12	1.37 1.42	1.37 1.42
4	2,600	0.5	16.5	37.3	41.0	44.7	+47.4 −48.8	1.15	+2 −4	4.5	1.16 1.19	1.06 1.09	1.27 1.31	1.27 1.31
5	2,600	0.5	18.9	37.3	41.0 −35.9	44.7 −38.4	+52.4 −50.4	+0.84 −0.66	+4 −6	2.0	1.28 1.40	1.17 1.31	1.40 1.35	1.40 1.35
6	4,200	0.5	16.5	55.9	58.0	78.3	+94.3 −94.6	1.83	+2 −4	2.0	1.63	1.20	1.69	1.69
7	4,200	0.83[a]	18.0	49.4	50.0	62.1 −73.0	+82.8 −82.5	1.89	±4	3.8	1.66	1.33 1.13	1.68	1.68
8	4,200	0.83[a]	18.0	49.4	50.0 −55.0	62.1 −76.3	+82.7 −93.2	1.60	±4	2.0	1.66 1.69	1.33 1.22	1.89	1.89
B1	2,600	0	—	46.0	—	75.0	76.5	1.57	—	5.4	—	1.02	1.66	1.66
B2	4,200	0	—	85.6	—	213.1	179.9	3.60	—	1.0	—	0.84	2.10	2.10
Units	mm	—	MPa	kN	kN	kN	kN	mm	—	—	—	—	—	—

Note: h_{op}/h_w ratio according to the largest opening on the wall; Walls B1 and B2 were tested by Brammer (1995); + denotes push loading; − denotes pull loading.

[a]Refers to magnitude of largest opening.

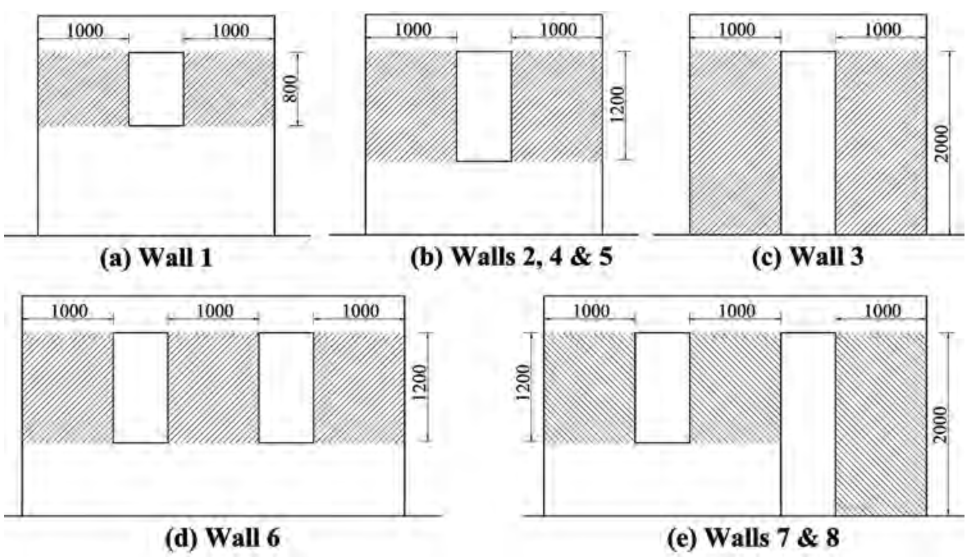

Fig. 3. Identification of bracing panels

Table 2. Bracing Capacities (kN) for 140 mm Partially Grouted Concrete Masonry (from NZS 4229)

Panel height (m)	Panel length (m)								
	0.8	1.2	1.6	2.0	2.8	3.6	4.4	5.2	6.0
0.8	19.3	32.5	50.3	71.3	125.3	155.5	222.8	302.0	393.5
1.2	13.8	23.5	36.5	51.8	91.3	113.3	162.5	220.8	287.5
2.0	9.0	15.3	24.0	34.0	60.3	75.0	107.8	146.5	191.3
3.0	6.3	10.8	17.0	24.5	43.5	54.3	78.0	106.3	139.0

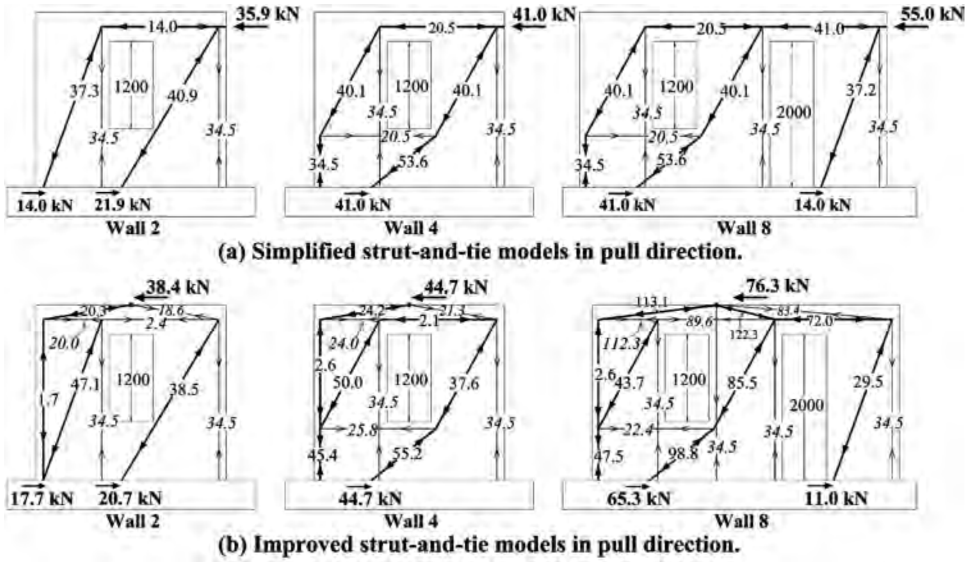

Fig. 4. Wall strength analysis models

80% of the evaluated capacity for panels having a length greater than 3.0 m. In addition, in all cases the calculation assumed the vertical ∅ 12 mm reinforcement to be distributed at a maximum spacing of 800 mm (where possible) or for bars to be spaced in the least favorable positions, resulting in the most conservative estimate of flexural strength.

Simple Strut-and-Tie Models

Two types of strut-and-tie models were employed to evaluate wall strengths (Yanez et al. 1991; Wu and Li 2003). The first type was a simplified strut-and-tie model, which assumed that all panels were pinned at the bond beam center and that lateral force was applied to the bracing panels at the center of the bond beam. These assumptions corresponded directly with those assumed in NZS 4229. In addition, the effect of wall self-weights was not considered in this simplified strut-and-tie model in order to ease the analysis process. The resultant strut-and-tie analyses using this simplified procedure are diagrammatically shown in Fig. 4(a) for a selection of walls loaded in the pull direction, where the strut components are indicated by a broader element thickness. It is illustrated in Fig. 4(a) that the introduction of extended trimming reinforcement beneath the window in Wall 4 results in an increase in wall strength when compared to that predicted for Wall 2. This is due to the change of slope of the diagonal strut in the left panel of Wall 4. The evaluated lateral wall strengths using this simplified strut-and-tie analysis procedure are identified as $F_{n,\mathrm{st0}}$ in Table 1.

Improved Strut-and-Tie Models

A second set of strut-and-tie models considered the lateral force to be applied as a single point load at the center of the wall top, which more accurately represented the loading system shown in Fig. 2(a). These models are referred to here as "improved" to clearly delineate them from the "simple" models previously discussed. The lateral force was transferred from the wall top to the bond beam center through a triangular truss, which was subsequently applied to the bracing panels. Unlike the simplified models presented in Fig. 4(a), the wall self-weight of 1.6 $\mathrm{kN/m^2}$ was considered to act along the bond beam center in the improved strut-and-tie models. The resultant strut-and-tie analyses using the above mentioned procedure are diagrammatically shown in Fig. 4(b) for pull direction loading.

Similar to the simplified strut-and-tie analysis procedure discussed earlier, an increase in predicted strength is illustrated in Fig. 4(b) when extended trimming reinforcement is included in Wall 4. The predicted lateral wall strengths using the improved strut-and-tie models are identified as $F_{n,\mathrm{st1}}$ in Table 1, where it is shown that the improved modeling procedure resulted in predicted strength increases of between 4 and 10% for the 2.6 m long perforated masonry walls. For the 4.2 m long walls with two openings, where double bending of the central pier was also modeled in the improved strut-and-tie formulation, the predicted strength increased by 23 to 52%.

Shear Strength

Wall shear strength was evaluated using the NZS 4230 (NZS 2004) procedure (Voon and Ingham 2007) and was found to be substantially in excess of the lateral force required to generate the nominal flexural strength.

Experimental Results

General wall behavior is summarized in Table 1, where $F_{\max}$ corresponds to the maximum wall strength measured in the test, and Δ_y is the evaluated yield displacement of the tested walls. $\mu_{\max}$ is defined as the displacement ductility level at which maximum strength was measured and μ_{av} is the experimentally determined available displacement ductility factor. It is important to note that NZS 4229 assumes maximum seismic demands and capacities based upon a maximum displacement ductility of $\mu = 2$ (Ingham et al. 2001). Consequently, wall response exceeding $\mu = 2$ represents superior wall response and correspondingly conservative seismic design.

The experimentally obtained force-displacement curves for masonry walls are presented in Fig. 5, depicting the lateral displacement at the top of the walls as a function of applied lateral shear force. Due to the lack of horizontal shear reinforcement and the fact that the walls were partially grout filled, all test walls were observed to fail in a diagonal tension mode. The general nature of the force-displacement responses presented in Fig. 5 are notably similar to those reported by Brammer (1995) and Davidson (1996). From the force-displacement histories illustrated in Fig. 5, a number of general characteristics of the masonry walls can be identified:

1. The maximum strength was typically developed during the first excursion to $\mu = 4$. Following this, cracking became significant in some walls and strength degradation began.
2. Despite the presence of widely open diagonal cracks, the partially grouted walls exhibited only gradual strength and stiffness degradation, and in no case did any wall suffer from sudden failure. This desirable behavior of the nominally reinforced partially grouted masonry walls with openings was due to the solid filled bond beam at the top of the walls, which caused a frame-type action at a later stage of testing.
3. The force-displacement plots consistently illustrated a pinched shape. This was primarily due to the presence of significant shear deformation in this type of masonry construction. Less hysteretic energy was expended during the second cycle to any displacement level, when compared with the first displacement cycle. This is illustrated by the more pinched hysteresis loops of the second cycle.
4. From the wall cracking patterns diagrammatically shown in Fig. 6, it is clearly illustrated that the absence of major damage in the solid grout-filled bond beam supported the notion of frame-type action being developed at a later stage of testing. This led to considerable inelastic displacement capacity of the partially grouted masonry walls, where μ_{av} was measured to consistently be above 2.0.

The μ_{av} values recorded in Table 1 show that the largest recorded ductility capacity for the perforated masonry walls corresponded to a wall length of 2,600 mm, and reduced significantly for the 4,200 mm long perforated masonry walls. It was observed during experimental testing that the 4,200 mm long masonry walls displayed greater cracking than the 2,600 mm long walls. Consequently, it was deduced that the lower observed ductility rating for the 4,200 mm long walls occurred because of the rapid-developing wide cracks that contribute to shear displacement, accelerating initiation of the diagonal tension mode of failure and subsequent strength degradation.

From Fig. 5(a), it was observed that Wall 1 did not achieve the bracing capacity prescribed by NZS 4229 (denoted F_{code}). NZS 4229 overpredicted the lateral strength of this perforated wall by about 3.3 and 5.4% in the respective push and pull directions. However, it is illustrated in Fig. 5 that the conservatism of NZS 4229 increases with the height of opening, and for a full height opening (e.g., a door in Wall 3) the NZS 4229 prediction had significant conservatism. Consequently, it was concluded that NZS 4229 is only nonconservative for window openings having a height of less than 1.2 m, though unfortunately this probably accounts for a significant proportion of window geometries.

Due to the lack of distributed horizontal shear reinforcement and the fact that the wall was partially grout filled, all test walls were observed to fail in diagonal tension mode. This type of failure was characterized by the development of early horizontal flexural cracking, which was later superseded by wide diagonal cracks that extended throughout the wall panels. The cracking patterns for these walls are depicted diagrammatically in Fig. 6, with the shaded areas indicating masonry crushing. From Fig. 6 it may be observed that the diagonal cracking patterns on the perforated concrete masonry walls aligned well with the load paths by which shear force was transferred to the foundation in the strut mechanisms of Fig. 4.

Discussion

The primary objective of this study was to validate the adequacy of NZS 4229 in addressing the bracing capacity of masonry walls containing openings. As shown in Fig. 1, design of the perforated masonry wall specimens was conceived to facilitate comparison of wall behavior between two or more walls with respect to variation of a given dimension of wall geometry.

Height of Openings

Test results successfully illustrated correlation between the reduction of wall strength and the increasing height of wall openings (h_{op}). This reduction of wall strength is also identified in the force-displacement envelopes presented in Figs. 7 and 8. In Fig. 7, it is shown that the lateral strength of the 2,600 mm long walls reduced from the maximum of 76.5 kN in the case of Wall B1 without an opening [reported by Brammer (1995)], to 50.2 kN when a window opening of 600×800 was included in Wall 1. The same figure also shows further reduction of wall strength to 41.2 kN and 34.4 kN when the height of openings was increased to 1,200 mm and 2,000 mm in Walls 2 and 3, respectively. As diagrammatically illustrated in the pull direction strut-and-tie models presented in Fig. 9, the further reduction of strength in Walls 2 and 3 (compared to Wall 1) was because of the more steeply inclined diagonal strut of the right piers as the height of openings increased. This resulted in a reduction of the horizontal shear component that could be resisted by the right pier, consequently leading to the overall reduction of lateral strength in Walls 2 and 3.

Similarly, the reduction of lateral strength in the 4,200 mm long walls is evident in Fig. 8. As compared to the strength recorded in Wall B2 [reported by Brammer (1995)], it is shown that the wall lateral strength was almost halved when openings were introduced on the 4,200 mm long masonry walls. In addition, by comparing the lateral strengths of Walls 6 and 7, it is shown that the introduction of a door opening resulted in the reduction of strength from 94 to 83 kN in Wall 7 (about 12% reduction of strength). Note that test results of Walls 4, 5, and 8 are not included in Figs. 7 and 9 because the detailing of trimming reinforcement in these walls differed from that specified in NZS 4229.

Effect of Trimming Reinforcement

Experimental results illustrated that the use of extended D16 trimming reinforcement affected wall strength considerably. This increase of wall strength can also be identified in the force-displacement envelopes presented in Figs. 10 and 11. Fig. 10 shows the force-displacement envelopes for Walls 2, 4, and 5. These three partially grout-filled masonry walls were constructed to identical geometries and consisted of identical longitudinal and bond beam reinforcement, with the only difference being the length of trimming reinforcement used in each wall. The trimming reinforcement in Wall 2 (see Fig. 1) was detailed according to the specifications of NZS 4229, but Walls 4 and 5 were detailed with extended trimming reinforcement. As shown in Fig. 1, the

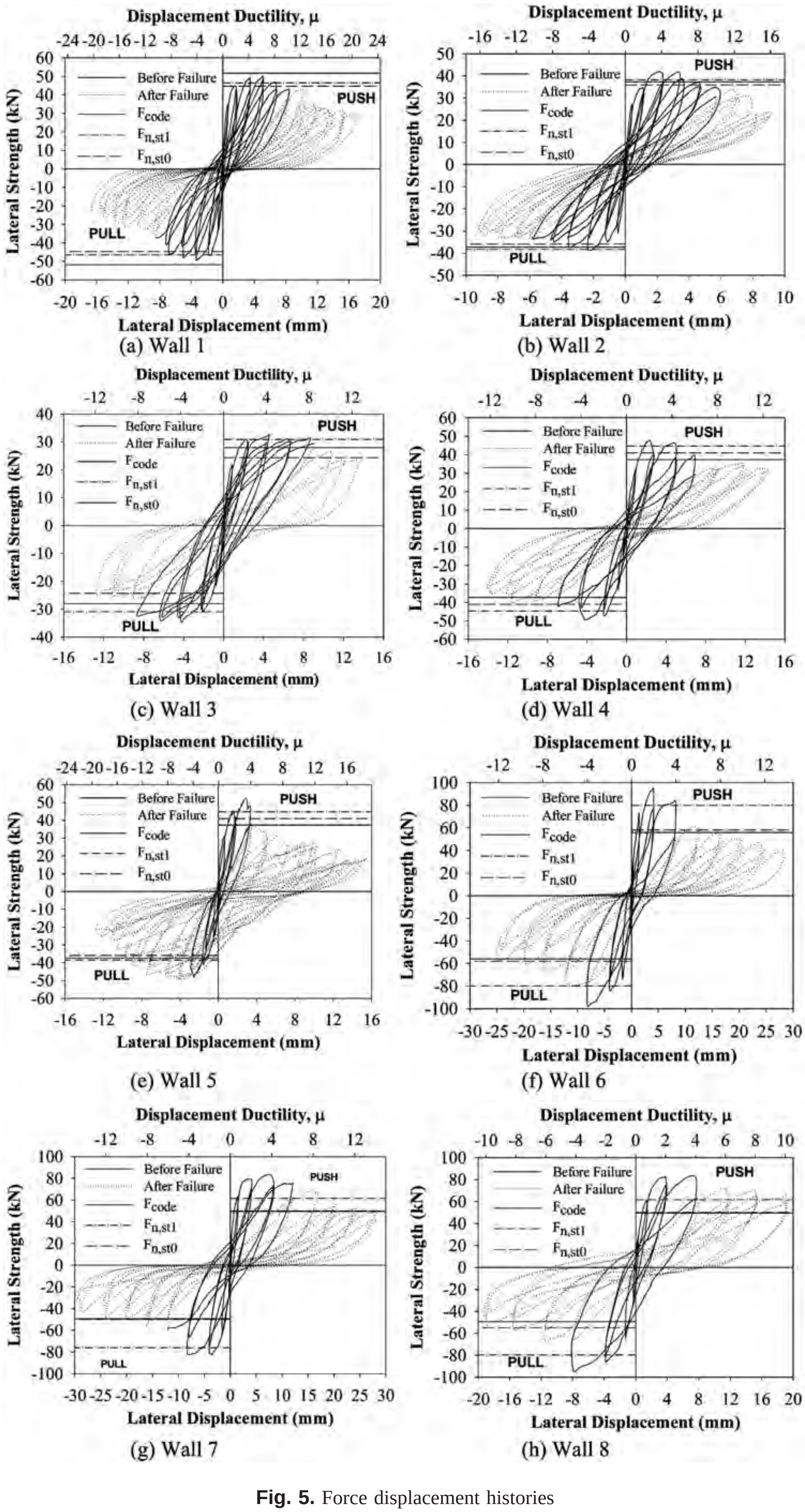

Fig. 5. Force displacement histories

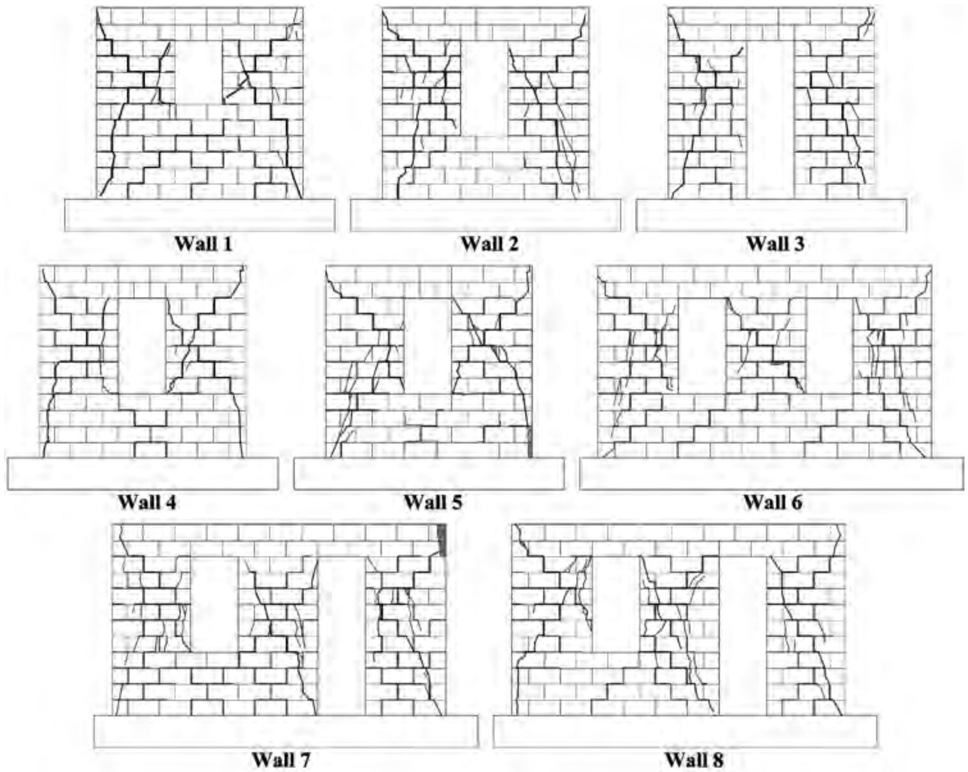

Fig. 6. Masonry wall cracking patterns

trimming reinforcement in Wall 5 was only extended to the outermost vertical reinforcement on one side of the wall, therefore resulting in higher strength being predicted in the push direction than in the pull direction.

The force-displacement envelopes in Fig. 10 clearly illustrate the increase of lateral strength from the maximum of 41.2 kN for Wall 2 to 47.7 and 52.4 kN when the trimming reinforcement was extended beneath the window opening in Walls 4 and 5, therefore resulting in strength increases of about 18% and 27%, respectively. Similar to Fig. 10, the force-displacement envelopes in Fig. 11 illustrate an increase in wall pull strength from 82.5 kN for Wall 7 to 93.2 kN when the trimming reinforcement of Wall 8 was extended to the outermost vertical reinforcement in the left pier (see Fig. 1), resulting in a pull direction strength increase of about 12%.

Wall Strength Prediction

The test results presented in Table 1 clearly demonstrate that the size of openings and the arrangement of trimming reinforcement significantly affect the lateral strength of perforated masonry walls. For the small window opening in Wall 1, the measured strength was slightly less than that prescribed by NZS 4229, resulting in $F_{\max}/F_{\text{code}}=0.95$. However, the results in Fig. 5 indicate that the conservatism of NZS 4229 increases with the height of opening, and for a full height opening (e.g., a door in Wall 3), the NZS 4229 prediction had significant conservatism.

Shown also in Fig. 5 is that a 50% increase in the number of piers for the 4,200 mm long walls resulted in approximately 100% increase in lateral strength when compared to the strengths recorded for the 2,600 mm long walls (compare Wall 6 to Wall 4). This is explained by considering the wall crack patterns shown in

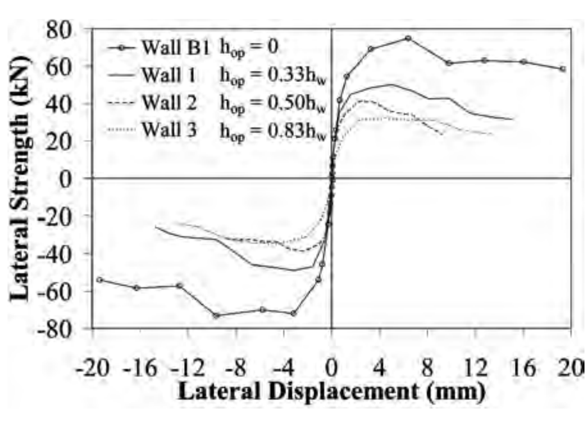

Fig. 7. Effect of opening on 2,600 mm long walls

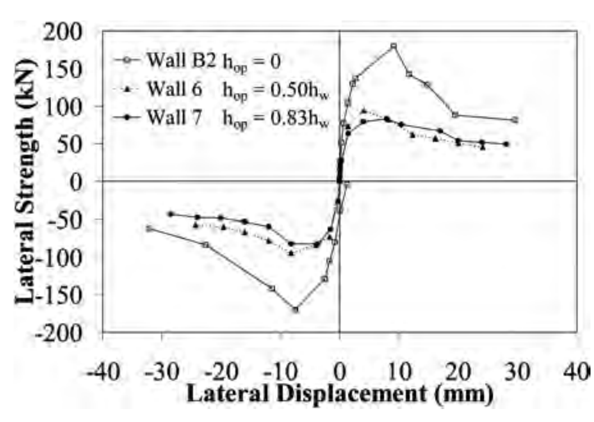

Fig. 8. Effect of openings on wall strength (shown for pull direction)

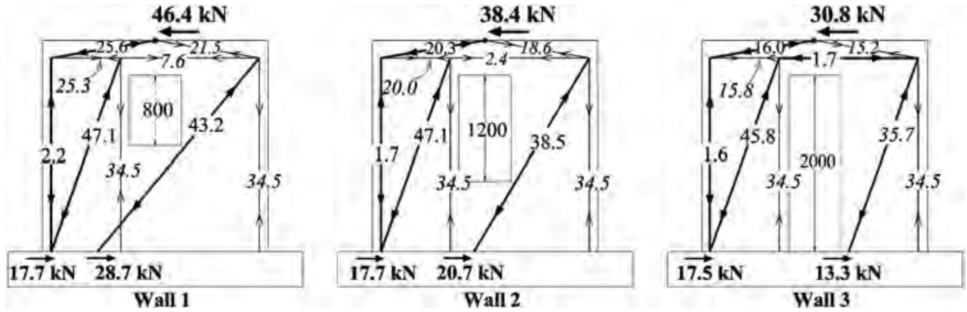

Fig. 9. Effect of openings on 4,200 mm long walls

Fig. 6, where it can be seen that diagonal cracks passed through the exterior lintel-pier joints of all walls, effectively resulting in a pinned connection at those locations, whereas all interior lintel-pin joints remained uncracked during testing and hence were capable of moment resistance. It is therefore established that double bending of the central pier significantly increased the lateral strength of perforated walls having interior piers. Consequently, failure of NZS 4229 to account for the extra strength generated by double bending of the central pier resulted in significant underprediction of strengths recorded in Walls 6–8. As shown in Table 1, ratios of $0.95 \leqslant F_{\max}/F_{\text{code}} \leqslant 1.40$ and $1.68 \leqslant F_{\max}/F_{\text{code}} \leqslant 1.89$ were observed for the 2,600 mm and 4,200 mm long perforated walls included in this study. This observation subsequently led to the conclusion that NZS 4229 is only nonconservative for walls containing a single opening with a height of less than 1,200 mm, but that the standard has considerable conservatism for walls that have more than one opening due to the inner piers undergoing double bending.

Comparisons of $F_{\max}/F_{n,\text{st0}}$ and $F_{\max}/F_{n,\text{st1}}$ are presented in Table 1. It is shown that the simplified strut-and-tie method adequately predicted the lateral strength of the 2,600 mm long perforated masonry walls, with $F_{\max}/F_{n,\text{st0}}$ varying from 1.12 to 1.28 (excluding the experimental result obtained in the pull direction for Wall 5). However, the effectiveness of this method was significantly reduced when predicting the strengths of the 4,200 mm long masonry walls included in this study. This was shown by the consistent underprediction of strengths for Walls 6–8 by about 60% when the simplified strut-and-tie method was used. Similar to NZS 4229, this underprediction of wall strength was due to the fact that double bending of the central pier was not accounted for in the simplified strut-and-tie models.

To address this identified fault in NZS 4229 it is proposed that a strength reduction factor of 0.8 be applied to panels having a height of 0.8 m and a similar factor of 0.9 be applied to panels having a height of 1.2 m. These recommended changes have been incorporated into Table 3, and are also identified in Table 1 by the ratio $F_{\max}/F^{*}_{\text{code}}$. From Table 1, it may be established that these recommended alterations to the standard then result in comparable levels of conservatism for the range of window openings considered in this study.

For the improved strut-and-tie method, it is illustrated in Fig. 3 that significantly improved strength predictions were attained when double bending of the central pier and when wall self-weight were considered in models presented in Fig. 4(b), resulted in average $F_{\max}/F_{n,\text{st1}}$ values of about 1.10 for the 2,600 mm long walls (excluding Wall 5 pull direction) and 1.22 for the 4,200 mm long masonry walls. The diagonal cracking patterns illustrated in Fig. 6 were observed to align well with the load paths by which lateral force was assumed to be transferred to the foundation in the strut mechanism. This observation supports use of the strut-and-tie method as the tool to evaluate the strength of nominally reinforced masonry walls with openings.

Conclusions

It was observed that the perforated partially grouted concrete masonry walls tested at the University of Auckland exhibited gradual strength and stiffness degradation, and in no case did any wall suffer from sudden failure. This desirable behavior of the nominally reinforced partially grouted masonry walls with openings was attributed to the solid filled bond beam at the top of the walls,

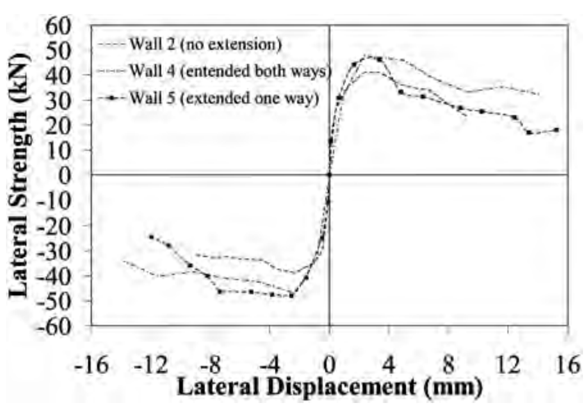

Fig. 10. Effect of trimming reinforcement on 2,600 mm long perforated masonry walls

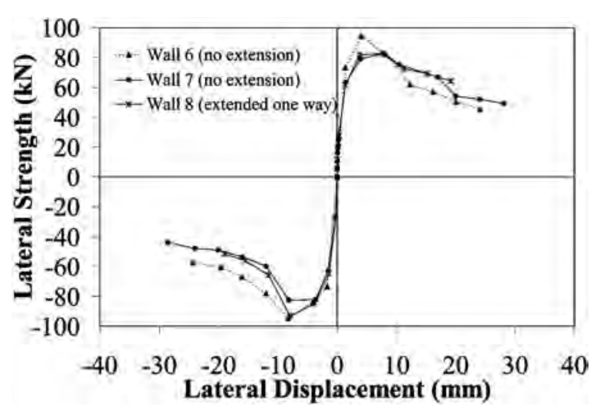

Fig. 11. Effect of trimming reinforcement on 4,200 mm long perforated masonry walls

Table 3. Recommended Bracing Capacities (kN) for 140 mm Partially Grouted Concrete Masonry

Panel height (m)	Panel length (m)								
	0.8	1.2	1.6	2.0	2.8	3.6	4.4	5.2	6.0
0.8	15.4	26.0	40.2	57.0	100.2	124.4	178.2	241.6	314.8
1.2	12.4	21.1	32.9	46.6	82.2	102.0	146.3	198.7	258.8
2.0	9.0	15.3	24.0	34.0	60.3	75.0	107.8	146.5	191.3
3.0	6.3	10.8	17.0	24.5	43.5	54.3	78.0	106.3	139.0

which caused frame-type action at latter stages of testing and permitted double bending to develop in the central piers of walls with two openings. This led to considerable inelastic displacement, where μ_{av} was consistently measured to be above 2.0. In addition, it was observed that maximum strength was typically developed during the first excursion to $\mu=4.0$. Following this, cracking became significant in the masonry walls and strength degradation began.

The test results clearly demonstrated that the size of openings significantly affected the lateral strength of the tested walls. It was shown that the reduction of wall strength corresponded to the increased height of an opening. This reduction of strength was because of the steepened diagonal strut when the height of openings increased. This in turn led to a reduction of the horizontal shear component that could be resisted by the masonry piers, which resulted in the overall reduction of lateral strength in the perforated masonry walls. In addition, it was demonstrated that extension of the trimming reinforcement below the window had the effect of increasing wall strength. It was also observed that the wall cracking pattern was altered when the trimming reinforcement was extended below the window opening.

It was established that NZS 4229 fails to correctly identify the geometry of bracing panels of perforated masonry walls, resulting in the overprediction of strength of walls containing a small opening. This was rectified by proposing a reduction to the assigned capacities of bracing panels having a height not greater than 1.2 m. It was shown in the experimental study that the conservatism of NZS 4229 increased when the height of opening is increased and when a wall contains more than one opening.

The diagonal cracking patterns on the perforated masonry walls were observed to align well with the load paths by which shear force was assumed to be transferred to the foundation in the strut mechanism. This observation supported use of the strut-and-tie method of analysis as the tool to evaluate the strength of nominally reinforced masonry walls with openings. Strength prediction using the improved strut-and-tie method was found to closely match the experimental results of walls having more than one opening. Strength prediction by the simplified strut-and-tie method was found to closely match the test results of masonry walls with a single opening, but significant underestimation of strength by this method was found for walls with two openings

Acknowledgments

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Notation

The following symbols are used in this paper:

F_{code} = NZS 4229 code specified wall nominal strength;
F^*_{code} = updated NZS 4229 code specified wall nominal strength including correction factor for openings of 0.8 m and 1.2 m;
$F_{\max}$ = maximum experimentally measured strength;
F_n = nominal flexural strength;
$F_{n,\text{st0}}$ = nominal wall strength according to simplified strut-and-tie model;
$F_{n,\text{st1}}$ = nominal wall strength according to improved strut-and-tie model;
f'_m = masonry compressive strength;
f_y = yield strength of reinforcement;
h_{op} = height of opening;
h_w = height of wall;
L_w = length of wall;
Δ_y = nominal yield strength;
μ = displacement ductility level;
μ_{av} = available displacement ductility factor;
$\mu_{\max}$ = displacement ductility level corresponding to maximum strength; and
ϕ = strength reduction factor.

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5.1 Veneer Walls

General

NOTE: ***This section has been revised to meet the provisions of E2/AS1 and E2/AS3.***

Cavity wall construction has been recognised as an excellent means of offering the greatest weather resistance.

The inner skin of such construction traditionally has been considered the structural member and therefore designed to carry all horizontal, vertical and seismic loads. In residential construction in New Zealand the timber frame has been fulfilling a structural role and arising from this, there has developed the veneer concept of the outer skin of masonry and an inner timber frame.

The principal advantage of using such construction is the excellent weather resistant construction not incurring any significant maintenance costs.

While traditionally the cavity width has been maintained at a minimum of 40 mm, new technology involving the use of a water resistant inner lining has resulted in the development of alternative systems.

A range of typical veneer construction is shown in Figure 1 (page 2).

An alternative form has been the use of two veneer skins to create a permanent formwork system. The structure of the system is provided by a reinforced grouted core.

Codes

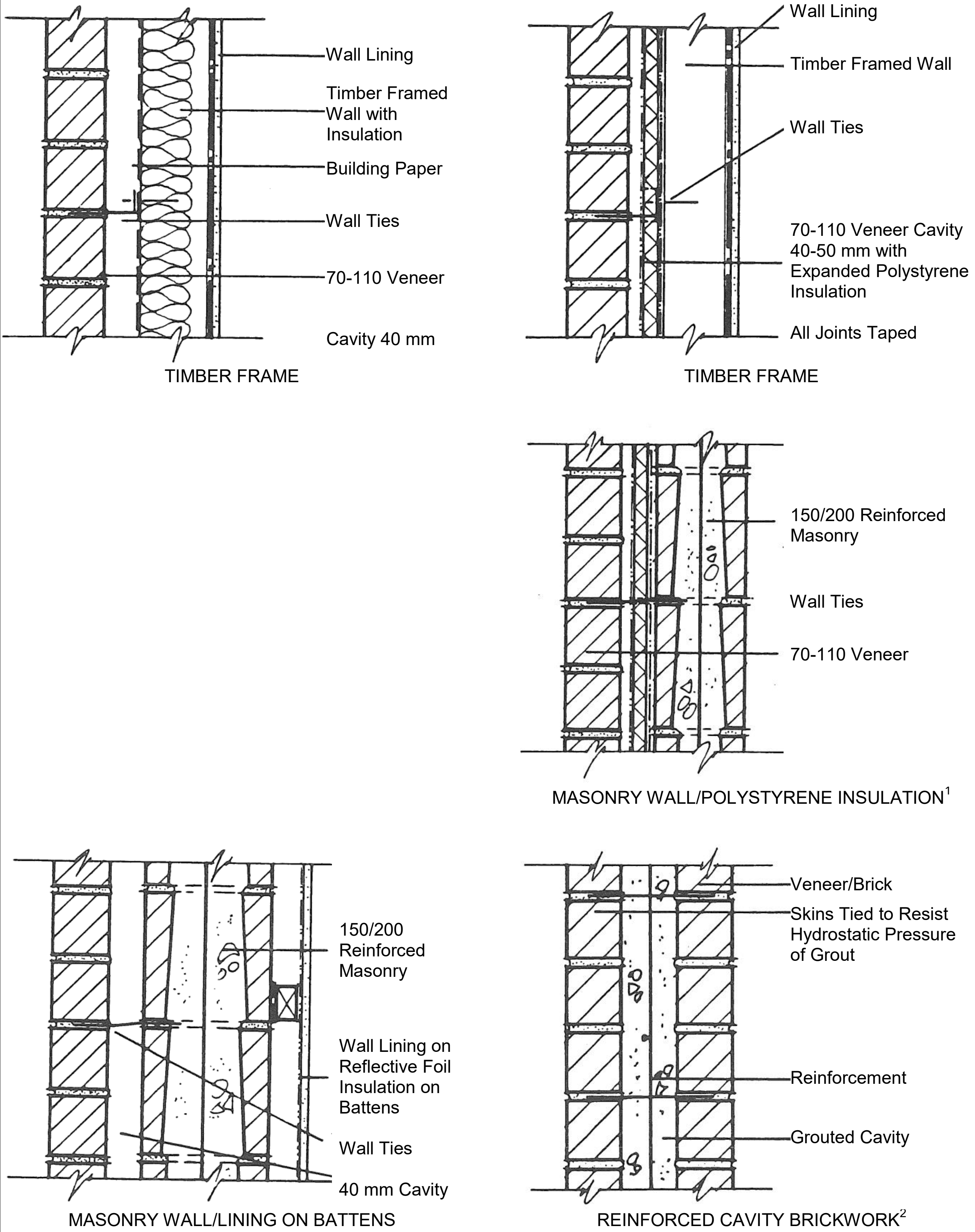
Concrete Masonry Wall Veneers are the subject of not one but several New Zealand Standards, mainly:

- AS/NZS 4455.1 *Masonry units, pavers, flags and segmental retaining wall units - Masonry units*
- NZS 4210 *Masonry construction: Materials and workmanship*
- NZS 4229 *Concrete masonry buildings not requiring specific engineering design*
- NZS 4230 *Design of reinforced concrete masonry structures*
- NZS 3604 *Timber framed buildings*
- New Zealand Building Code: E2/AS1 and E2/AS3

The principal code for veneer materials and workmanship is NZS 4210.





[1] Where a cavity is less than 40 mm, the construction becomes Specific Engineering Design.
[2] This construction is Specific Engineering Design.

Figure 1: Veneer Walls – Range of Typical Veneer Construction

Reinforced Veneers

Reinforced veneers are covered by NZS 4230 requiring walls to comply with Section 7 of that Standard in relation to thickness.

Unreinforced Veneers

Unreinforced veneers tied to walls of structures built within the provisions of NZS 4229 or NZS 3604 are limited in height.

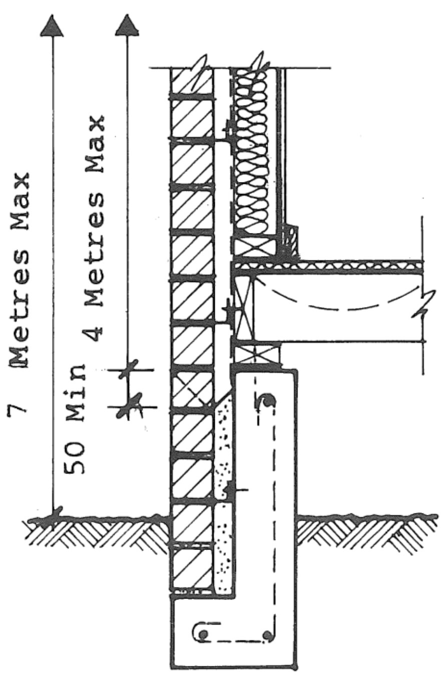
The principal details of the Standards are illustrated in Figures 2 and 3.

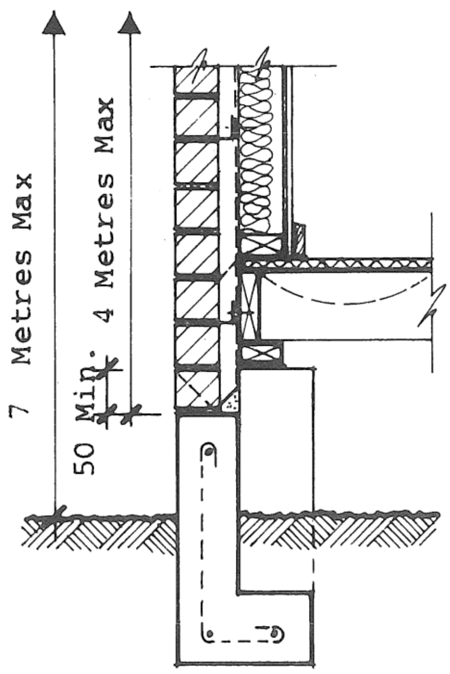
Figure 2 details the particular aspect of the measurement of the height of veneer walls from the top of the supporting foundation.

Figure 3 (page 4) deals with Non Specific Design Construction, (i.e. NZS 4229 and NZS 3604).

Note that Specific Design allows greater wall heights.

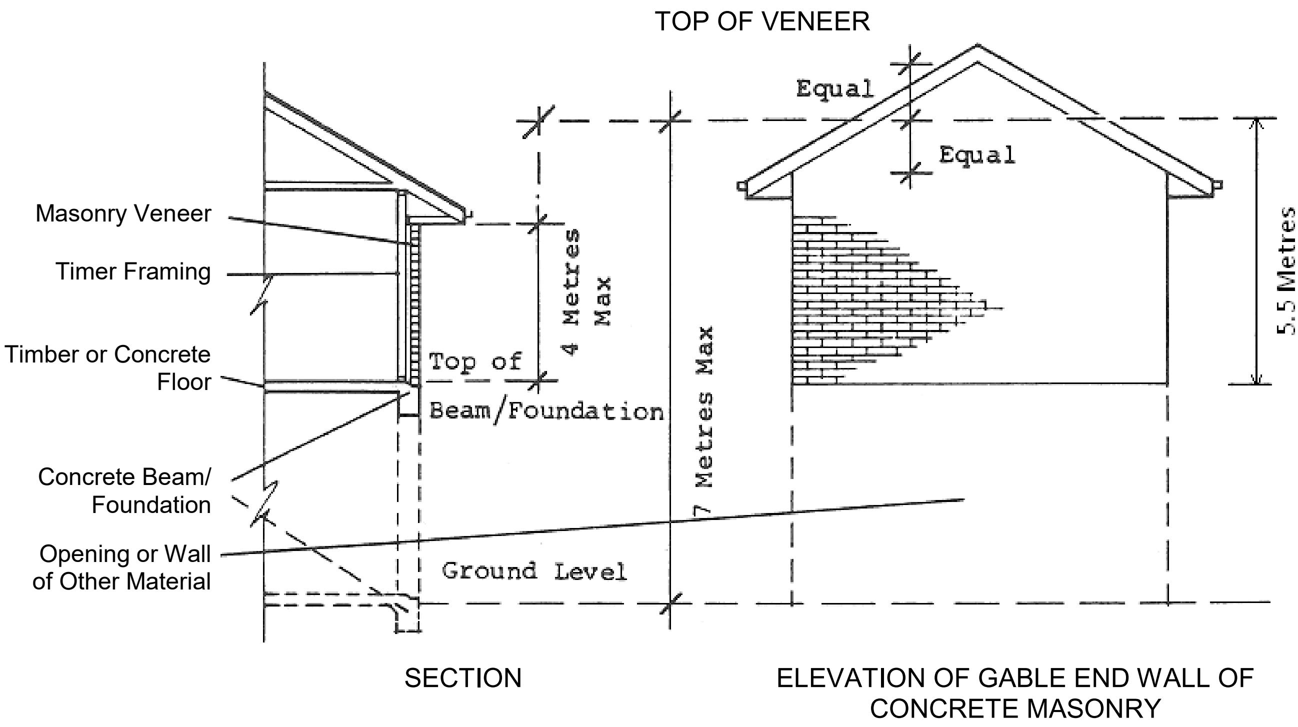
TO TOP OF THE VENEER*



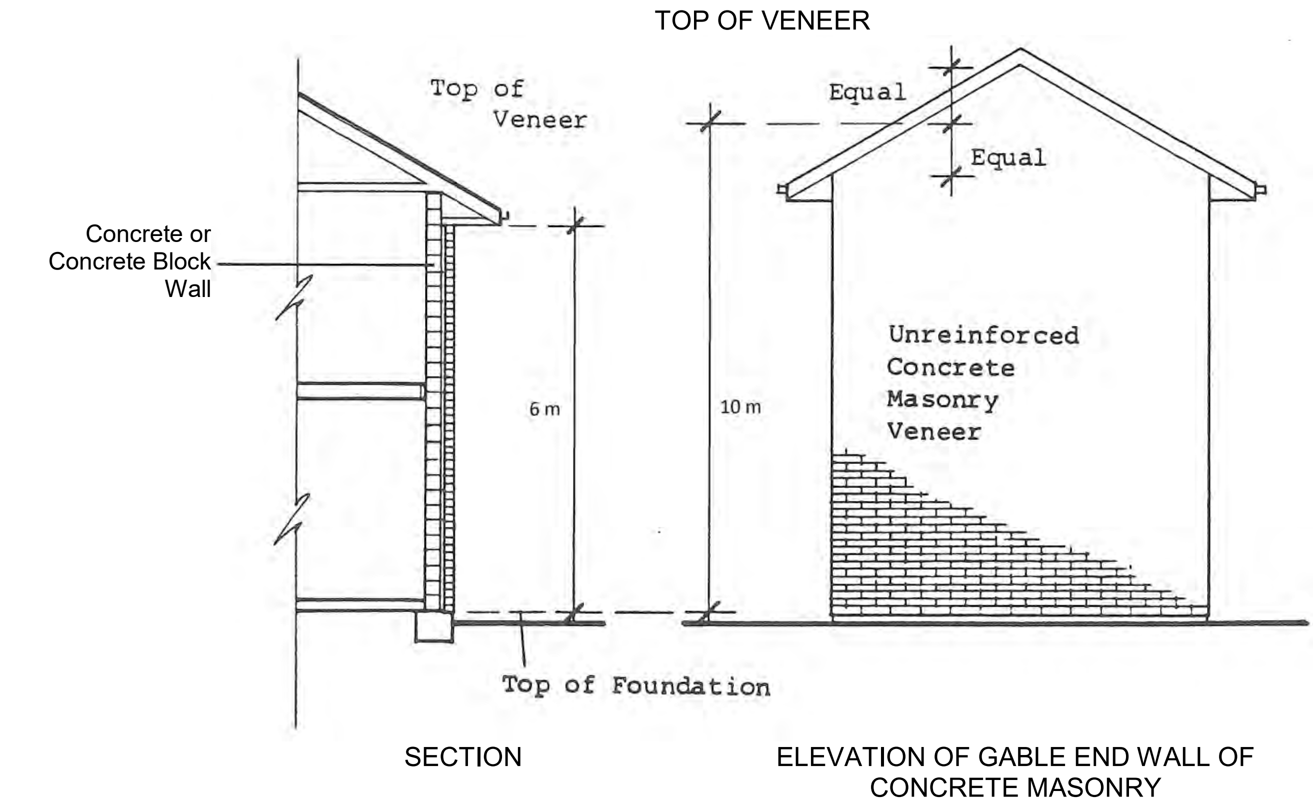


** The maximum veneer height is 5.5 m on a gable end wall*

Figure 2: Determination of Veneer Heights



Diagrams illustrating maximum height of unreinforced masonry veneer attached to timber framed wall as permitted by NZS 3604.



Diagrams illustrate maximum height of unreinforced masonry veneer attached to structural masonry or reinforced concrete wall as permitted by NZS 4229. If the gable wall adjoined on egress way than the height was restricted to 6 m.

Figure 3: Buildings NOT requiring specific design

Veneer Construction

Veneer Thickness

Minimum thickness of veneer is 70 mm. This follows changes introduced by the clay brick industry.

Maximum mass of veneer is 220 kg/m^3.

The New Zealand Concrete Masonry Association members confirm that the manufacturing standard for their veneer/bricks follow AS/NZS 4455.1. The normal bonding pattern lies between one quarter to one half.

The minimum length of a veneer return is 230 mm.

Cavity

The maximum width is 75 mm and the minimum width is 40 mm unless there is included some additional waterproofing.

Drainage must be provided at the base of each cavity. This is done most usually by providing weep holes through the lowest vertical joint. Typical details, such as F4, are shown in the Construction Section (3.3).

The detail indicates that weep holes are provided at 800 mm centres and that a step of 50 mm minimum is required from the underside of the veneer to the level of the inside slab.

A cement mortar flaunching or fillet is often used to direct cavity moisture to the outside.

While in some cases moisture from the cavity can be dissipated into the sub floor space, it is recommended that sub floor ventilation is not directly connected into the wall cavity.

Moisture Control

An overriding advantage of veneer walls is that any moisture penetration is halted by the cavity. Excess moisture must be drained away as described above.

Care must be exercised not to introduce moist air into the cavity and certainly not discharge air from the cavity into the roof space.

Air from the underfloor space of a timber floor should not be allowed to enter the cavity. The head of each cavity should be sealed and ventilating holes provided through the top veneer vertical joints. Adequate ventilation of the cavity can reduce possible surface efflorescence problems.

Under no circumstances should a drying cabinet, shower room ventilation or cooker vents be discharged into the cavity.

Wall Ties

The ties should be non-corrodible as defined in AS/NZS 2699.1:2000 Built-in components for masonry construction - Wall ties and NZS 4210, and embedded at least 50% of the veneer thickness. The cover to the outside edge of the tie should not be less than:

- 25 mm galvanised ties
- 15 mm stainless steel ties
- 10 mm plastic ties.

Ties are classified as Light, Medium and Heavy duty stiff ties and flexible ties. The latter are only used in specific design circumstances and require special joint details similar to that described on page 10, to allow the structural frame to move relative to the veneer.

Ties shall comply with the durability of Table 1:

Table 1: Table E1 - Protection for masonry veneer ties supporting masonry veneer using AS/NZS 2699.1 (see E5.2).

Location (NZS 3604 Exposure Zones)	Grades 316, 316L or 304 stainless steel	470 g/m^2 galvanising on mild steel
Zone B	Yes	Yes
Zone C	Yes	Yes
Zone D	Yes	No

Design performance criteria are available from the manufacturers of the special flexible ties, which includes various sliding joint details for window and door frames.

The conventional EM tie has maximum spacing limits as shown in Figure 4. EM ties can be used for veneer not exceeding 220 kg/m^2 in earthquake Zones 1 and 2 and 180 kg/m^2 in Zone 3.

Maximum Spacing

H x V	Timber Stud:	600 x 350 mm 450 x 450 mm
H x V	Concrete/Masonry:	600 x 400 mm

More details on earthquake zoning and spacing of ties is given in Section 5.2.

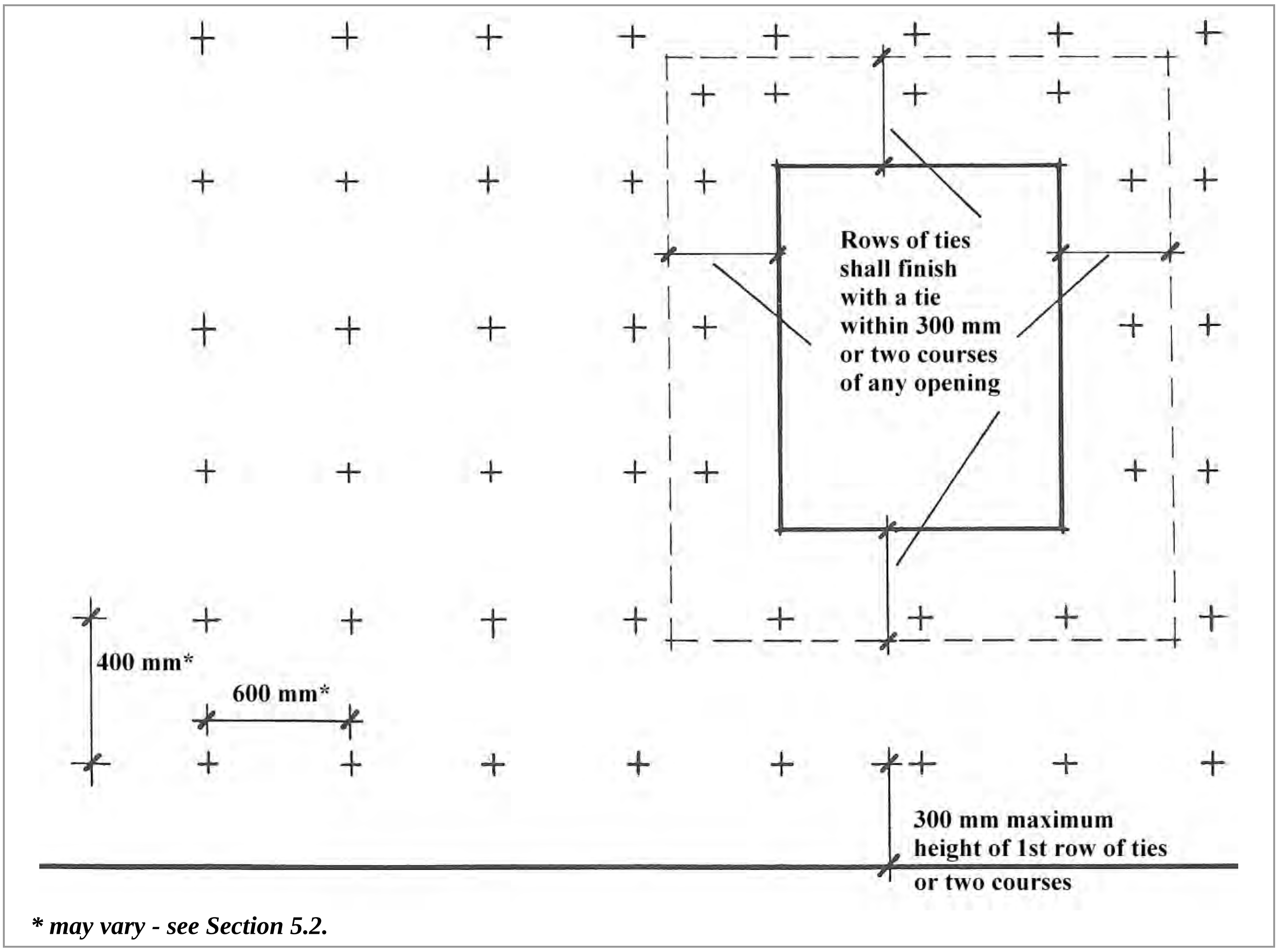


Figure 4: Spacing of Wall Ties

The current NZS 4210 and E2 AS1 require ties to be bedded within the mortar joint. However, the industry has been 'dry bedding' ties for many years. The practice has been structurally checked by BRANZ and found to be satisfactory. It is now accepted in NZS 4229 Appendix E 2013.

Where the veneer sits on a supporting or damp proof metal flashing, ties need to be placed in the next two courses above this position (see Figure 5). Where the veneer sits on a bitumen painted concrete surface at ground level, only one row of ties is needed in the first available course.

Control Joints

Concrete masonry veneer blocks, like other solid building materials, are subject to expansion and contraction from various causes including:

(i) Moisture change (principal factor)
(ii) Temperature change
(iii) Movement of other structural elements

In a long veneer wall the expansion/contraction may cause cracks to appear. Although such cracks may not be serious from a structural point of view, they can be unsightly.

Various rules for locating control joints have been developed from experience, and will probably continue to be refined.

Since there are many possible layouts of walls with their openings for windows, etc., some judgement must be used by the designer in determining the location of these joints.

Control joints are continuous, vertical joints built into the veneer, to relieve stresses which may occur. The most common requirement is for shrinkage control joints.

The predominant movement of a concrete masonry veneer is shrinkage which is a characteristic different to clay brick veneer where expansion is likely.

In long walls (unbroken by window openings, etc.) joints are usually spaced to produce panels having lengths of approximately 1½ to 2 times their height, i.e. 5 to 6 metres.

When there is a bonded corner (return end) then one side of the panel can be regarded as fixed. In this case the panel length should be reduced (Figure 6).

Window and door openings up to 1.8 m width require a vertical control joint inline with only one jamb (Figure 7), but wider openings should have joints at both sides (Figure 8).

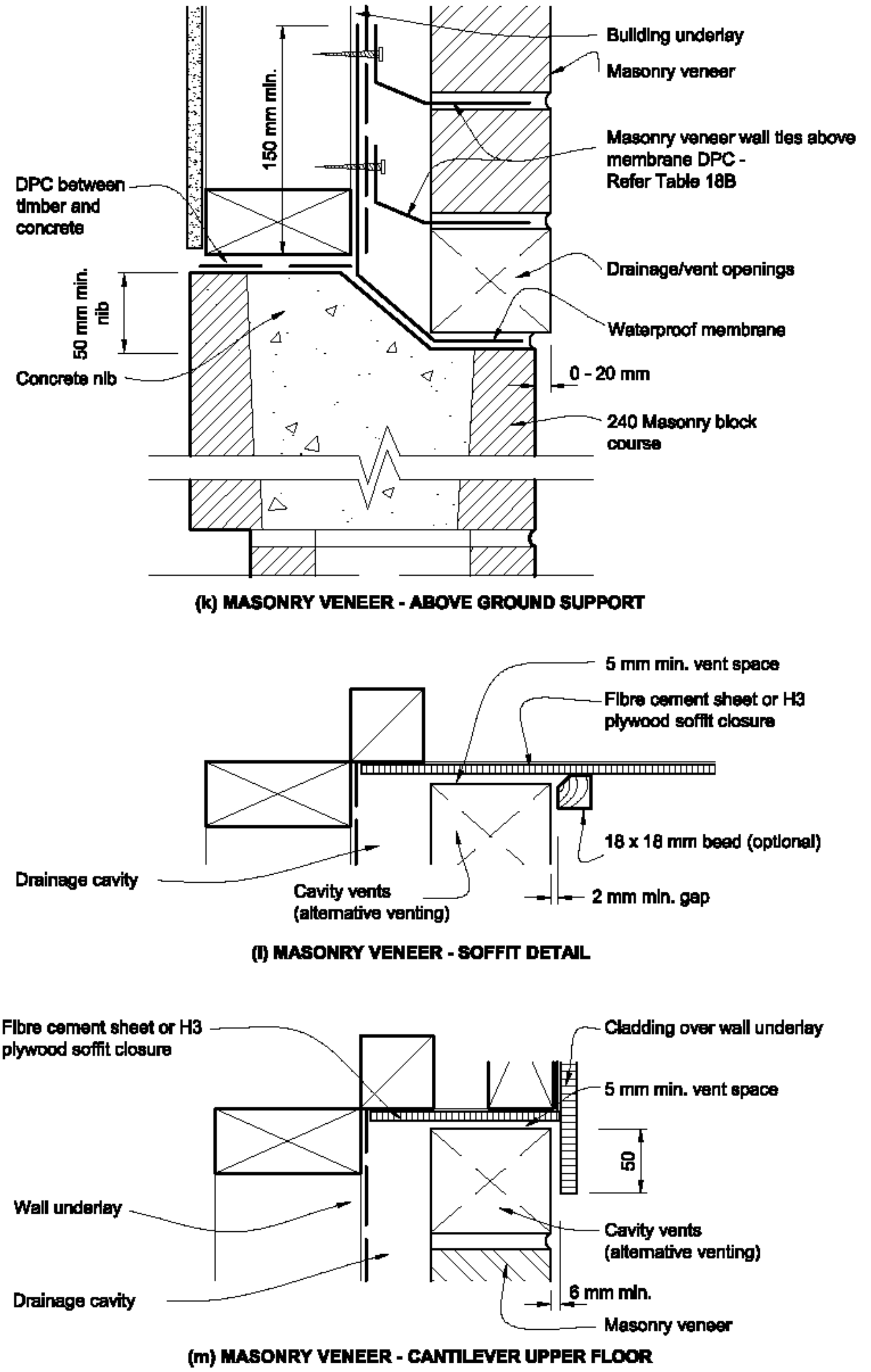
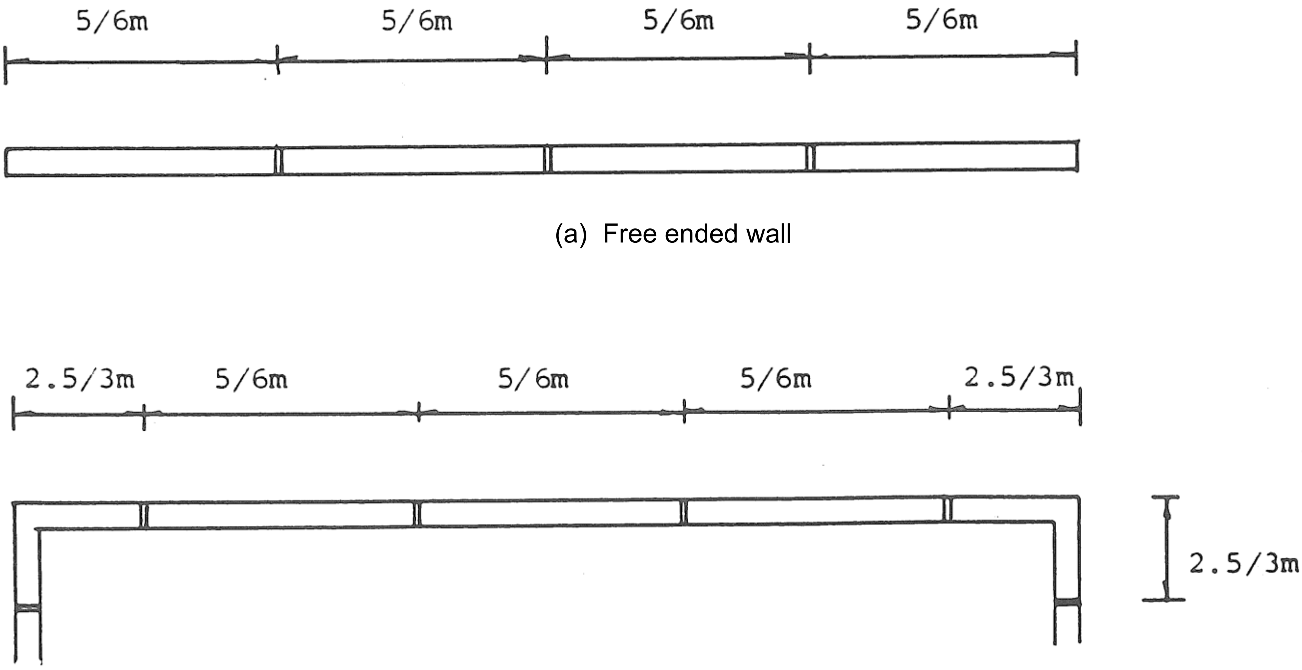


Figure 5: Veneer support, first floor level (Figure 73E, DBH E2 AS1 document)



Note: Windows/doorways will influence final exact position of joint.

Figure 6: Overall Positions of Shrinkage Control Joints

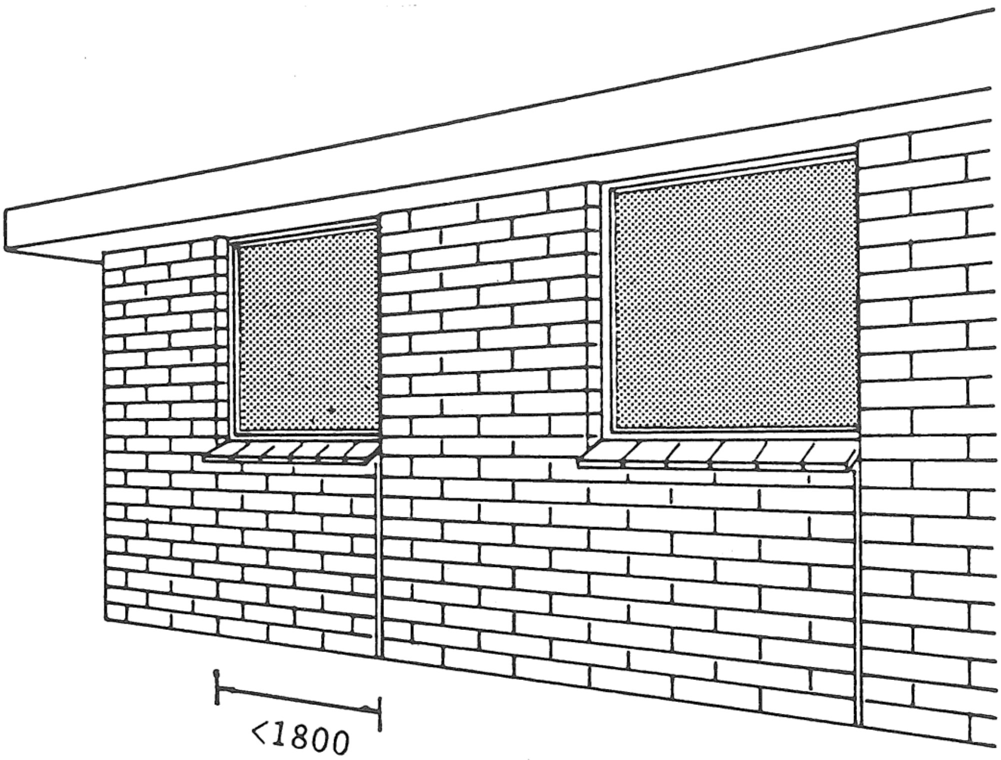
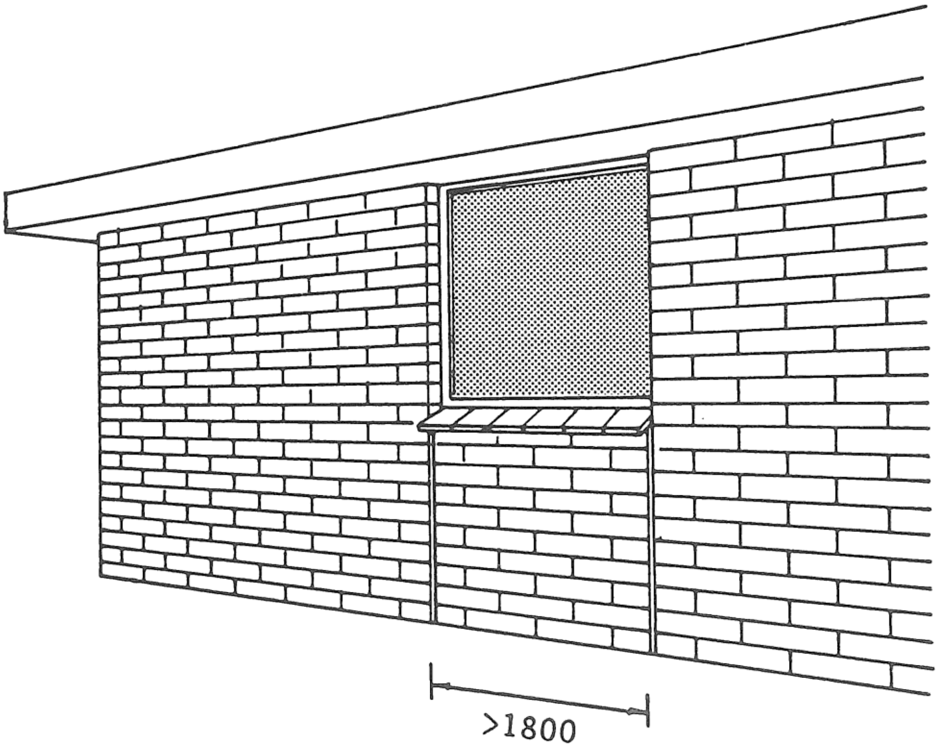


Figure 7



This panel can often be recessed or projected for effort.

Figure 8

A control joint between two closely situated windows should be avoided (Figure 9) because this would create two relatively narrow panels that would not allow the joint to function properly. Zigzag cracking would be likely to occur at the sills of the openings.

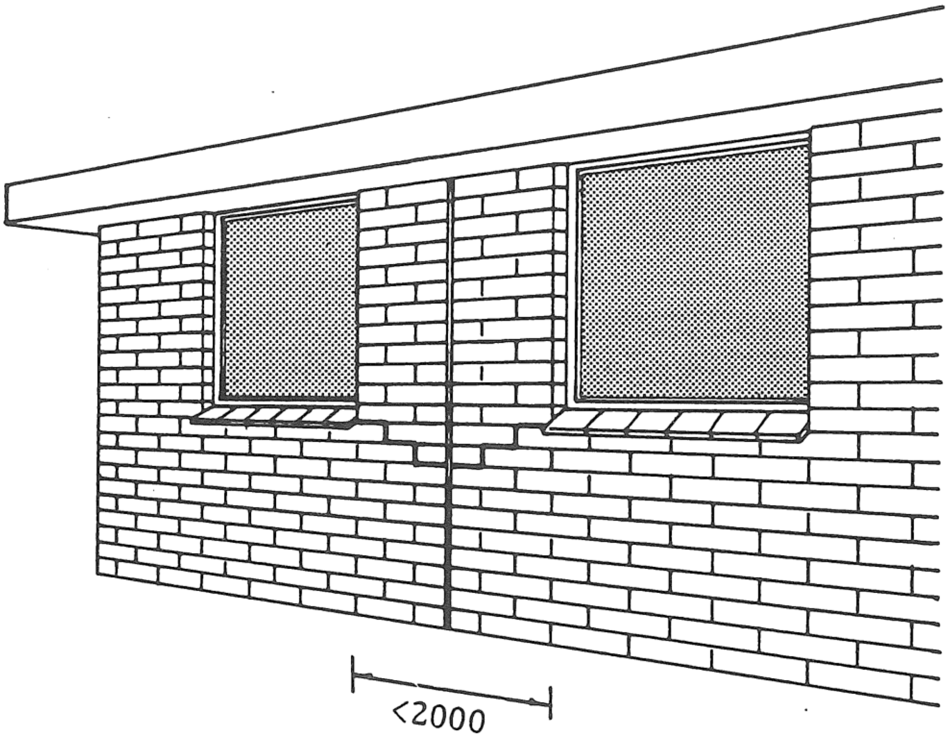


Figure 9

An alternative to control joints under windows is joint reinforcement as described in the next paragraph.

It should be noted that often jointing above door or window openings in veneers does not occur since in modern practice these openings are often carried full height to the frieze board, soffit or slab above.

The function of joint reinforcement (i.e. metal bonding mesh) is not to eliminate cracking in masonry veneers but to reduce it and prevent the formation of conspicuous shrinkage cracks.

Joint reinforcement becomes effective when shrinkage stresses commence. At this time the stresses are transferred to, and redistributed by the steel.

The result is evenly distributed to produce very fine cracks hardly visible to the naked eye.

Joint reinforcement should be located as follows:

In the first and second bed joints immediately above and below wall openings. Alternatively it may be used above the window with a conventional control joint below. It is difficult to align a vertical control above a window due to the influence of for example a steel lintel angle that is required to bear 200 mm beyond the jamb.

The reinforcement should extend not less than 600 mm past both sides of the opening. Figure 10.

This reinforced joint method is not a usual technique.

Note that this type of reinforcement under window openings is to deal with local stresses and therefore should not be regarded as replacing conventional control joints in any adjacent long length wall panels.

Where control joints are used they should be built from full and half-length units to maintain the bonding pattern.

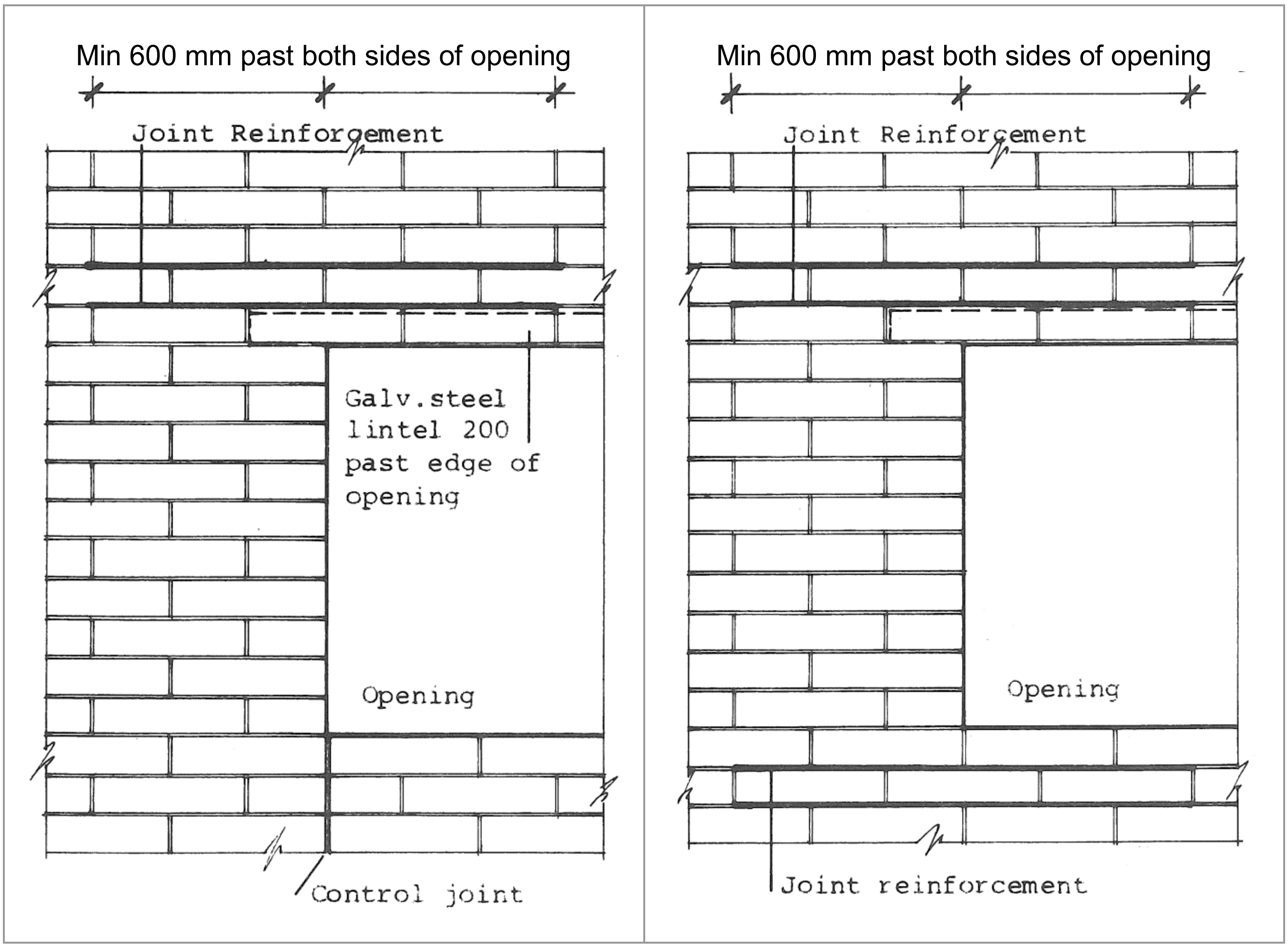


Figure 10: Control Joints and In Joint Reinforcement

An expansion control joint should be packed with a bitumen impregnated compressible filler strip and caulked with a sealant on the outside of the wall (Figure 11). A shrinkage control joint may be formed by raking onto the vertical joint by 20/25 mm. A sealant would be optional since any moisture penetrating the joint would be collected by the cavity construction. In areas of high wind/rain exposure a sealant would be advisable to reduce water transfer to the cavity (Figure 11).

NOTE:

Control joint details may vary according to individual manufacturer's recommendations.

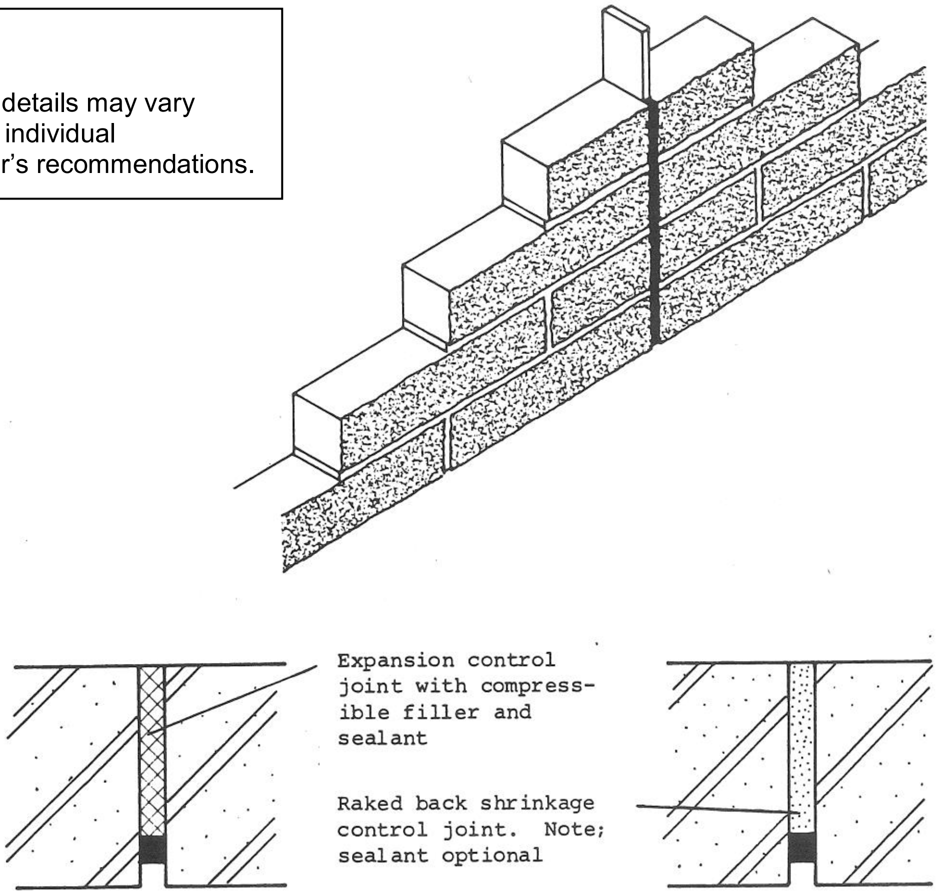


Figure 11: Control Joint Detail

Seismic Separation Joints

Veneers are a non structural element and as such are expendable under severe earthquake action. Examination of seismic damage to veneers however points to consistent problems associated with corners of buildings.

When a house has been built with masonry internal bracing, the movement at the top of a storey height is slight, approximately 1 mm. By contrast a house built using timber frame bracing has been shown to move up to 40 mm at the top of the storey.

Essentially the stiff veneer panel cannot accommodate such 40 mm movements and severe cracking occurs. The problem is particularly related to corners where whole bonded wall quoins become displaced.

To reduce veneer wall damage at corners it is preferable to form a non-bonded corner allowing for some differential movement.

The design for providing 40 mm of movement involves special closure flashings (see Figure 12) which will be distorted during an earthquake.

The replacement of the flashings would probably involve significant demolition and rebuilding. However, the risk of losing substantial corner sections of falling veneer would be reduced.

There are other ways of reducing the 40 mm maximum requirement by stiffening the timber frame for example by incorporating reinforced masonry panels. In this case a simpler corner detail which is just an extension of the control joint philosophy could be applied (see Figure 12).

The use of a backed sealant in respect of weather penetration is optional for veneer cavity wall construction. A suitable dual purpose product could be an expandable bitumen impregnated foam such as Compriband.

Essentially the use of a non-bonded corner is a move away from traditional construction but nevertheless is considered to have merit in dealing with movements arising from earthquakes.

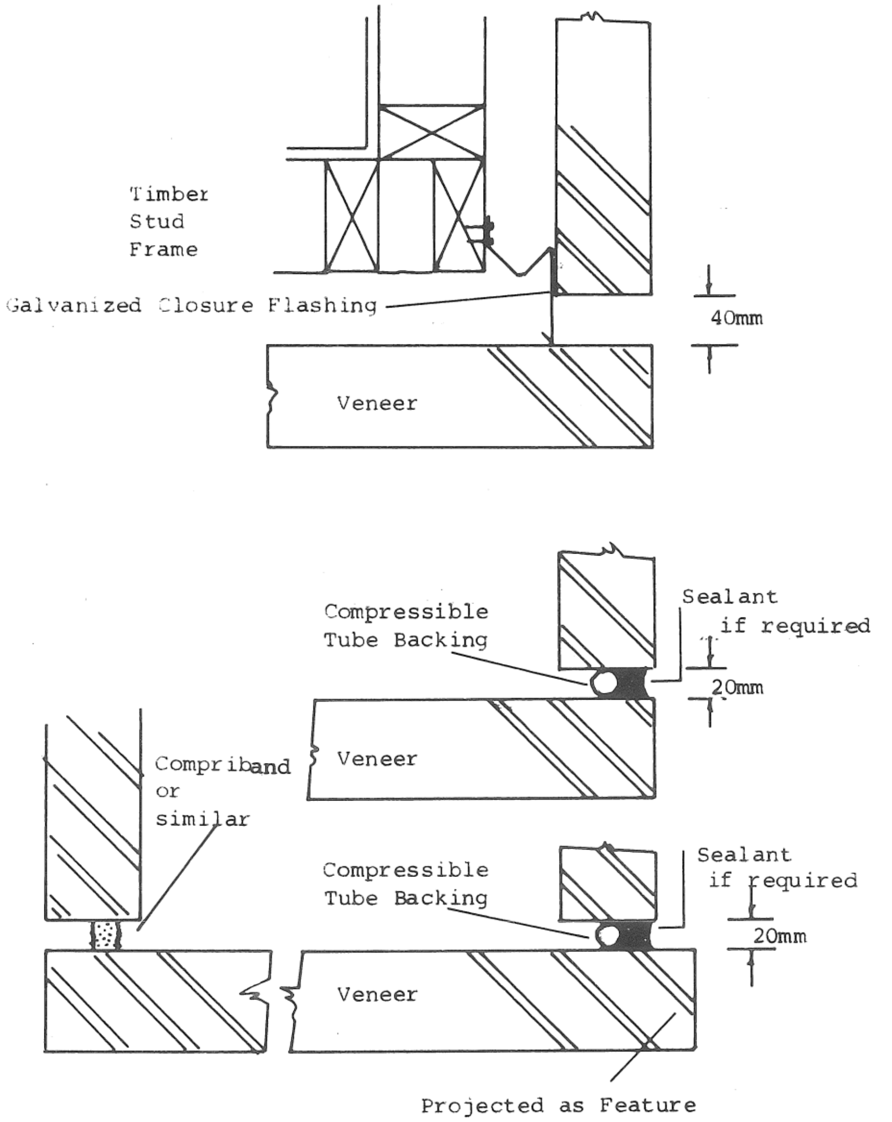


Figure 12: Seismic Control Joints

Construction Details

Veneer construction is illustrated in the Construction Section showing details at foundations F 4 and 7, intermediate floors I 6 and roof R 1.

Details shown in figures 13, 14, 15, 16 and 17 show joint, head, sill or threshold details for windows and doors.

Table 2 (page 13) is taken from E2 AS1/NZS 4229 giving the steel angle lintel spans permitted.

Lintels should bear at least 200 mm onto the supporting jambs.

Lintel steel shall comply with the durability aspects of Table 3 (page 13).

Table 2: Table E5 – Veneer lintel table – Steel angles (see E6.1)

Maximum lintel span (mm)	**Thickness of veneer** (mm)					
	70 mm			**90 mm**		
	Maximum height of veneer supported (mm)			**Maximum height of veneer supported** (mm)		
	350	**700**	**2000**	**350**	**700**	**2000**
2000	60 x 60 x 6	60 x 60 x 6	60 x 60 x 6	60 x 80 x 6	60 x 80 x 6	80 x 80 x 6
2500	60 x 60 x 6	80 x 80 x 6	80 x 80 x 6	80 x 80 x 6	80 x 80 x 6	80 x 80 x 8
3000	60 x 60 x 6	80 x 80 x 6	125 x 75 x 6	80 x 80 x 6	80 x 80 x 8	90 x 90 x 10
3500	80 x 80 x 6	80 x 80 x 6	125 x 75 x 6	80 x 80 x 8	90 x 90 x 10	125 x 75 x 10
4000	80 x 80 x 8	125 x 75 x 6	125 x 75 x 10	80 x 80 x 10	125 x 75 x 6	150 x 90 x 10
4500	125 x 75 x 6	125 x 75 x 10	–	125 x 75 x 6	125 x 75 x 10	–
4800	125 x 75 x 6	125 x 75 x 10	–	125 x 75 x 6	125 x 75 x 10	–

NOTE:

(1) All sections are steel angles.

(2) Stainless steel sections of equivalent section modulus are a permitted alternative.

Table 3: Table E2 – Protection for masonry veneer lintels supporting masonry veneer using AS/NZS 2699.2 (see E6.1)

Location (NZS 3604 Exposure Zones)	**Grades 316, 316L or 304 stainless steel or 600 g/m^2 galvanising on mild steel plus duplex coating**	**600 g/m^2 galvanising on mild steel or 300 g/m^2 galvanising on mild steel plus duplex coating**
Zone B	Yes	Yes
Zone C	Yes	Yes
Zone D	Yes	No

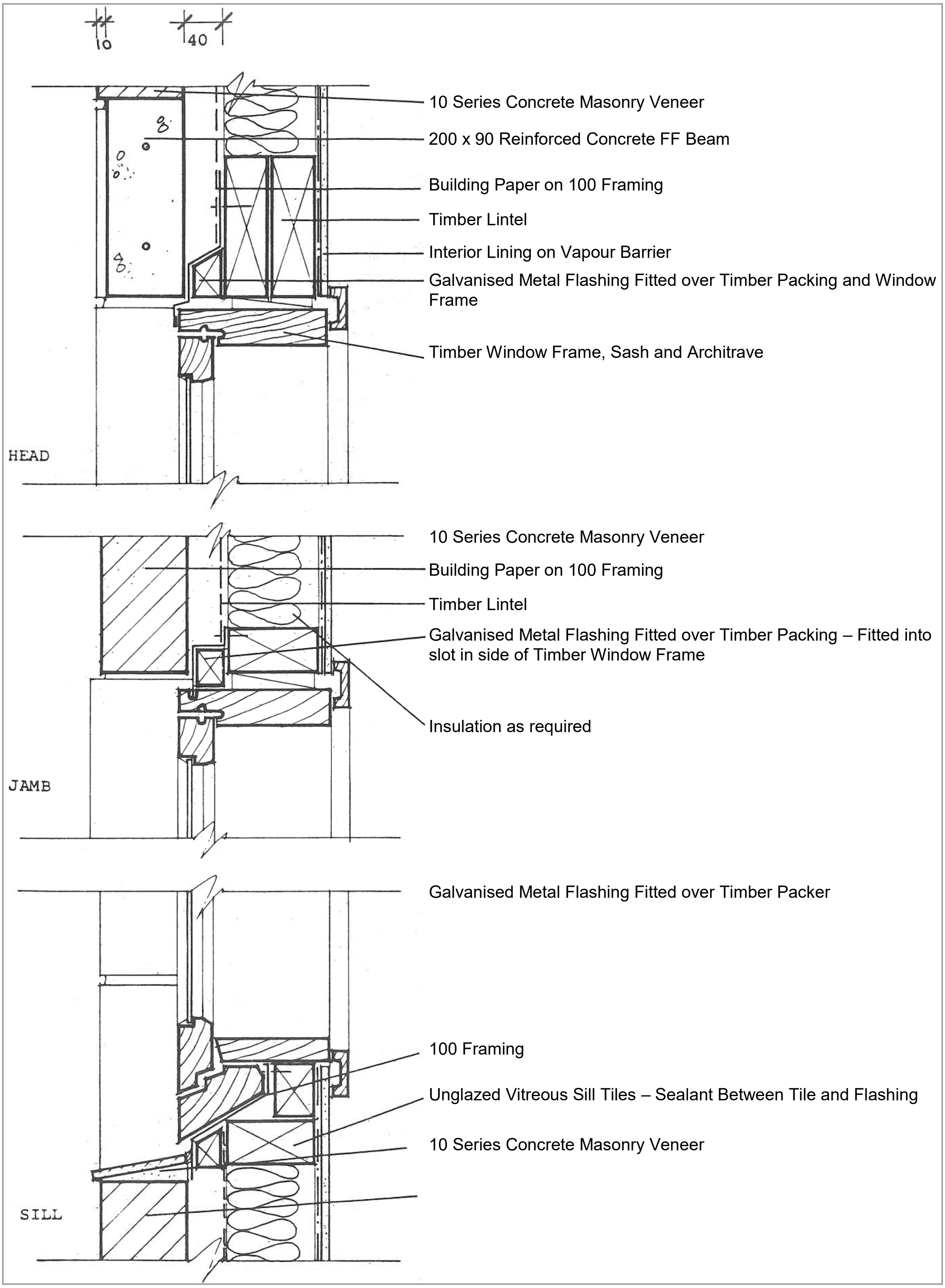


Figure 13: Veneer Details – Timber Windows

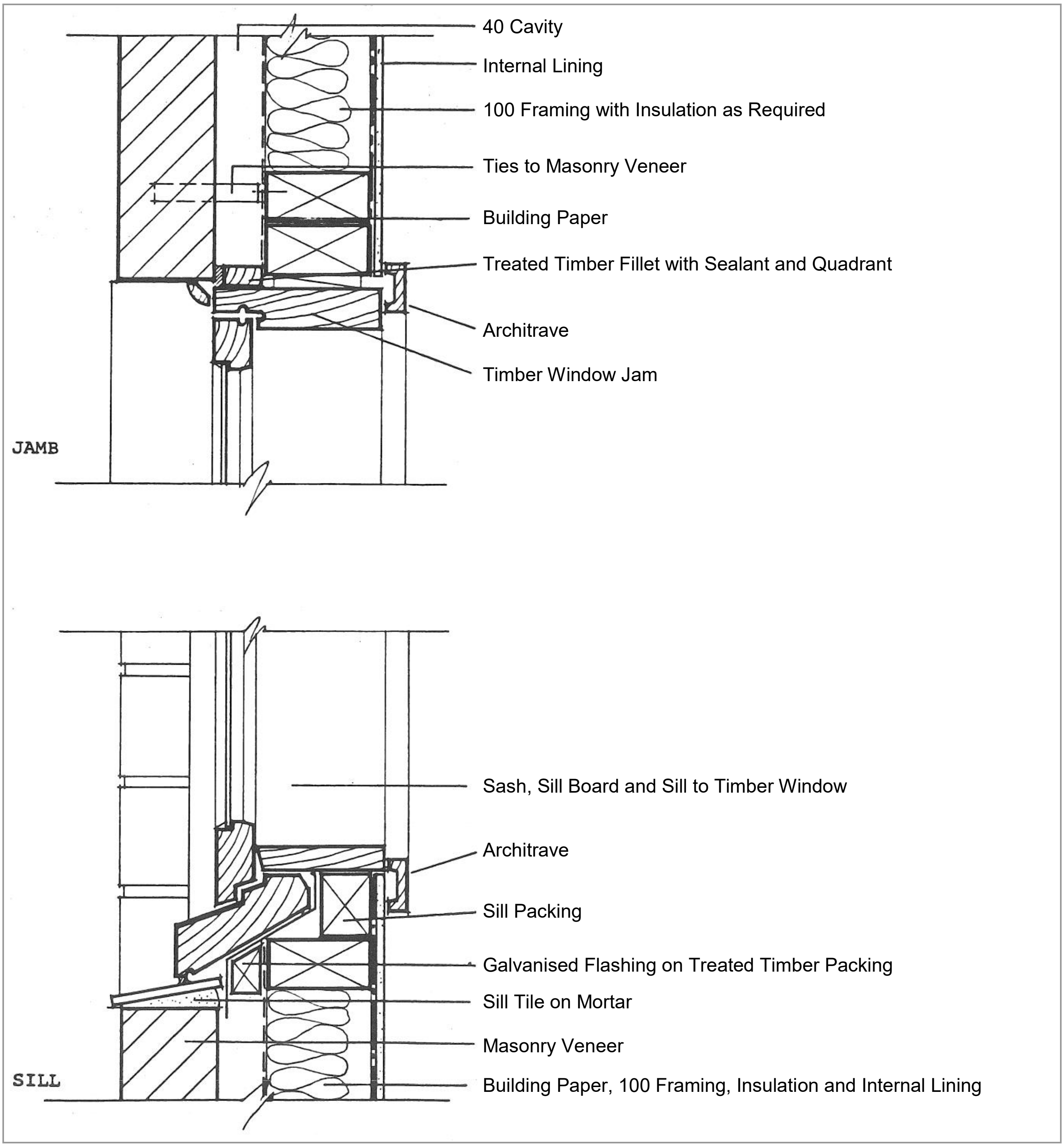


Figure 14: Jamb and Sill Details – Traditional Timber Windows

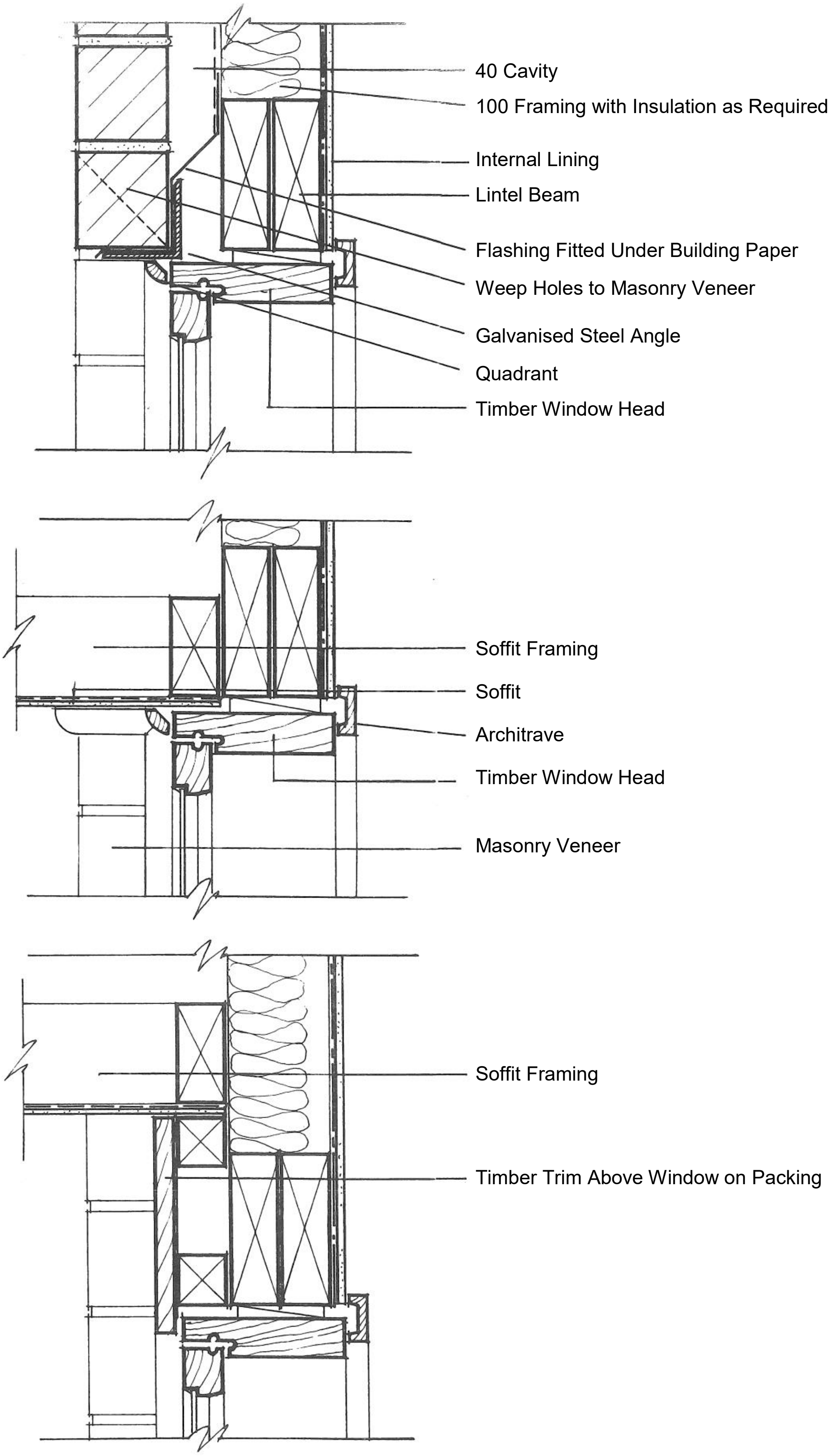


Figure 15: Timber Window Head Details

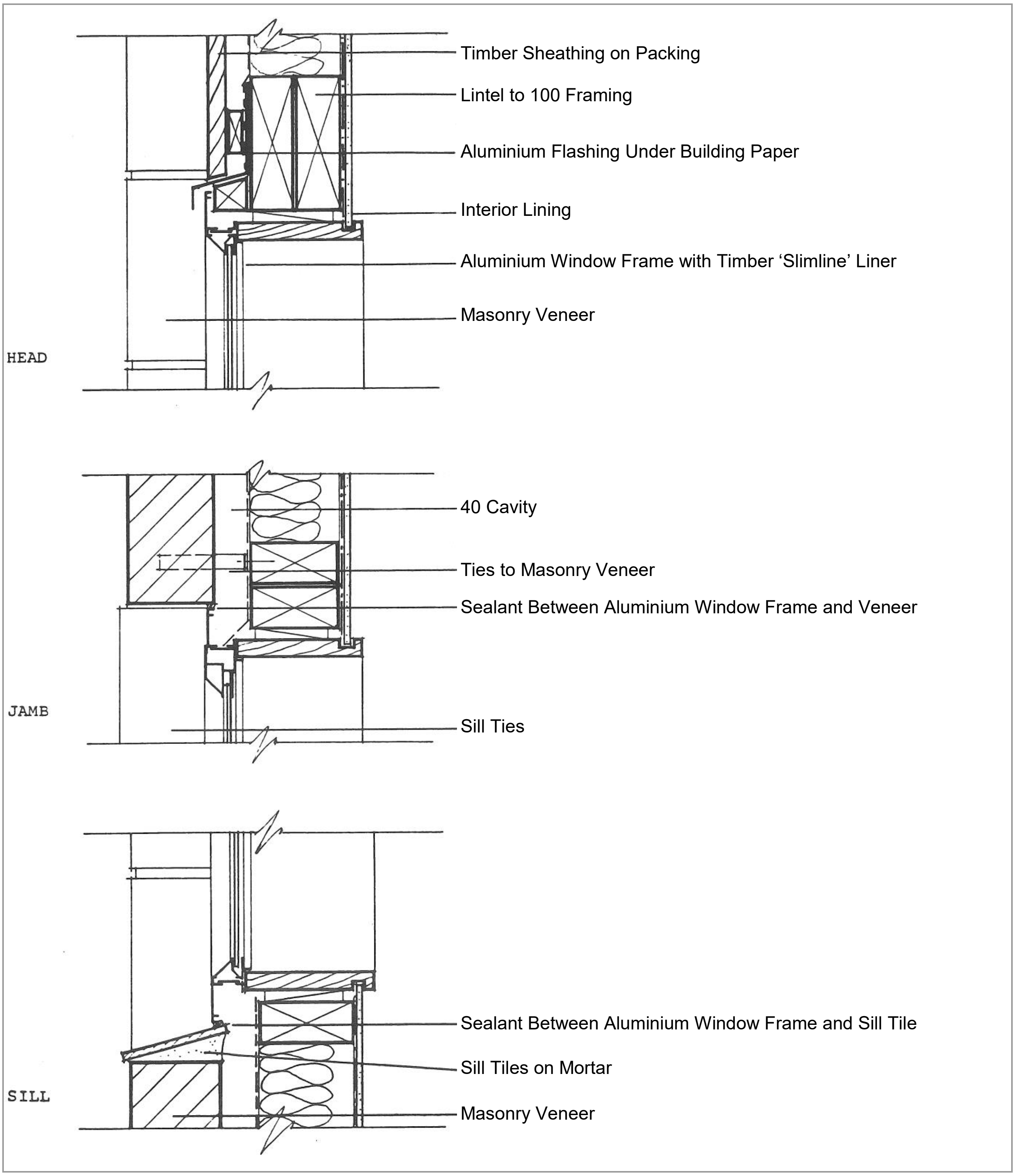


Figure 16: Aluminium Window Details

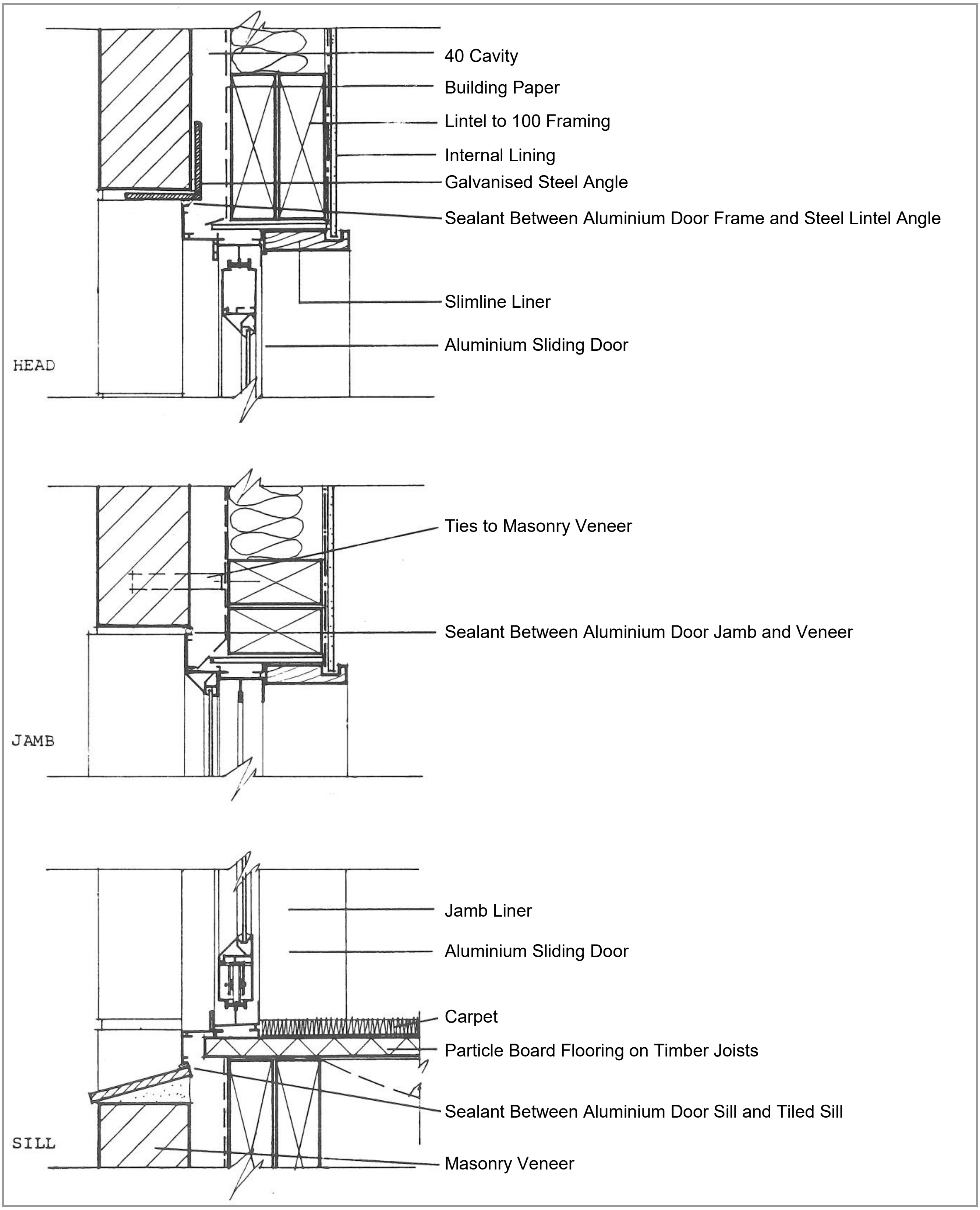


Figure 17: Aluminium Sliding Door Details

5.2 Concrete Bricks and Wall Ties

Bond Tests

Information on the bond strength characteristics of concrete bricks was required by the Standards New Zealand Committee revising NZS 4210. In addition, the matter of pull values for ties particularly with regard to the reduced embedment lengths for 70 mm veneer needed to be investigated.

The bond strengths and pull-out strengths achieved are seen as compatible with both standard veneer and reduced thickness veneer construction. The tests were carried out by the New Zealand Concrete Research Association in accordance with NZS 4210 requirements for bond strength test. The pull-out test was developed to measure the outline performance characteristics listed in NZS 4210.

The pull-out test method used has now been superceded by the requirements of AS/NZS 2699. For more recent tests on pull-out tests undertaken to AS/NZS 2699 reference should be made to BRANZ publications:

(i) Critical properties of mortar for good seismic performance of brick veneer by S. J. Thurston. BRANZ Study Report, SR 258 (2011)

(ii) Investigation of the Strength and Stiffness of Dry-bedded Ties in Various Veneer Types by G. J. Beattie. BRANZ Study Report No. 152 (2006).

However, the basic premise of the comparison between the two different mortar strengths remains valid.

Bond Tests, Concrete Bricks: Summary of Findings

Brick Type	Laying Condition	Mortar Type	Recorded Bond Strength	Mean of 5 Bond (kPa)
Solid	Dry	Rich[1]	761-1078	900
	Dry	Lean[2]	411-789	562
	Pre-moist	Rich	557-737	642
	Pre-moist	Lean	297-529	404
Holed	Dry	Rich	537-1029	776
	Dry	Lean	430-757	573
	Pre-moist	Rich	672-1020	802
	Pre-moist	Lean	453-659	558

Note:
[1] 1:3 by volume
[2] 1:5 by volume

The simple bending stress within a one way spanning panel based on 600 mm x 400 mm is 82 kPa for 87 mm brick and 102 kPa for 70 mm brick.

Tie Pull-out Tests, Concrete Bricks: Summary of Findings

(All units laid dry with 25 mm x 1.5 mm standard galvanized crimped steel tie).

Brick Type	Mortar Type	Tie Embedded (mm)	Pull-out Loads (kN)	Mean of 5 Pull-out (kN)
Solid	Rich	60	5.2 - 6.4	5.7
		45	4.1 - 5.3	4.8
	Lean	60	3.2 - 7.1	5.4
		45	3.3 - 5.3	4.5
Holed	Rich	60	8.5 - 9.1	8.8
		45	5.0 - 7.0	6.1
	Lean	60	4.1 - 6.7	5.4
		45	4.0 - 5.4	4.7

The horizontal design load for a tie as per NZS 4210 is to be taken as twice the area of dead load held by the tie. For 87 mm wide concrete brick with tie spacing at the maximum permitted (600 mm x 400 mm) the design load is 0.82 kN and for 70 mm wide concrete brick 0.66 kN.

NZS 4229/E2AS1/E2AS3

There has been a recent (2012) review of the spacing/specific requirements for ties due to changes in the requirements of the loading code AS/NZS 1102. The latest requirements can be found in **Tables 1 and 2** (next page). This information was provided by the Department of Building and Housing

The durability requirements can be summarized as follows, although reference to NZS 3604, to determine the actual zone boundaries, is required. Broadly, the sea spray Zone R4 extends from the high tide water mark up to 500 metres inland. Ties in this zone are required to be stainless steel 316 or 316L grade. Elsewhere ties need to have 470 g/m^2 galvanised coating or they could be stainless steel to 304 grade.

See more durability details in Section 5.1.

Special requirements are needed for geothermal areas.

Table 1: Masonry Veneer Area/Tie

Earthquake Zone	Area of masonry veneer attached to each Type B veneer tie of the duty specified. (m^2)					
	Veneer less than 180 kg/m^2			Veneer 180 kg/m^2 to 220 kg/m^2		
	EL	EM	EH	EL	EM	EH
1	0.24	0.24	0.24	0.20	0.24	0.24
2	0.16	0.24	0.24	0.13	0.20	0.24
3	0.11	0.16	0.24	0.09	0.13	0.24
4	0.08(6)	0.12	0.23	0.07(6)	0.11	0.22

(1) The horizontal tie spacing multiplied by the vertical tie spacing selected must equal or be less than the area of masonry veneer given for the earthquake zone and the veneer mass. The maximum spacing of ties is 600 mm horizontal and 400 mm vertical.
(2) Type B and prefix E indicate ties are manufactured to meet the testing conditions set out in AS/NZS 2699.1.
(3) L (light), M (medium) and H (high) indicate strength capabilities of ties to meet the testing conditions set out in AS/NZS 2699.1.
(4) Using high strength ties does not permit the maximum spacing of ties to be increased.
(5) Ties may be face fixed to blockwork or fully embedded in the structural masonry wall joint.
(6) Some small veneer areas may be impracticable.
(7) Minimum strength for fixings to blockwork are 0.5 kN (EL), 0.75 kN (EM) and 1.5 kN (EH).

Table 2: Tie Duty Schedule

Earthquake Zone	Masonry veneer attached by Type B veneer ties of the duty and spacings specified	
	Veneer less than 180 kg/m^2	Veneer 180 kg/m^2 to 220 kg/m^2
	Spacing 600 mm x 400 mm	Spacing 500 mm x 400 mm
1	EL	EL
2	EM	EM
3	EH(6)	EH(6)
4	SED(6)	EH

(1) Maximum spacing of ties with veneer less than 180 kg/m^2 and for SED is 600 mm by 400 mm. SED means actual spacing of ties is to be determined by specific engineering design.
(2) Maximum spacing of ties with veneer between 180 kg/m^2 and 220 kg/m^2 is 500 mm by 40 mm.
(3) Type B and prefix E indicate ties are manufactured to meet the testing conditions set out in AS/NZS 2699.1.
(4) L (light), M (medium) and H (high) indicate strength capabilities of ties to meet the testing conditions set out in AS/NZS 2699.1.
(5) Using high strength ties does not permit the maximum spacing of ties to be increased.
(6) Ties may be face fixed to blockwork or fully embedded in the structural masonry wall joint.
(7) Minimum strength for fixings to blockwork are 0.5 kN (EL), 0.75 kN (EM) and 1.5 kN (EH).

5.3 Specification for Concrete Masonry Bricks

Specification

The specification for the manufacture of concrete bricks follows the provisions of AS/NZS 4455 Part 1 *Masonry Units* and NZS 4210 with testing carried out in accordance with NZS 4456.

The specific details are already included in Section 1.7 because the requirements for concrete masonry units are similar whether the masonry is hollow block or solid brick, except for strength where blocks are 12.5 MPa and solid brick 10 MPa.

The 10 MPa requirement comes from Clause 2.1.4.5 of NZS 4210 and relates to durability, particularly in respect of frost effects. In practical terms, the brick strength as tested to AS/NZS 4456.4 should be recording approximately 14 MPa to allow for the height/width correction factor K_a of the Standard.

The manufacturing size tolerances defined in Section 1.7 do not apply to the length or width of units having a split, profiled, rumbled or textured face.

Other Comments

(i) No frog or perforation shall be within 25 mm of the faces of the unit.

(ii) No frog shall be more than 25 mm deep.

Where the client wishes to check the risk of efflorescence, then test AS/NZS 4456 Part 6 should be carried out.

5.5 Veneer – Stack Bond

Introduction

This section has been prepared to provide designers, territorial authorities and builders with some standard design details for block masonry veneer using stack bond laying format.

The scope of this section is limited to the masonry modules described in the Materials section below.

Standards

The veneer and supporting construction referred to in this section is to follow the requirements of New Zealand Building Code Section E2/AS1. The following standards are referenced in that section:

- NZS 3604 Timber Framed Buildings
- NZS 4210 Masonry Construction: Materials and Workmanship

Additionally, the provisions of this section may be applied when used in conjunction with construction to NZS 4229, Concrete Masonry Buildings Not Requiring Specific Design.

If the scope of the proposed work is outside the limitations and requirements of the above then specific engineering design advice must be sought.

Materials

Masonry veneer shall comply with the following requirements:

- 90 mm maximum thickness, 70 mm minimum thickness.
- Length and height of units to be 390 mm and 190mm respectively
- May be either hollow or solid. Hollow blocks to be 10.01, Standard Whole.
- Shall be manufactured to the requirements of AS/NZS 4455.

Lattice mesh shall be Eagle Wire Bricklock, or equivalent, and comply with the following requirements:

- Comprise two parallel longitudinal steel rods of minimum 4 mm diameter, held apart approximately 55 mm on centre by welded cross wires of 2 mm diameter, at 200 mm centres.
- Steel shall have a minimum yield strength of 300 MPa and be hot dip galvanised to minimum 470 gm/m^2. Coat any cut ends of both main, and cross wires, with zinc rich epoxy primer.
- Mesh to be supplied and installed in minimum 2 m length modules, contact-lapped to the detail shown in Figure 1(a) following, or alternatively by offset-course lapping to the detail shown on Figure 1(b).

Mortar shall be to the requirements of NZS 4210.

Veneer Construction

In addition to the requirements given in the Standards section above the stack bond veneer shall be constructed to the following requirements:

1. Studs in timber framed walls to be at 400 mm centres.
2. Wall ties, to the requirements of NZS 4210, are to be provided at 400 mm centres both vertically and horizontally.
3. Lattice mesh shall be laid continuously in horizontal courses at 800 mm maximum vertical centres, commencing no higher than the second course above the veneer base.
4. Lattice mesh shall also be laid in the course or courses directly above and below openings, extending a minimum 600 mm past the edge of the opening.
5. Lap joins in lattice mesh shall be made at mid-length of 390 mm block units and shall be staggered so that adjacent laps do not occur within the same vertical block stack.
6. Lattice mesh may be discontinued only at control joints.
7. Use purpose made 'L' formed lattice mesh at corner intersections.

Figures 1(a) and 1(b) on pages 3-4, show typical details for stack bond veneer.

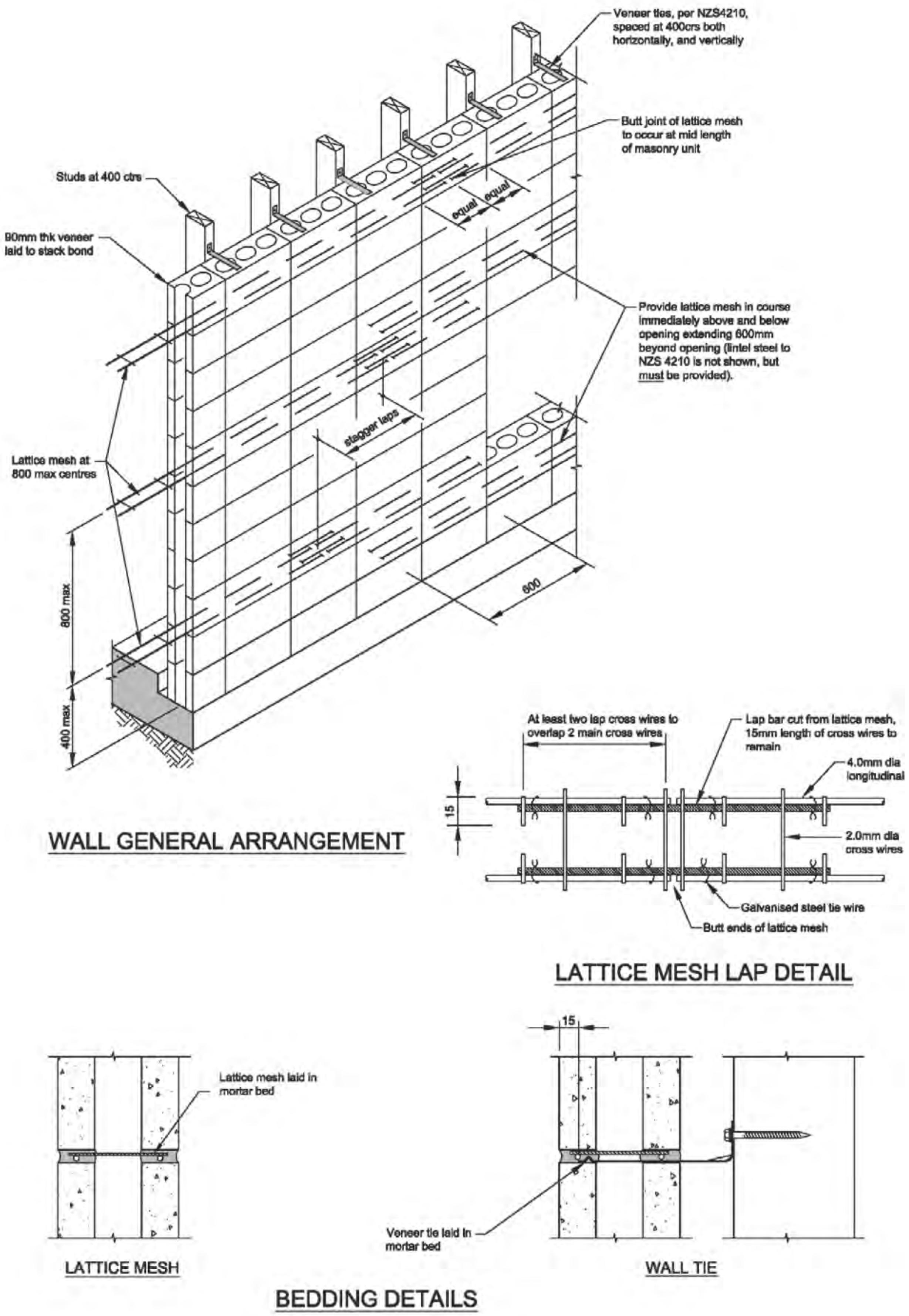
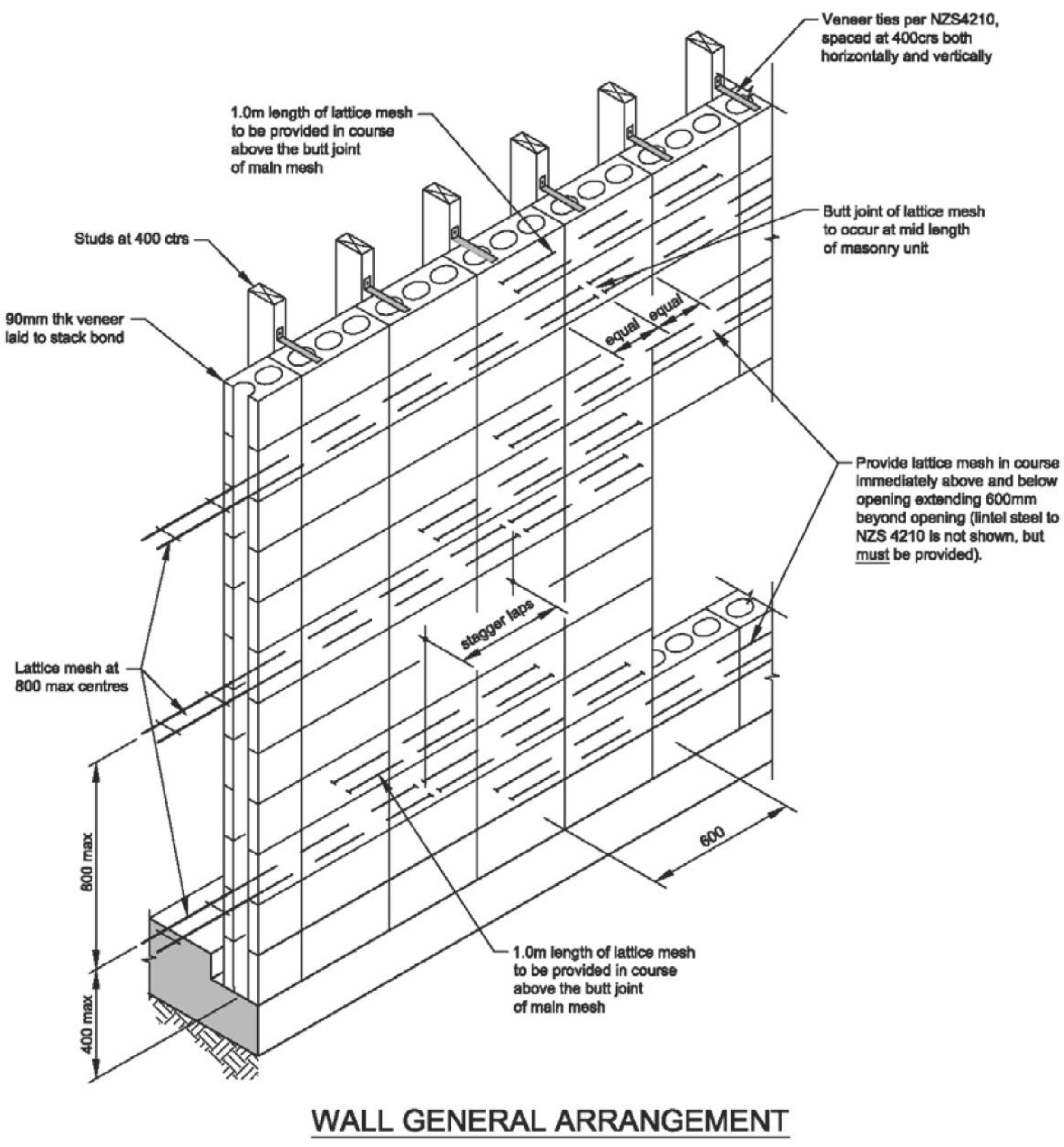


Figure 1(a): Stack Bond Veneer with Contact Laps



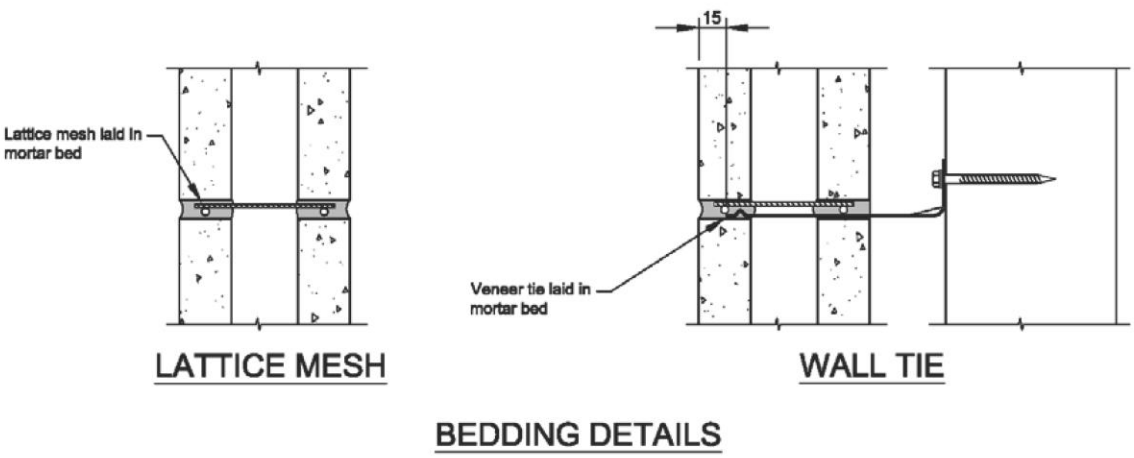


Figure 1(b): Stack Bond Veneer with Offset-Course Laps (alternative to Contact Laps)

6.1 Masonry Retaining Walls

Introduction

This section has been prepared to provide designers and builders with some standard design details for reinforced concrete masonry retaining walls. It has been updated from the previous version (2019) with the following changes:

- The foundation designs incorporate updates to the reinforced concrete design standard NZS 3101 that now requiring greater reinforcing content.
- Some wall designs that required larger wall reinforcing contents have been removed.
- The tables for walls carrying surcharge have been updated and 140 mm wall options have been removed from this category.

It is emphasized that the contents of section 6.1 are intended to provide guidance only. Professional engineering, and possibly geotechnical engineering, advice must be sought in relation to final design and submission for any building consent application being sought.

It is also important to note that the standard design details in this section ***do not*** address the following matters:

- Retaining walls built integrally with, or directly or indirectly supporting, building structures.
- Retaining walls that support sealed carriageways such as public roads that will be subject to heavy vehicle loads.
- Global stability of the land being constructed on.
- Retaining walls built near cliffs, ridge lines, or on moderate to steeply sloping ground.
- Retaining walls constructed in potentially liquefiable soil.
- Walls required to have an AS/NZS 1170.0 Importance Level of IL 3, and above (e.g. where they are part of post-disaster or emergency centres).
- Walls in geothermal hotspots defined in NZS 4230; 140 series walls in corrosion zone D defined in NZS 3604.

Further detail and commentary on assumptions made in the structural design of the walls are provided in later sections.

The principal advantages of reinforced concrete masonry walls over their reinforced concrete counterparts are the elimination of shuttering and the uniformity of the concrete surface texture.

Two types of wall more commonly encountered have been considered, as follows (refer Figure 1).

Type I might be used when excavation is below the level of a neighbouring property and is to be built as close as possible to the boundary.

Type II might be used when filling above and against a neighbouring boundary.

Design details and tables are provided for the two wall types, for three foundation soil conditions, and three earthquake zones, covering the following loading cases:

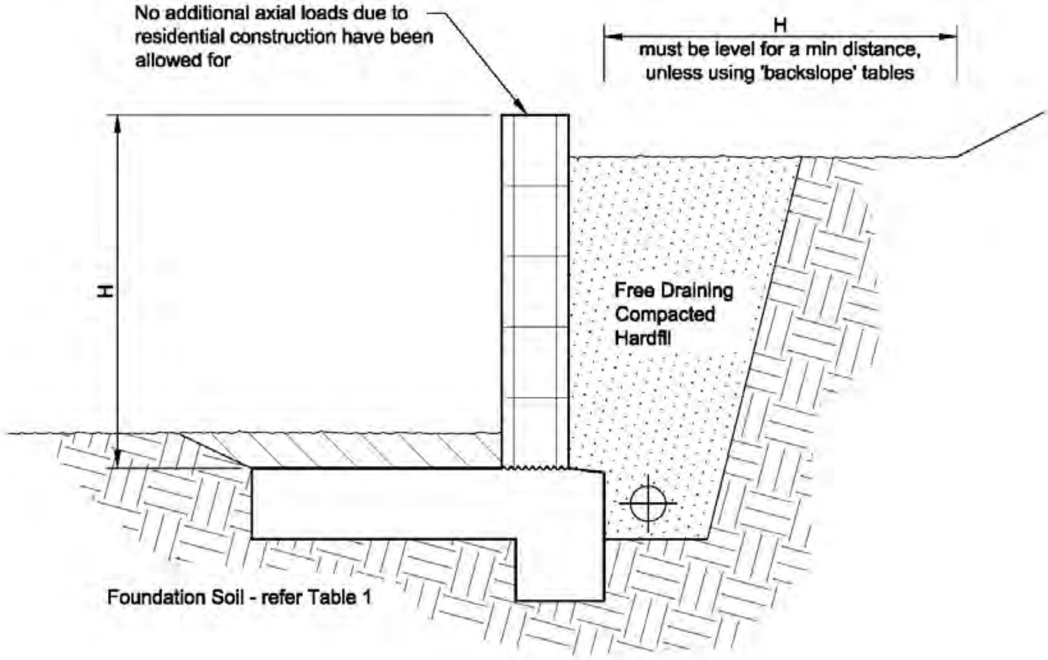
- Retained soil is level behind the wall, no surcharge.
- Retained soil is level behind the wall, but supports imposed surcharge loading.
- Retained soil behind the wall has a backslope.

Selection of Retaining Wall

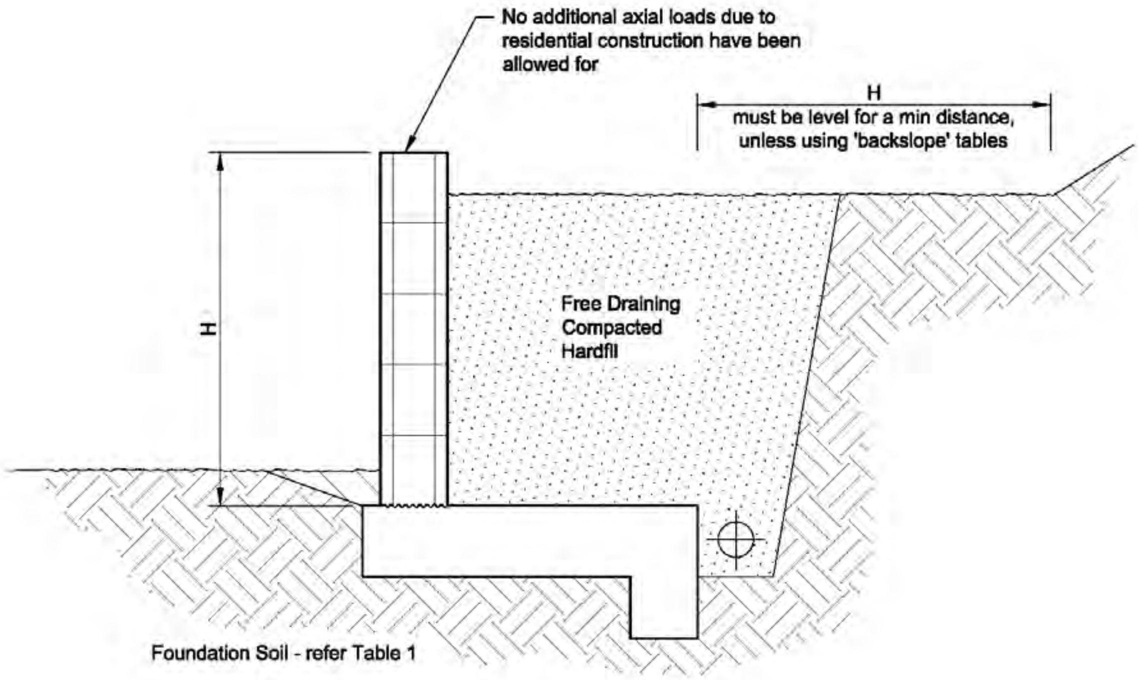
Boundary and Site Conditions

By reference to the particular site conditions the type of wall to be used can be selected as either Type I or II, refer to Figure 1 (page 2).

Note the minimum level distance requirement (except the load case provided where a specified maximum back slope angle exists behind the wall and there is no other concurrent form of wall surcharge).



TYPE I RETAINING WALL



TYPE II RETAINING WALL

Figure 1: Key to Wall Types/Loads Permitted

Foundation Soil Conditions

Foundation soil conditions on the site need to be assessed so that an appropriate foundation soil type can be selected from Table 1. The design charts provided have classed soils into three representative types for design simplicity. These are shown in Table 1.

If soil types for the site differ markedly from the the types listed then professional engineering advice must be sought.

Wall Backfill

Design soil loadings are based on assumed parameters for medium dense, free-draining, imported backfill, such as that meeting as follows (see Table 2).

Should a backfill material with a greater bulk density, γ, and/or a smaller friction angle, ϕ, be used then the assumptions used in these sample designs will not be valid.

Table 1: Soil Types Used in Design of Wall Foundations

Soil Type	Classes of Soil Included	Design Parameters		
		γ (kN/m^3)	ϕ (º)	C' (kPa)
A	Gravel and sand – medium dense	19	32	0
B	Silt and Sandy Silt – firm/medium dense	20	25	2
C	Clay and Silty Clay - firm	19	17	25

The design parameters for each of the soil classes are chosen to be representative, but will vary in practice.

Table 2: Wall Backfill

Soil Type	Classes of Soil Included	Design Parameters		
		γ (kN/m^3)	ϕ (º)	C' (kPa)
Backfill	Imported, free draining, hardfill	19	32	0

Loading Conditions

Design charts have been produced for:

(a) Level ground, no water pressure*, and no surcharge (e.g. garden wall within property boundary).

(b) Level ground, no water pressure*, and an imposed surcharge (taken as G=5.0 kPa permanent, or Q=12 kPa transient, as required as a default minimum by some territorial authorities when retaining below and against boundary lines, to allow for use as a private driveway).

(c) Nominal 20^0 maximum back slope angle of soil retained, no water pressure* or other surcharge present.

**An allowance for pore water pressure in clay soils only is included in the designs.*

The appropriate loading conditions (a), (b), or (c) must be selected. Earthquake effects are considered for all walls, but do not necessarily govern the design.

It is **vital** to ensure that adequate drainage is provided behind the retaining wall.

Load Combinations

Ultimate Limit State (ULS) load combinations considered, in accordance with the provisions of AS/NZS 1170.0 and Module 6, are as follows:

- For loads that produce net stabilising effects (Ed,stb)
  - Ed,stb = [0.9G)
- For loads that produce net destabilising effects (Ed,dst):
  - Ed,dst = [1.2G + 1.5FE + 0.4Q] gravity case
  - Ed,dst = [G + Eu+ 0.3Q] earthquake case

in which:

- $E_{d,stb}$ = design action effect, stabilising
- $E_{d,dst}$ = design action effect, destabilising
- FE = static earth pressure, including surcharge where applicable
- Eu = ultimate earthquake action (pseudo-static earth pressure and wall inertia)
- G = self-weight (dead load)
- Q = imposed action (live load)

Serviceability Limit State (SLS) conditions have not been checked as for the most part there are no governing criteria for the wall types considered in this section. Where the wall supports ground adjacent to either a building foundation or a sealed vehicle carriageway that may be subject to heavy vehicle loads, SLS effects need to be assessed specifically by the design engineer.

Earthquake Loading

The contents of the MBIE publication *Earthquake Geotechnical Engineering Practice, Module 6: Earthquake Resistant Retaining Wall Design* provide detailed guidance on the parameters and processes used for the design of retaining walls for both gravity and earthquake conditions. These criteria have been incorporated into the retaining wall designs. A number of assumptions have been made in key areas and these are summarised as follows:

1. Blockwork cantilever walls are structurally flexible and can sustain permanent deformation following a design seismic event of at least 0.4% of their height (e.g. 10 mm for a 2.5 m high wall).
2. The structure importance level to AS/NZS 1170.0 is no greater than IL2 ('normal' buildings), giving a ULS design seismic event return period of 500 years, appropriate for a 50 year design working life for the structure.
3. The retaining wall is not constructed near a cliff, or on a ridged hillside greater than 30 m high and having an average slope exceeding 15^0, such that the Module 6 earthquake action topographic magnification factor, A_{topo}=1.0, is valid.
4. The wall is not attached to, or does not directly support, a building structure and has a Module 6 wall displacement factor no greater than W_d=0.5.
5. The NZS 1170.5 Earthquake Action standard site soil category is Class C, shallow soil, giving the strongest design peak ground acceleration for any given earthquake zone.
6. During earthquake loading in higher seismic zones backslopes greater than 15^0 may spill surface soil back to that angle. Where this is considered to be unacceptable, limit backslopes to 15^0.
7. Earthquake zones follow the designations provided in the light timber framed building standard NZS 3604, as reproduced in Figure 2 herein. Only two options are considered, being Zone 1 and grouped Zones 2 and 3 (Zone 3 governs). Wall designs for the sparsely-populated Zone 4 are excluded. For reference, the NZS 1170.5 hazard factors, z, for the three hazard zones considered are given as follows:

NZS 3604 Earthquake Zone 1: z_{max} = 0.20

NZS 3604 Earthquake Zone 2: z_{max} = 0.30

NZS 3604 Earthquake Zone 3: z_{max} = 0.46

Figure 2 (page 5) is a map of the seismic zones in NZS 3604, modified to include the Christchurch earthquake region in accordance with Amendment 10 of New Zealand Building Code clause B1/AS1.

A more detailed depiction of the zone boundaries is provided in NZS 3604.

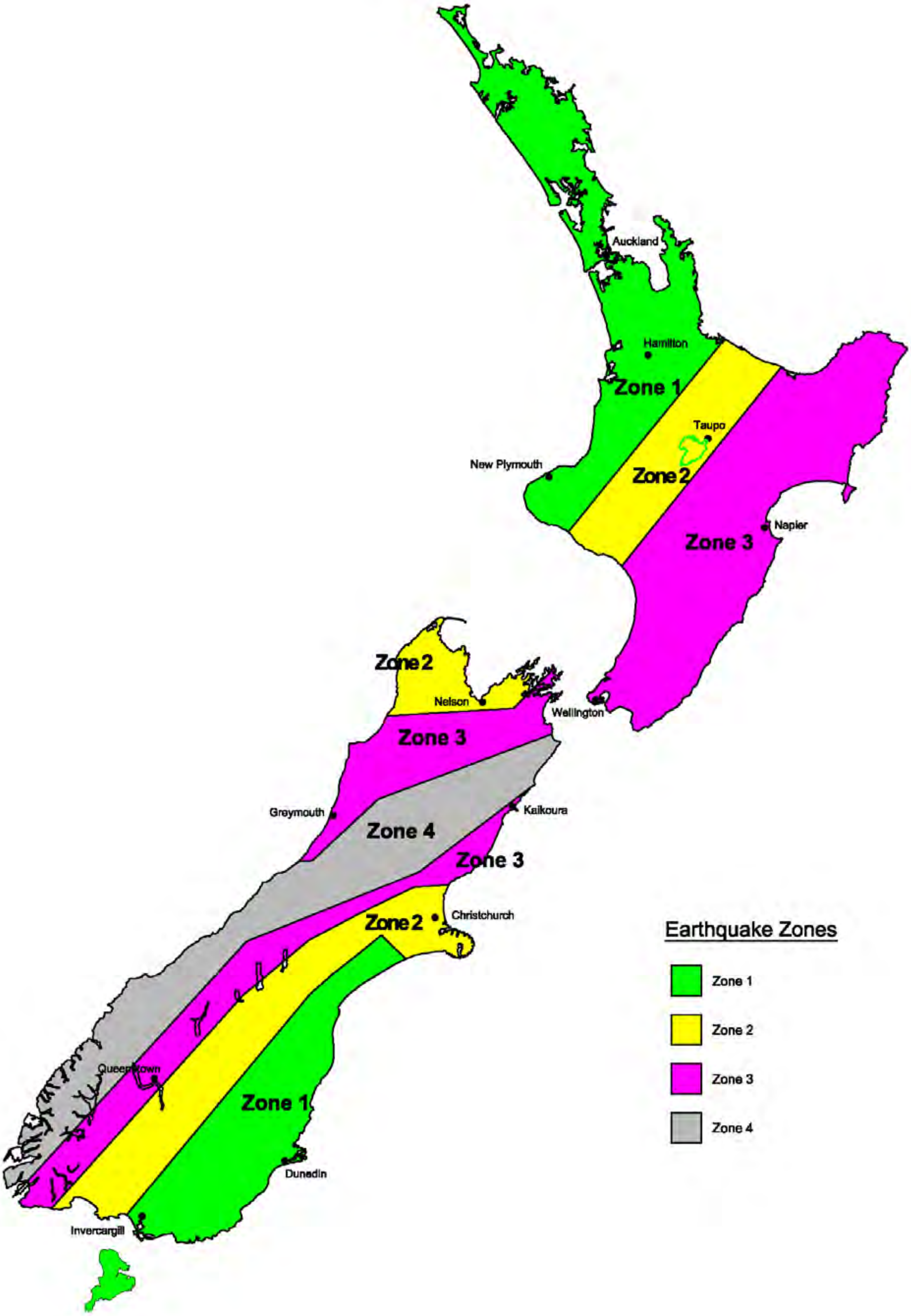


Figure 2: Earthquake Zones

Construction Methods and Requirements

Two standards of criteria on construction observation have been set down in the design tables, as follows:

1. Construction With Overview: Observation Type B

   Inspection by either a suitably qualified Engineer, Territorial Authority building inspector or Licenced Building Practitioner (Bricklaying and Blocklaying) is required during construction.

2. Construction Without Overview: Observation Type C.

   This category is intended where supervision is not provided. This grade may be used for retaining walls having a maximum retained soil height up to 1.5 m.

The use of the terms Observation Type B and Observation Type C arises from New Zealand Standards NZS 4230:2004 *Design of Masonry Structures* and NZS 4229:1998 *Concrete Buildings not Requiring Specific Design.*

A Licensed Building Practitioner, Bricklaying and Blocklaying, qualified in Structural Masonry, is acceptable to most territorial authorities as able to provide a producer statement for workmanship in accordance with NZS 4210.

Specification of Materials

Concrete for Footings

Concrete shall comply with NZS 3109:1997 for concrete having a minimum compressive strength of 25.0 MPa at 28 days.

Ready mixed concrete should be ordered having 20 mm maximum size aggregate, 25 MPa strength and with a maximum 100 mm slump.

Blockwork Infill Grout

Grout for infilling blockwork shall comply with NZS 4210 *Masonry Construction: Materials and Workmanship*, having minimum crushing strength of 20 MPa at 28 days and a spread between 450-530 mm when tested in accordance with the appropriate test requirements of NZS 3112:1986 *Specification for Methods of Test for Concrete".*

When the minimum dimension of the grout core is less than 60 mm, then a fine grout consisting of concreting sand and cement should be used, otherwise a coarse grout is required with maximum aggregate size of 12.5 mm or 19.0 mm.

Mortar for Laying Blocks

Mortar shall comply with NZS 4210 *Masonry Construction: Materials and Workmanship*, having a minimum compressive strength of 12.5 MPa when tested in accordance with Appendix 2.A of NZS 4210.

Reinforcing Steel

Reinforcing steel shall be Grade 500E to AS/NZS 4671:2019, Steel for the reinforcement of concrete.

Particular attention should be taken to the cover requirements for vertical reinforcing shown on the construction diagrams as it is critical to the design strength of the walls. Reinforcing steel extending from footing pours that does not meet dimensional requirements must ***not*** be bent to achieve a fit.

Masonry Construction

Wall construction shall follow the provisions of NZS 4210. Construction will predominantly use open ended, depressed web units, i.e. 1516, 2016, 2516; or where available H block configuration, e.g. H2016. All cells are to be filled with grout.

Vertical control joints are to be in accordance with the detail provided in NZS 4210 and located at:

- 8.0 m maximum spacing and/or
- at a change in direction of the retaining wall and/or
- at a change in height of the retaining wall and/or
- at a step in the foundation

Design Notes

Relevant criteria used for the designs are noted as follows:

- The retained soil at the top of the wall from the back of the footing heel is level for a distance equal to the height of wall (except for tables where a specified back slope angle exists). All soil contained from the back of the wall to a 45° line from the base of the rear of the footing must be sound graded rock filling with a minimum Φ = 32° and maximum γ_d = 19 kN/m^3.

- The walls are not designed for the forces due to compacting machinery working on the retained soil. Adequate precautions, e.g. shoring, strutting, etc. must be taken to ensure no damage occurs to the wall during this operation.

- The design considers stability of the wall for sliding, overturning and bearing on the soil immediately under and adjacent to the wall.

- Global stability of the soil mass at the site has not been considered.

- A drainage layer of suitable granular material is provided at the back of the wall, with a perforated pipe at the base discharging to the open. Surface water must also be prevented from accumulating at the top of the wall and overloading the drainage system.

- The assumed weights of materials are:

Reinforced Concrete:	24.0 kN/m^3
Reinforced Concrete Masonry:	22.0 kN/m^3
Backfill Aggregate:	19.0 kN/m^3

Blockwork and concrete are designed to the requirements of the New Zealand Building Code, Clause B1, Structure. Key requirements are as follows:

- The design is based on NZS 4230:2004 Code of Practice for the Design of Masonry Structures. 60 mm cover to reinforcing steel from soil side of the wall has been used for 140 mm and 190 mm walls, and 70 mm cover for 240 mm walls. Refer to Figure 3 for construction details showing block layout, reinforcing location, bar bend radii, etc.

- Seismic action effect options cover three of the four most-commonly applicable seismic zones defined by the Timber Framed Buildings Standard, NZS 3604:2011. Mononobe-Okabe equations have been used to assess seismic action effects, in accordance with Module 6.

- Calculations assume a minimum 100 mm cover of earth or paving materials on top of projecting footings. In front of the wall (non-retained side).

- Soil forces are calculated using the Coulomb active earth pressure theory assuming wall movements, lateral and rotational, are sufficient to allow active pressure to develop and that wall/soil friction can develop.

- For both cohesive and granular soil types, the soil and surcharge are assumed to act at an angle of Φ to the 'virtual back' of the wall. The existence of a heel for both wall types allows this assumption to remain valid.

- The length of the wall is a factor in assessing bearing capacity of the soil beneath the foundation and has been conservatively assumed at 20 m. Shorter length walls will typically have greater bearing capacity and may allow a small reduction in footing width than generated under these standard designs.

Use of Design Charts

- By reference to boundary and site conditions, the appropriate wall type can be selected, Type I or II (see Figure 1).

- Select the appropriate soil type (see Table 1).

- Determine if a surcharge for vehicle access and parking is required, or if the retained soil will have a back slope angle.

- Determine what Earthquake Zone is appropriate (see Figure 2).

- Reinforcement tables indicating the maximum height to be retained for the appropriate wall, soil types, and loading conditions, will determine whether a 140 mm, 190 mm, or a 240 mm wall should be used. The top row for the 190 mm series walls has been provided to give minimised footing dimensions for lower height wall options.

- Enter selected chart, using maximum height of soil retained, to read off reinforcing and minimum footing dimensions required.

Examples

The following two examples illustrate the use of the design charts:

Example 1

A wall is to be constructed along a site boundary and will support flat neighbouring land above. The ground to be retained is flat for a distance of 3 m from the wall face, is to be used as a domestic driveway, and is determined to be 2,150 mm above the top of the footing. The site is in Earthquake Zone 1. A geotechnical report for the site has determined that the wall foundation soil is a firm silty clay with design parameters equivalent to the Masonry Manual Type C soil description.

- As the excavation is against the neighbour's property it is determined a Type I wall is appropriate.
- As the wall supports a domestic driveway use surcharge-based charts.
- There are no 140 mm Type I wall with surcharge options and 190 mm Type 1 options only cover up to 2,000 mm height, therefore use ***Type I 240 mm*** *Retaining Wall* - With Surcharge.
- The design dimension table is then referenced and by referring to the Soil C column of Table 1 it is found that the closest greater "Maximum Height" retained is 2,200 mm, giving:

Vertical Reinforcing (70 mm rear cover)	HD16-400
Horizontal Reinforcing	HD12-400
Footing Length "L"	1,700 mm

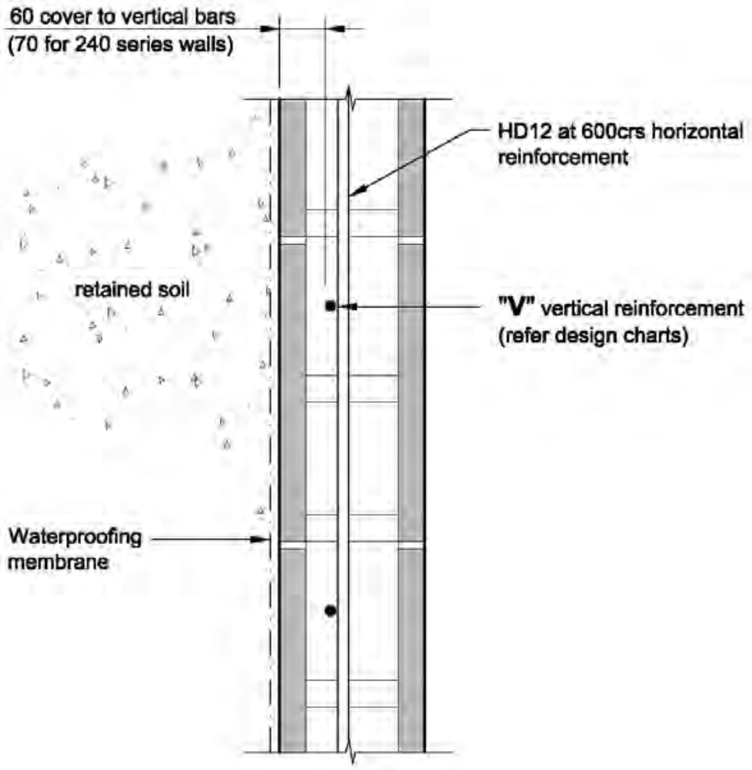
The parameter "K" is shown as 0, therefore there is no requirement for a key for this design.

Example 2

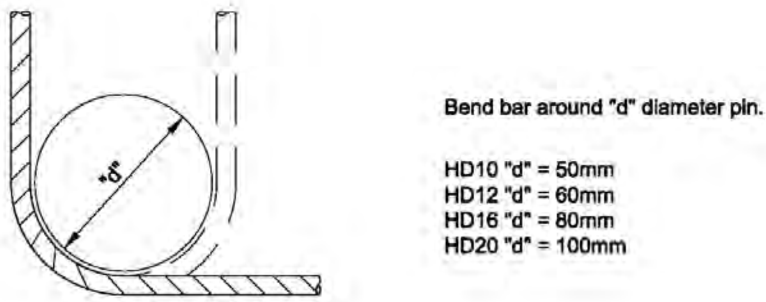
A Type II wall, 1,750 mm high, is to be constructed in a medium dense sandy silt, which a geotechnical report has identified as matching the Masonry Manual Soil B description. The wall has a back slope of 15°. A 190 mm wall is chosen and the *"**Type II 190 mm** Retaining Wall* – With Backslope*"* chart is referenced. The site is within Earthquake Zone 3.

- By entering the Soil B column in Table 3 the following design requirements are found:

Vertical Reinforcing (60 mm rear cover)	HD16-400
Horizontal Reinforcing	HD12-600
Footing Length "L"	1,550 mm
Key Depth "K"	350 mm



PLAN VIEW - REINFORCEMENT PLACEMENT



BAR BEND RADII

Figure 3: Retaining Wall Details

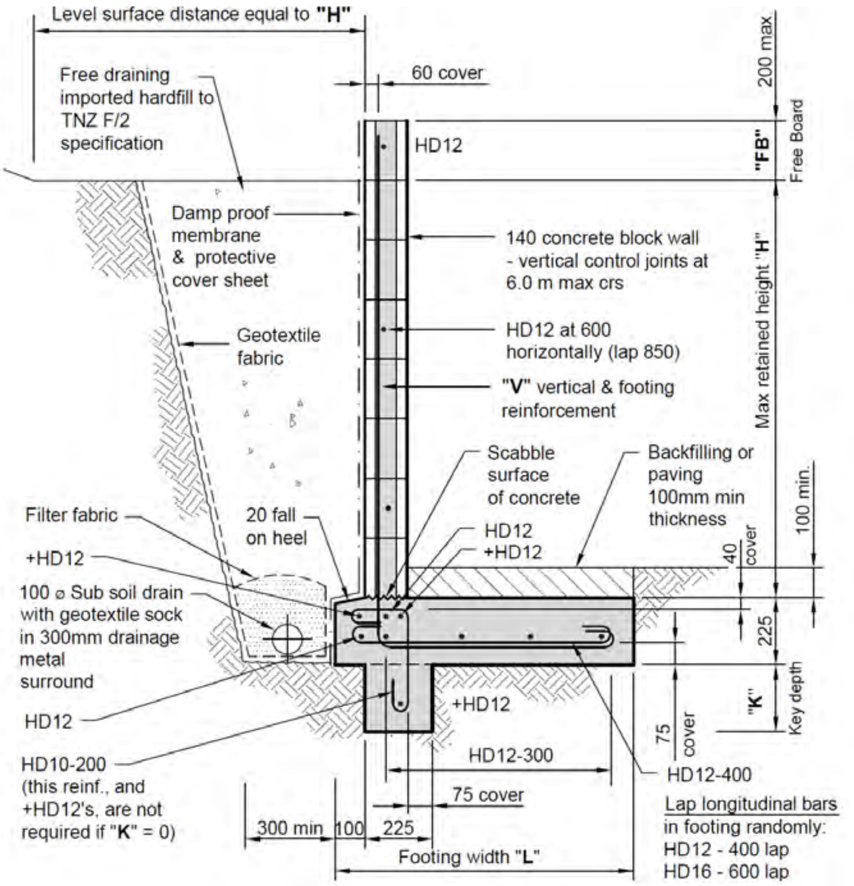
Type I. *140mm Retaining Wall* - Without Surcharge

Do not use in NZS 3604 - Exposure Zone D

140

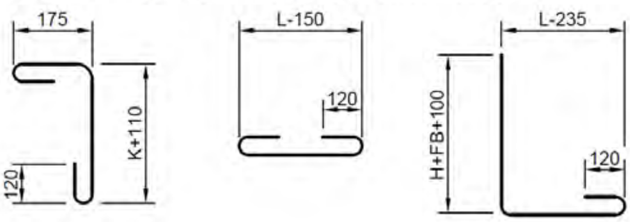
Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	"V"		"V"		"V"	
1200	750	100	750	150	700	0
	HD10-600		HD10-600		HD10-600	
1400	950	100	1000	200	800	0
	HD12-600		HD12-600		HD12-600	
1600	1300	150	1250	250	950	0
	HD12-400		HD12-400		HD12-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	750	100	750	150	700	0
	HD10-600		HD10-600		HD10-600	
1400	950	100	1000	200	850	0
	HD12-600		HD12-600		HD12-600	
1600	1300	150	1250	250	950	0
	HD12-400		HD12-400		HD12-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1400	200	750	150	850	0
	HD12-600		HD12-600		HD12-600	
1400	1950	300	1000	200	950	0
	HD12-400		HD12-400		HD12-400	
1600	*		1250	250	1100	0
			HD12-400		HD12-400	

* Wall/foundation detailing considerations fall outside the scope of these standard designs



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



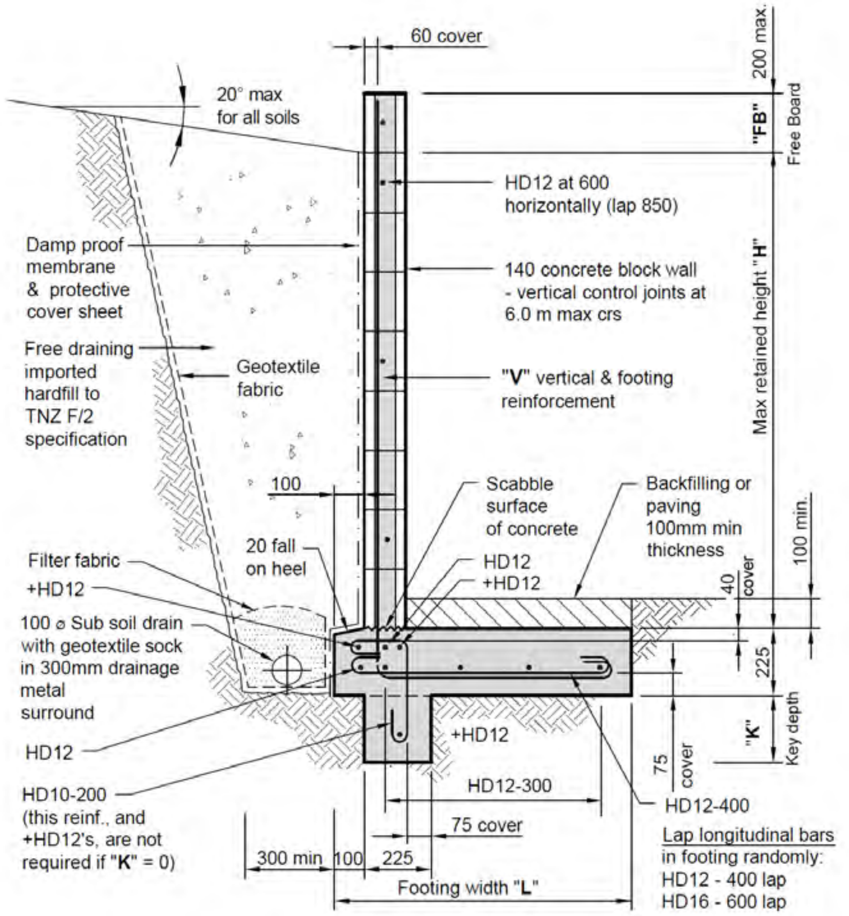
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

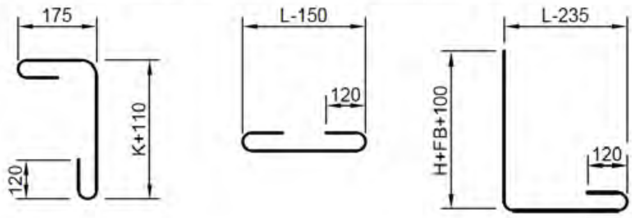
Type I. *140mm Retaining Wall* - With Backslope

Do not use in NZS 3604 - Exposure Zone D



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



140

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	Soil A "V"		Soil B "V"		Soil C "V"	
1200	1250	150	1200	250	800	0
	HD10-600		HD10-600		HD10-600	
1400	1700	200	1550	300	900	0
	HD12-400		HD12-400		HD12-400	
1600	2150	200	2000	350	1100	0
	HD12-400		HD12-400		HD12-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	*		1200	250	850	0
			HD12-600		HD12-600	
1400	*		1550	300	1000	0
			HD12-400		HD12-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	*		1200	250	950	0
			HD12-400		HD12-400	

* Wall/foundation detailing considerations fall outside the scope of these standard designs

NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

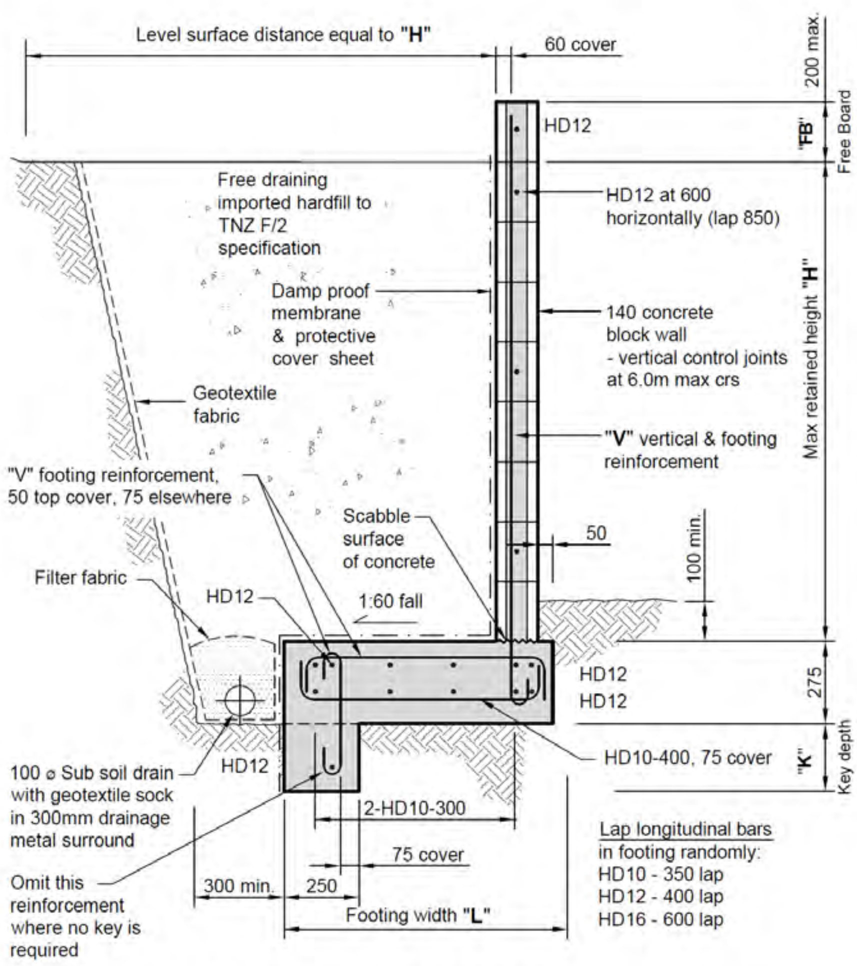
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *140mm Retaining Wall* - Without Surcharge

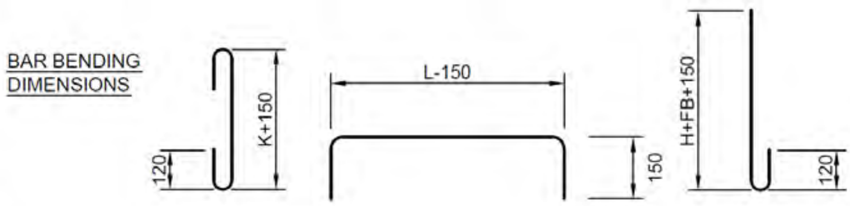
Do not use in NZS 3604 - Exposure Zone D

140

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	Soil A "V"		Soil B "V"		Soil C "V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	800	0	850	0	800	0
	HD10-600		HD10-600		HD10-600	
1400	950	0	950	0	900	0
	HD12-600		HD12-600		HD12-600	
1600	1050	0	1050	0	1050	0
	HD12-400		HD12-400		HD12-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	950	0	1050	0	950	0
	HD10-600		HD10-600		HD10-600	
1400	1200	0	1200	0	1200	0
	HD12-600		HD12-600		HD12-600	
1600	1300	0	1300	0	1300	0
	HD12-400		HD12-400		HD12-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1250	0	1250	0	1250	0
	HD10-600		HD10-600		HD10-600	
1400	1450	0	1500	0	1500	0
	HD12-400		HD12-400		HD12-400	
1600	1700	0	1650	0	1650	0
	HD12-400		HD12-400		HD12-400	



CROSS-SECTION OF RETAINING WALL



NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

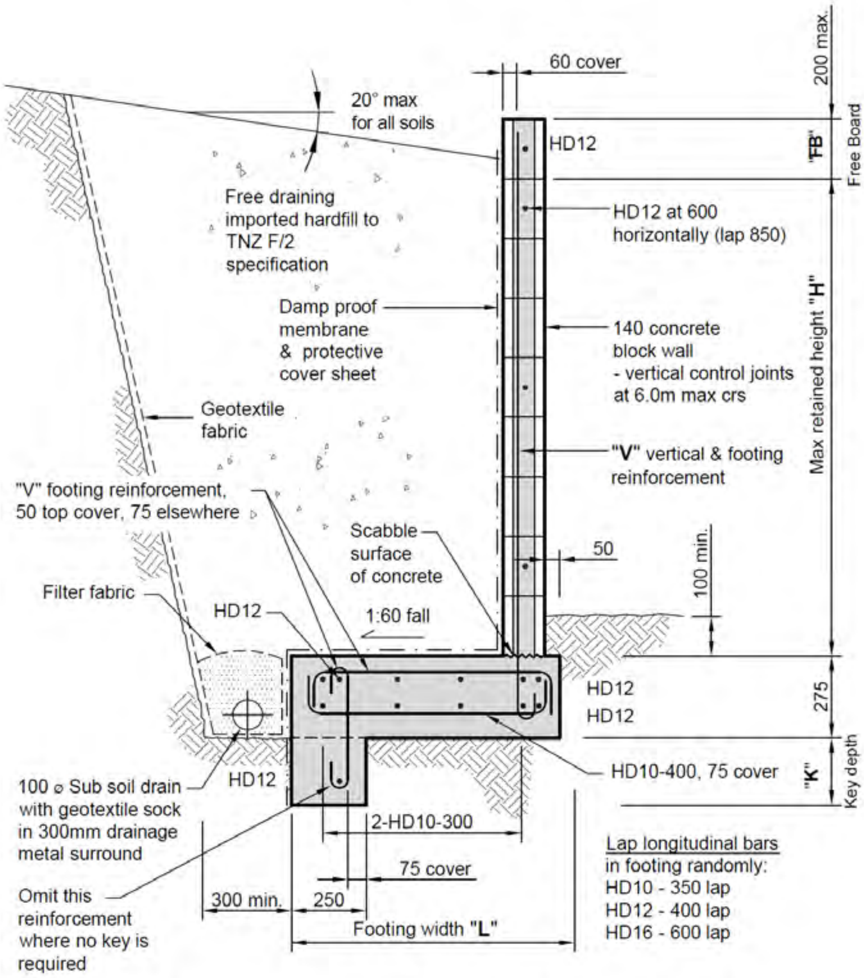
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *140mm Retaining Wall* - With Backslope

Do not use in NZS 3604 - Exposure Zone D

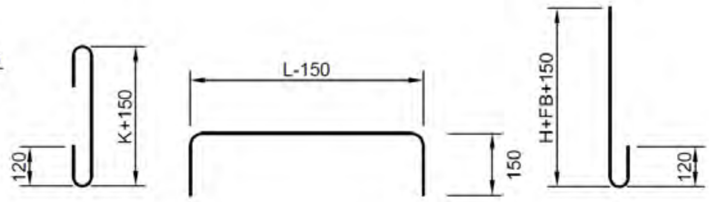
140

Maximum Height "H"	Soil A		Soil B		Soil C	
TABLE 1 ZONE 1 EARTHQUAKE						
	"L"	"K"	"L"	"K"	"L"	"K"
	"V"		"V"		"V"	
1200	900	0	900	0	900	0
	HD10-600		HD10-600		HD10-600	
1400	1000	0	1050	0	1000	0
	HD12-400		HD12-400		HD12-400	
1600	1150	0	1250	0	1150	0
	HD12-400		HD12-400		HD12-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1100	0	1050	0	1050	0
	HD12-600		HD12-600		HD12-600	
1400	1300	100	1200	0	1200	0
	HD12-400		HD12-400		HD12-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1100	0	1250	0	1250	0
	HD12-400		HD12-400		HD12-400	



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



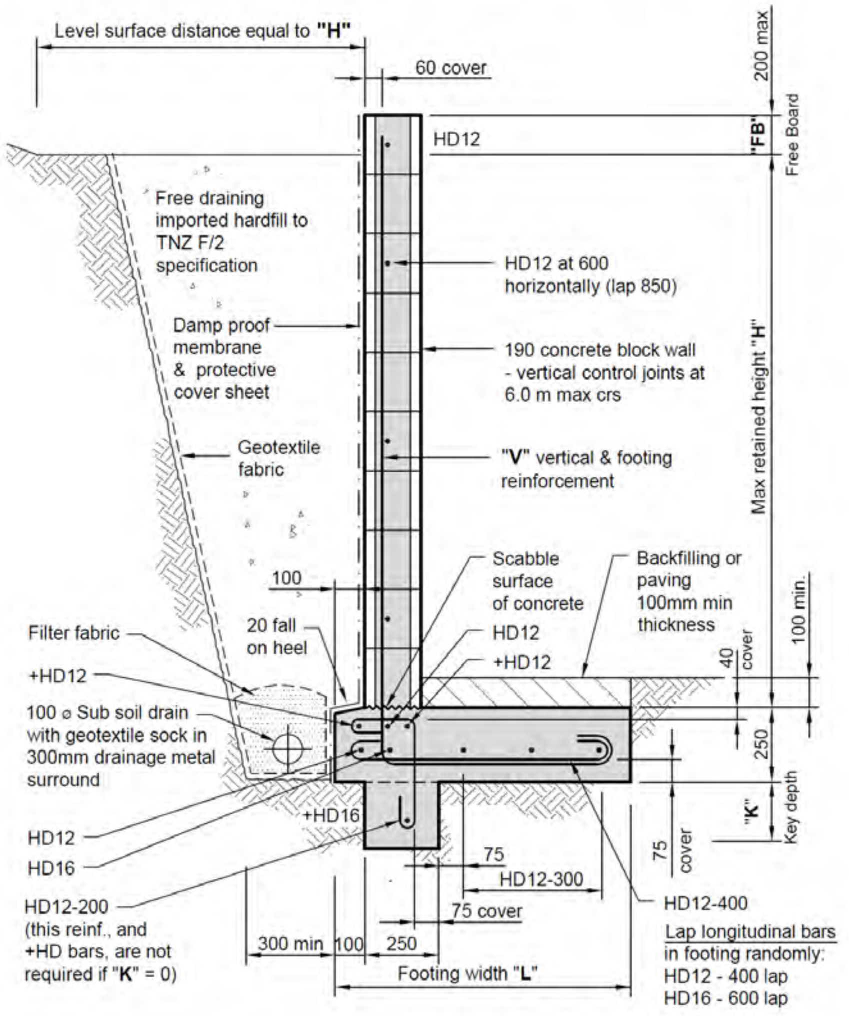
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

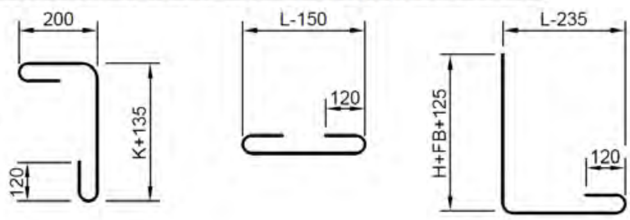
Type I. *190mm Retaining Wall* - Without Surcharge

190



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	"V"		"V"		"V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	750	0	800	0	650	0
	HD12-600		HD12-600		HD12-600	
1400	900	0	950	100	800	0
	HD12-600		HD12-600		HD12-600	
1600	1150	100	1200	200	900	0
	HD12-600		HD12-600		HD12-600	
1800	1500	200	1550	200	1050	0
	HD12-600		HD12-600		HD12-600	
2000	1800	200	1800	300	1150	0
	HD12-400		HD12-400		HD12-400	
2200	2100	200	2200	300	1300	0
	HD16-600		HD16-600		HD16-600	
2400	2500	300	2500	400	1400	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	750	0	800	0	700	0
	HD12-600		HD12-600		HD12-600	
1400	900	0	950	100	850	0
	HD12-600		HD12-600		HD12-600	
1600	1150	100	1200	200	950	0
	HD12-600		HD12-600		HD12-600	
1800	1500	200	1550	200	1050	0
	HD12-600		HD12-600		HD12-600	
2000	1800	200	1800	300	1150	0
	HD12-400		HD12-400		HD12-400	
2200	2100	200	2200	300	1300	0
	HD16-400		HD16-400		HD16-400	
2400	2500	300	2500	400	1400	0
	HD16-400		HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1350	200	850	0	850	0
	HD12-600		HD12-600		HD12-600	
1400	1850	300	950	100	950	0
	HD12-600		HD12-600		HD12-600	
1600	2200	300	1200	200	1050	0
	HD12-600		HD12-600		HD12-600	
1800	2650	400	1550	200	1200	0
	HD12-400		HD12-400		HD12-400	
2000	*		1800	300	1300	0
			HD16-600		HD16-600	
2200	*		2200	300	1400	0
			HD16-400		HD16-400	

* Wall/foundation detailing considerations fall outside the scope of these standard designs

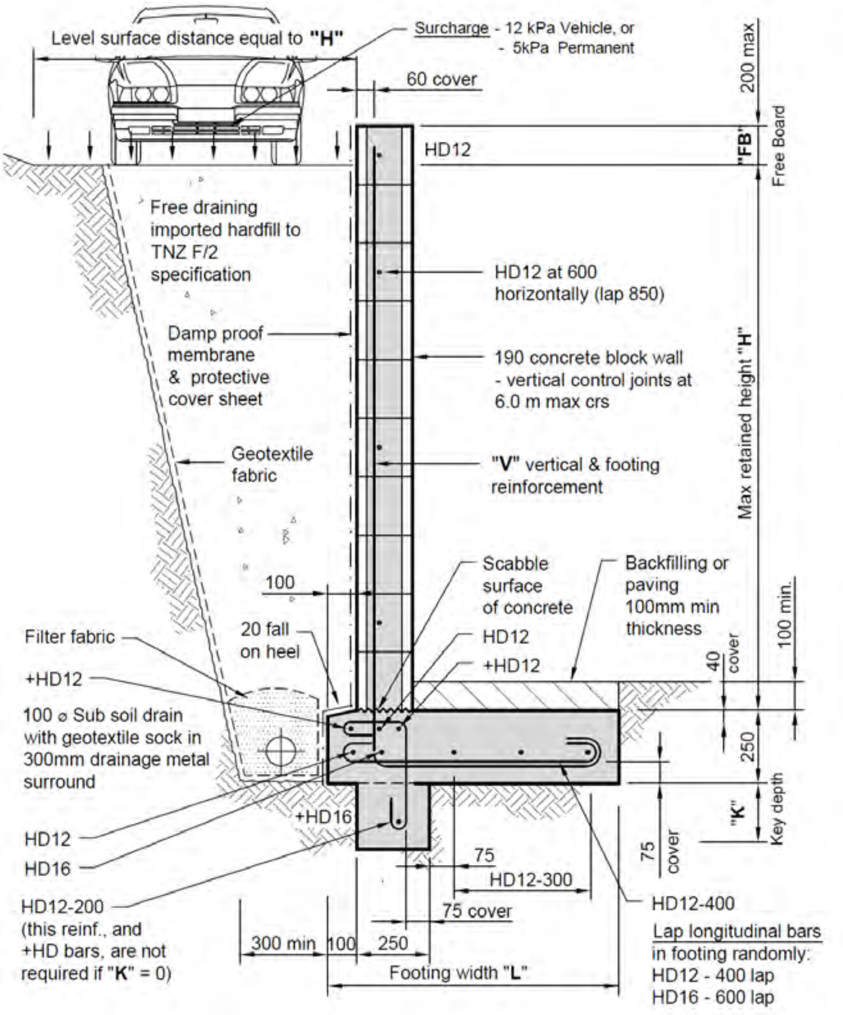
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

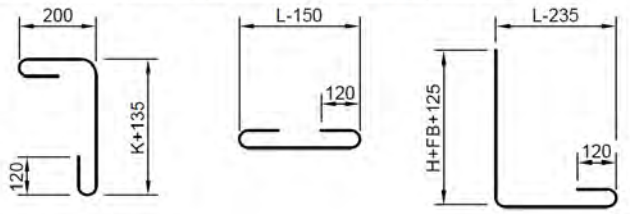
Type I. *190mm Retaining Wall* - With Surcharge

190



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	"V"		"V"		"V"	
1200	1550	150	1550	250	1000	0
	HD12-400		HD12-400		HD12-400	
1400	1900	200	1900	300	1150	0
	HD12-400		HD12-400		HD12-400	
1600	2300	250	2250	350	1300	0
	HD16-600		HD16-600		HD16-600	
1800	2650	250	2600	400	1450	0
	HD16-400		HD16-400		HD16-400	
2000	3150	300	3000	450	1600	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1550	150	1550	250	1000	0
	HD12-400		HD12-400		HD12-400	
1400	1900	200	1900	300	1150	0
	HD12-400		HD12-400		HD12-400	
1600	2300	250	2250	350	1300	0
	HD16-600		HD16-600		HD16-600	
1800	2650	250	2600	400	1450	0
	HD16-400		HD16-400		HD16-400	
2000	3150	300	3000	450	1600	0
	HD16-400		HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1750	200	2000	350	1000	0
	HD12-400		HD12-400		HD12-400	
1400	2200	250	2300	350	1150	0
	HD12-400		HD12-400		HD12-400	
1600	2600	250	2700	400	1300	0
	HD16-600		HD16-600		HD16-600	
1800	3050	250	3100	450	1450	0
	HD16-400		HD16-400		HD16-400	
2000	*		3100	450	1600	0
			HD16-400		HD16-400	

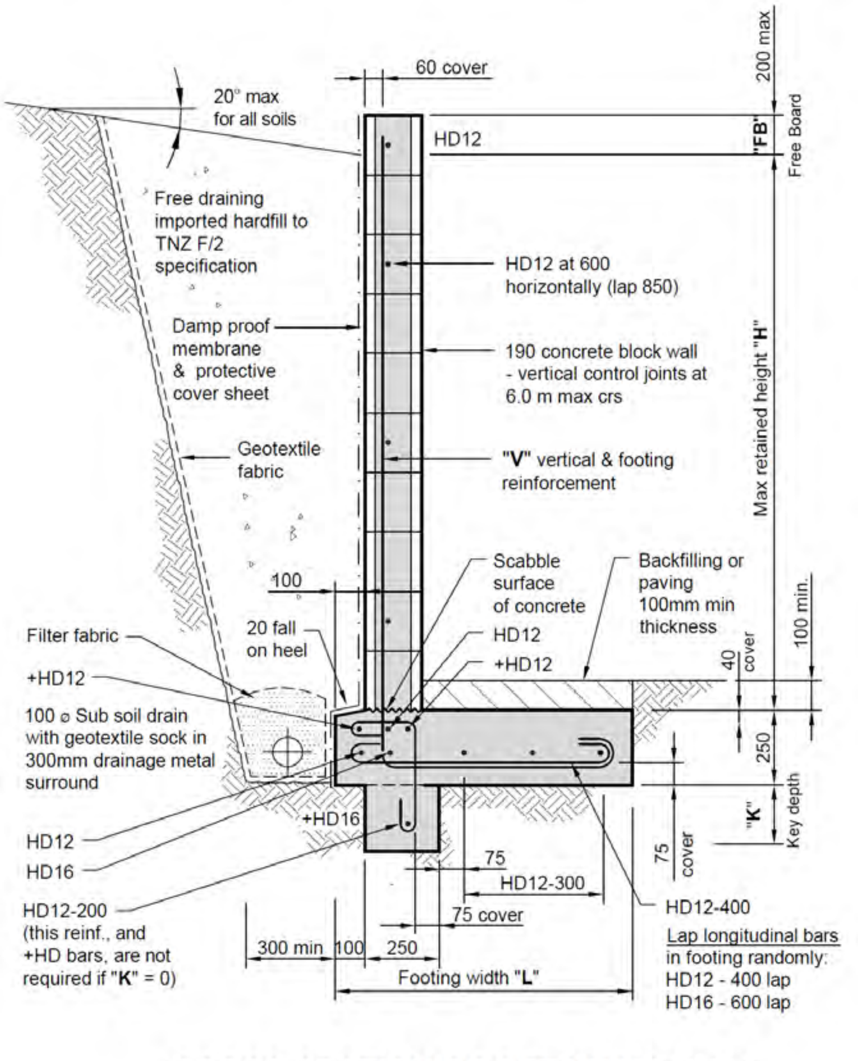
* Wall/foundation detailing considerations fall outside the scope of these standard designs

NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

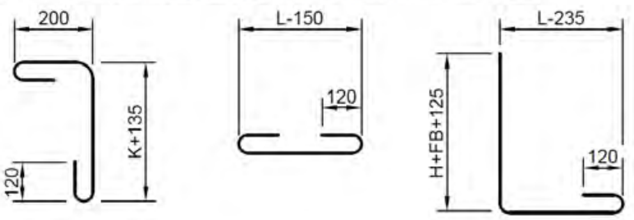
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type I. *190mm Retaining Wall* - With Backslope



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



190

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	"V"		"V"		"V"	
1200	1000	100	1100	200	800	0
	HD12-600		HD12-600		HD12-600	
1400	1450	200	1450	250	900	0
	HD12-600		HD12-600		HD12-600	
1600	1800	200	1850	300	1050	0
	HD12-600		HD12-600		HD12-600	
1800	2300	200	2250	400	1200	0
	HD12-400		HD12-400		HD12-400	
2000	2850	300	2700	450	1350	0
	HD16-600		HD16-600		HD16-600	
2200	3300	300	3200	500	1500	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	*		1100	200	850	0
			HD12-600		HD12-600	
1400	*		1450	250	1000	0
			HD12-400		HD12-400	
1600	*		1850	300	1150	0
			HD16-600		HD16-600	
1800	*		2250	400	1250	0
			HD16-600		HD16-600	
2000	*		2700	450	1400	0
			HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	*		1100	200	950	0
			HD12-600		HD12-600	
1400	*		1450	250	1100	0
			HD12-400		HD12-400	
1600	*		1850	300	1250	0
			HD12-400		HD12-400	
1800	*		2250	400	1450	0
			HD16-400		HD16-400	

* Wall/foundation detailing considerations fall outside the scope of these standard designs

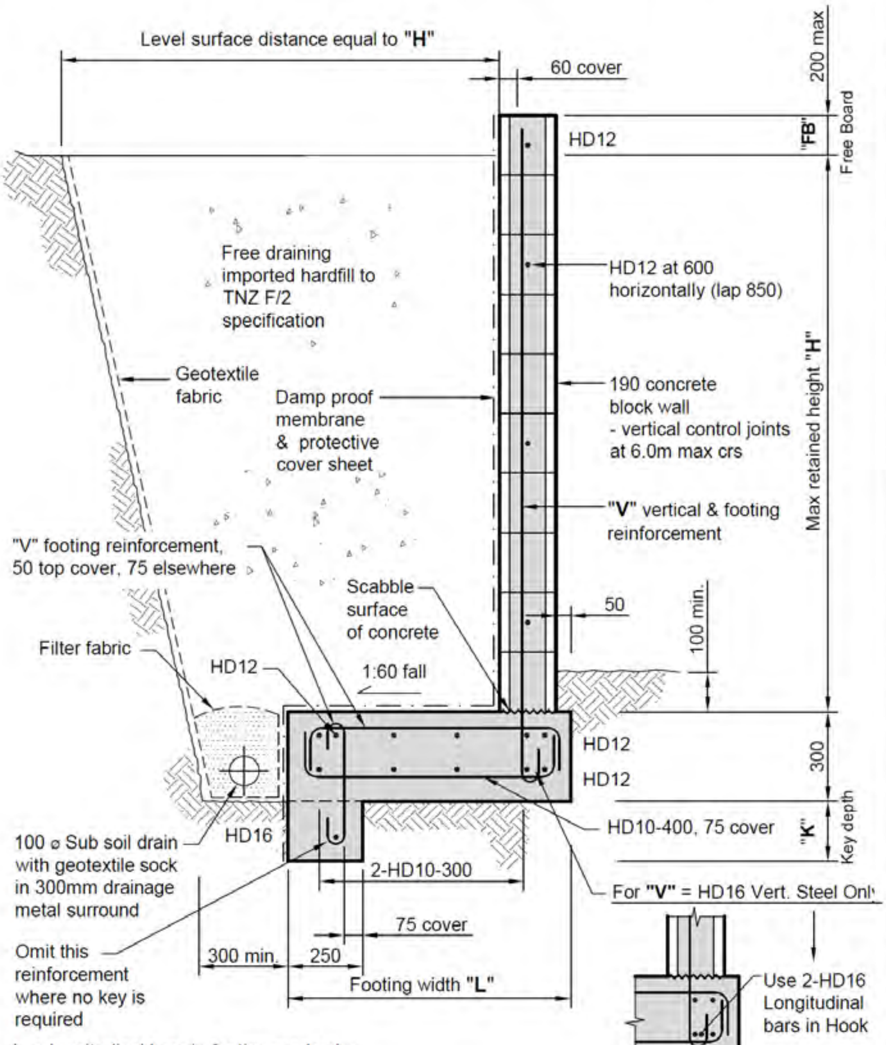
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *190mm Retaining Wall* - Without Surcharge

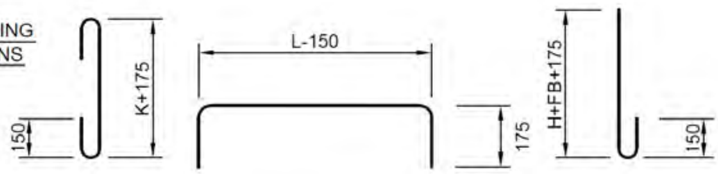
190



Lap longitudinal bars in footing randomly:
HD10 - 350 lap; HD12 - 400 lap; HD16 - 600 lap

CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	Soil A "V"		Soil B "V"		Soil C "V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	850	0	850	0	850	0
	HD12-600		HD12-600		HD12-600	
1400	950	0	950	0	950	0
	HD12-600		HD12-600		HD12-600	
1600	1050	0	1050	0	1050	0
	HD12-600		HD12-600		HD12-600	
1800	1250	0	1250	0	1250	0
	HD12-600		HD12-600		HD12-600	
2000	1350	0	1350	0	1350	0
	HD12-400		HD12-400		HD12-400	
2200	1450	0	1500	0	1500	0
	HD16-600		HD16-600		HD16-600	
2400	1550	0	1650	0	1600	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1050	0	1050	0	1050	0
	HD12-600		HD12-600		HD12-600	
1400	1200	0	1200	0	1200	0
	HD12-600		HD12-600		HD12-600	
1600	1300	0	1300	0	1300	0
	HD12-600		HD12-600		HD12-600	
1800	1450	0	1450	0	1450	0
	HD12-600		HD12-600		HD12-600	
2000	1600	0	1550	0	1550	0
	HD12-400		HD12-400		HD12-400	
2200	1750	0	1700	0	1700	0
	HD16-400		HD16-400		HD16-400	
2400	1850	0	1850	0	1850	0
	HD16-400		HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1350	0	1250	0	1250	0
	HD12-600		HD12-600		HD12-600	
1400	1450	0	1500	0	1500	0
	HD12-600		HD12-600		HD12-600	
1600	1700	0	1650	0	1650	0
	HD12-600		HD12-600		HD12-600	
1800	1850	0	1850	0	1850	0
	HD12-400		HD12-400		HD12-400	
2000	2050	0	2000	0	2000	0
	HD16-600		HD16-600		HD16-600	
2200	2150	0	2200	0	2200	0
	HD16-400		HD16-400		HD16-400	

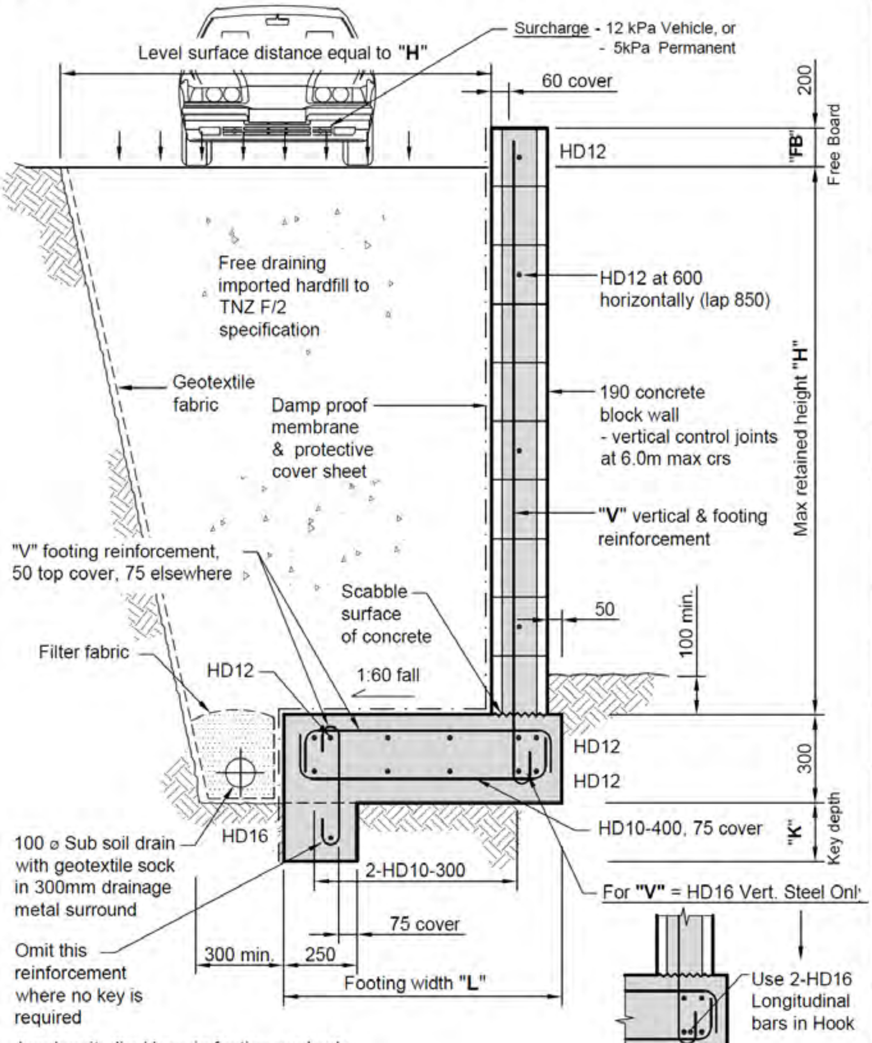
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *190mm Retaining Wall* - With Surcharge

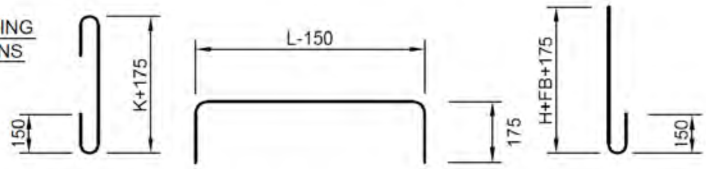
190



CROSS-SECTION OF RETAINING WALL

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	"V"		"V"		"V"	
1200	1000	0	1150	0	1000	0
	HD12-400		HD12-400		HD12-400	
1400	1150	0	1300	0	1150	0
	HD12-400		HD12-400		HD12-400	
1600	1250	0	1450	0	1250	0
	HD12-400		HD12-400		HD12-400	
1800	1350	0	1600	0	1350	0
	HD16-400		HD16-400		HD16-400	
2000	1450	0	1800	0	1450	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1100	0	1150	0	1150	0
	HD12-400		HD12-400		HD12-400	
1400	1200	0	1300	0	1250	0
	HD12-400		HD12-400		HD12-400	
1600	1350	0	1450	0	1350	0
	HD12-400		HD12-400		HD12-400	
1800	1500	0	1600	0	1500	0
	HD16-400		HD16-400		HD16-400	
2000	1650	0	1800	0	1650	0
	HD16-400		HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1350	0	1350	0	1350	0
	HD12-400		HD12-400		HD12-400	
1400	1500	0	1500	0	1500	0
	HD12-400		HD12-400		HD12-400	
1600	1700	0	1700	0	1650	0
	HD16-600		HD16-600		HD16-600	
1800	1850	0	1850	0	1850	0
	HD16-400		HD16-400		HD16-400	
2000	2000	0	2000	0	2000	0
	HD16-400		HD16-400		HD16-400	

BAR BENDING DIMENSIONS

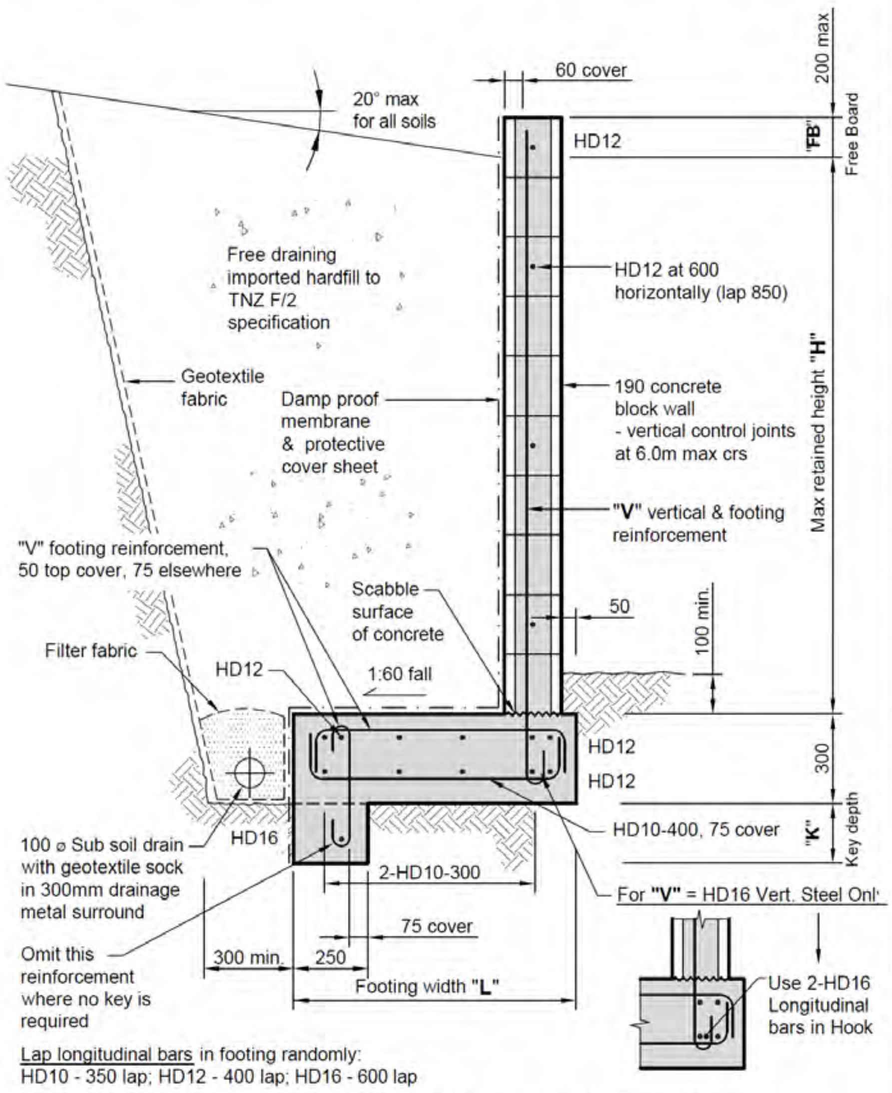


NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

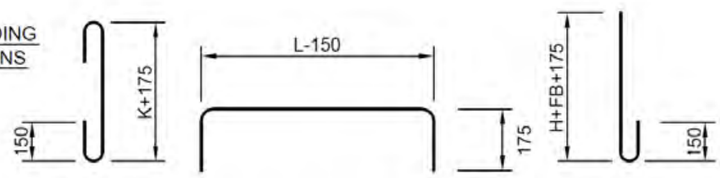
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *190mm Retaining Wall* - **With Backslope**



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



190

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	"V"		"V"		"V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	900	0	900	0	900	0
	HD12-600		HD12-600		HD12-600	
1400	1000	0	1050	0	1000	0
	HD12-600		HD12-600		HD12-600	
1600	1150	0	1250	0	1150	0
	HD12-600		HD12-600		HD12-600	
1800	1250	0	1450	0	1250	0
	HD12-400		HD12-400		HD12-400	
2000	1400	0	1600	0	1400	0
	HD16-600		HD16-600		HD16-600	
2200	1500	0	1800	0	1500	0
	HD16-400		HD16-400		HD16-400	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1100	0	1050	0	1050	0
	HD12-600		HD12-600		HD12-600	
1400	1300	100	1200	0	1200	0
	HD12-600		HD12-600		HD12-600	
1600	1500	150	1350	0	1250	100
	HD12-400		HD12-400		HD12-400	
1800	1700	200	1500	0	1350	150
	HD16-600		HD16-600		HD16-600	
2000	1900	250	1650	100	1400	200
	HD16-400		HD16-400		HD16-400	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	*		1250	0	1250	0
			HD12-600		HD12-600	
1400	*		1400	0	1250	100
			HD12-400		HD12-400	
1600	*		1450	150	1300	200
			HD12-400		HD12-400	
1800	*		1550	350	1450	300
			HD16-400		HD16-400	

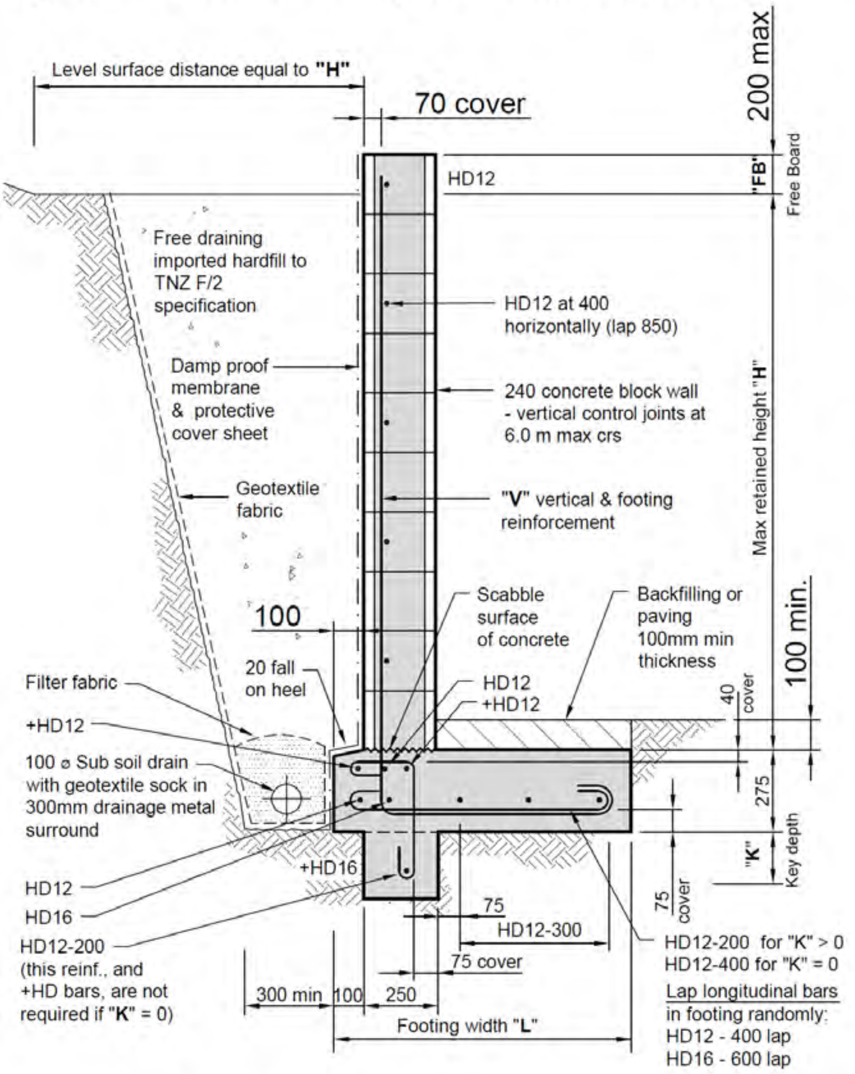
* Wall/foundation detailing considerations fall outside the scope of these standard designs

NOTES

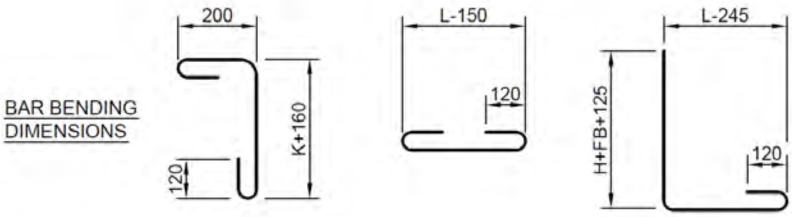
1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type I. *240mm Retaining Wall* - Without Surcharge



CROSS-SECTION OF RETAINING WALL



240

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
TABLE 1 ZONE 1 EARTHQUAKE						
	"V"		"V"		"V"	
1200	750	0	800	0	650	0
	HD12-400		HD12-400		HD12-400	
1400	900	0	950	100	800	0
	HD12-400		HD12-400		HD12-400	
1600	1150	100	1200	200	900	0
	HD12-400		HD12-400		HD12-400	
1800	1500	200	1550	200	1050	0
	HD12-400		HD12-400		HD12-400	
2000	1800	200	1800	300	1150	0
	HD12-400		HD12-400		HD12-400	
2200	2100	200	2200	300	1300	0
	HD16-600		HD16-600		HD16-600	
2400	2500	300	2500	400	1400	0
	HD16-400		HD16-400		HD16-400	
2600	2850	300	2950	500	1600	0
	HD16-200		HD16-200		HD16-200	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	750	0	800	0	700	0
	HD12-400		HD12-400		HD12-400	
1400	900	0	950	100	850	0
	HD12-400		HD12-400		HD12-400	
1600	1150	100	1200	200	950	0
	HD12-400		HD12-400		HD12-400	
1800	1500	200	1550	200	1050	0
	HD12-400		HD12-400		HD12-400	
2000	1800	200	1800	300	1150	0
	HD12-400		HD12-400		HD12-400	
2200	2100	200	2200	300	1300	0
	HD16-400		HD16-400		HD16-400	
2400	2500	300	2500	400	1400	0
	HD16-400		HD16-400		HD16-400	
2600	2850	300	2950	500	1600	0
	HD16-200		HD16-200		HD16-200	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1350	200	850	0	850	0
	HD12-400		HD12-400		HD12-400	
1400	1850	300	950	100	950	0
	HD12-400		HD12-400		HD12-400	
1600	2200	300	1200	200	1050	0
	HD12-400		HD12-400		HD12-400	
1800	2650	400	1550	200	1200	0
	HD12-400		HD12-400		HD12-400	
2000	*		1800	300	1300	0
			HD16-600		HD16-600	
2200	*		2200	300	1400	0
			HD16-400		HD16-400	
2400	*		2500	400	1600	0
			HD16-200		HD16-200	
2600	*		2950	500	1700	0
			HD16-200		HD16-200	

* Wall/foundation detailing considerations fall outside the scope of these standard designs

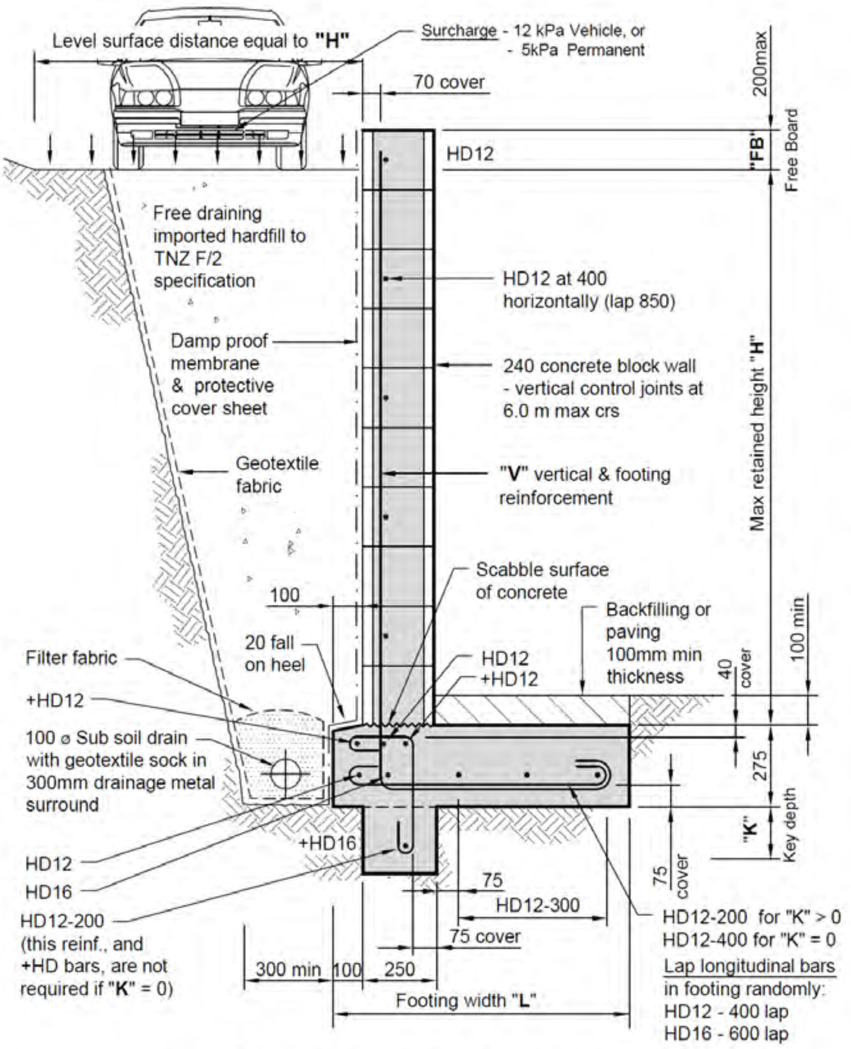
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

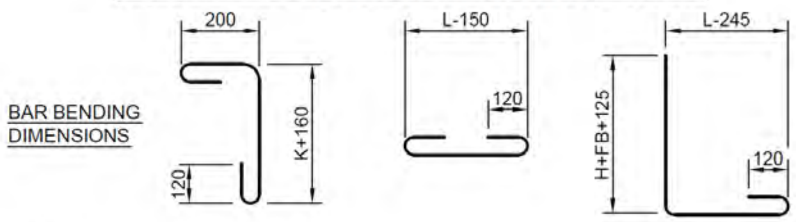
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type I. *240mm Retaining Wall* - With Surcharge

240



CROSS-SECTION OF RETAINING WALL



Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	"V"		"V"		"V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	1400	150	1500	300	1000	0
	HD16-600		HD16-600		HD16-600	
1400	1750	200	1800	350	1100	0
	HD16-600		HD16-600		HD16-600	
1600	2150	250	2150	400	1250	0
	HD16-600		HD16-600		HD16-600	
1800	2500	250	2450	400	1400	0
	HD16-600		HD16-600		HD16-600	
2000	2950	300	2800	450	1550	0
	HD16-400		HD16-400		HD16-400	
2200	3350	300	3200	500	1700	0
	HD16-400		HD16-400		HD16-400	
2400	*		*		1800	0
					HD16-200	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1400	150	1500	300	1000	0
	HD16-600		HD16-600		HD16-600	
1400	1750	200	1800	350	1100	0
	HD16-600		HD16-600		HD16-600	
1600	2150	250	2150	400	1250	0
	HD16-600		HD16-600		HD16-600	
1800	2500	250	2450	400	1400	0
	HD16-600		HD16-600		HD16-600	
2000	2950	300	2800	450	1550	0
	HD16-400		HD16-400		HD16-400	
2200	3350	300	3200	500	1700	0
	HD16-400		HD16-400		HD16-400	
2400	*		*		1800	0
					HD16-200	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1550	150	1500	300	1000	0
	HD16-600		HD16-600		HD16-600	
1400	2100	250	1800	350	1100	0
	HD16-600		HD16-600		HD16-600	
1600	2600	300	2150	400	1250	0
	HD16-600		HD16-600		HD16-600	
1800	3250	400	2450	400	1400	0
	HD16-600		HD16-600		HD16-600	
2000	3800	450	2800	450	1550	0
	HD16-400		HD16-400		HD16-400	
2200	4450	500	3200	500	1700	0
	HD16-400		HD16-400		HD16-400	
2400	*		*		1800	0
					HD16-200	

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NOTES

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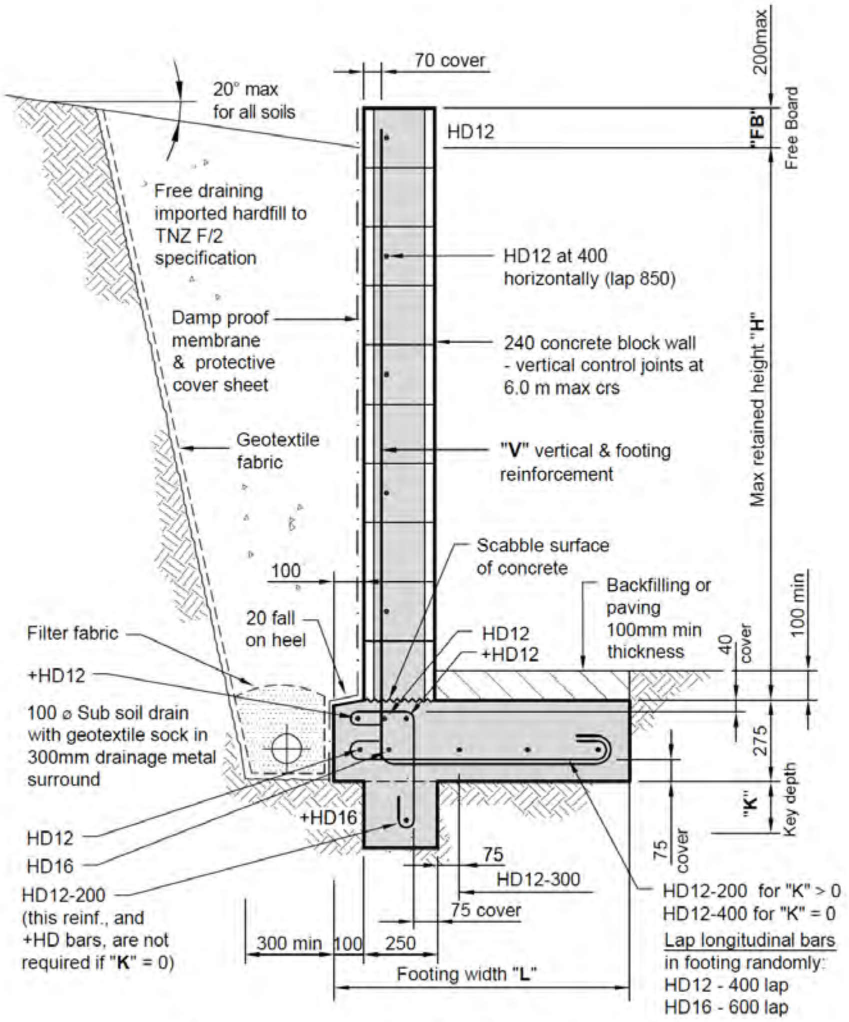
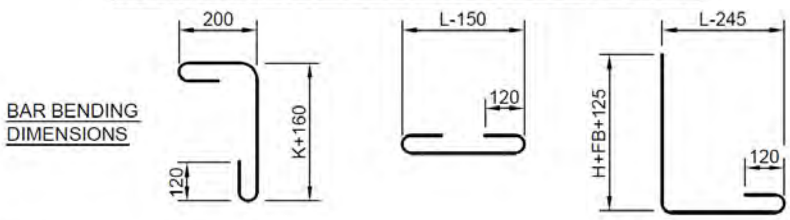
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type I. *240mm Retaining Wall* - With Backslope



CROSS-SECTION OF RETAINING WALL



Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	"V"		"V"		"V"	
240						
TABLE 1 ZONE 1 EARTHQUAKE						
1200	1000	100	1100	200	800	0
	HD12-400		HD12-400		HD12-400	
1400	1450	200	1450	250	900	0
	HD12-400		HD12-400		HD12-400	
1600	1800	200	1850	300	1050	0
	HD12-400		HD12-400		HD12-400	
1800	2300	200	2250	400	1200	0
	HD12-400		HD12-400		HD12-400	
2000	2850	300	2700	450	1350	0
	HD16-600		HD16-600		HD16-600	
2200	3300	300	3200	500	1500	0
	HD16-400		HD16-400		HD16-400	
2400	*		*		1700	0
					HD16-200	
2600	*		*		1900	0
					HD16-200	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	*		1100	200	850	0
			HD12-400		HD12-400	
1400	*		1450	250	1000	0
			HD12-400		HD12-400	
1600	*		1850	300	1150	0
			HD16-600		HD16-600	
1800	*		2250	400	1250	0
			HD16-600		HD16-600	
2000	*		2700	450	1400	0
			HD16-400		HD16-400	
2200	*		3200	500	1500	0
			HD16-200		HD16-200	
2400	*		*		1700	0
					HD16-200	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	*		1100	200	950	0
			HD12-400		HD12-400	
1400	*		1450	250	1100	0
			HD12-400		HD12-400	
1600	*		1850	300	1250	0
			HD12-400		HD12-400	
1800	*		2250	400	1450	0
			HD16-400		HD16-400	
2000	*		2700	450	1650	0
			HD16-200		HD16-200	
2200	*		3200	500	1850	0
			HD16-200		HD16-200	
2400	*		*		2100	0
					HD16-200	

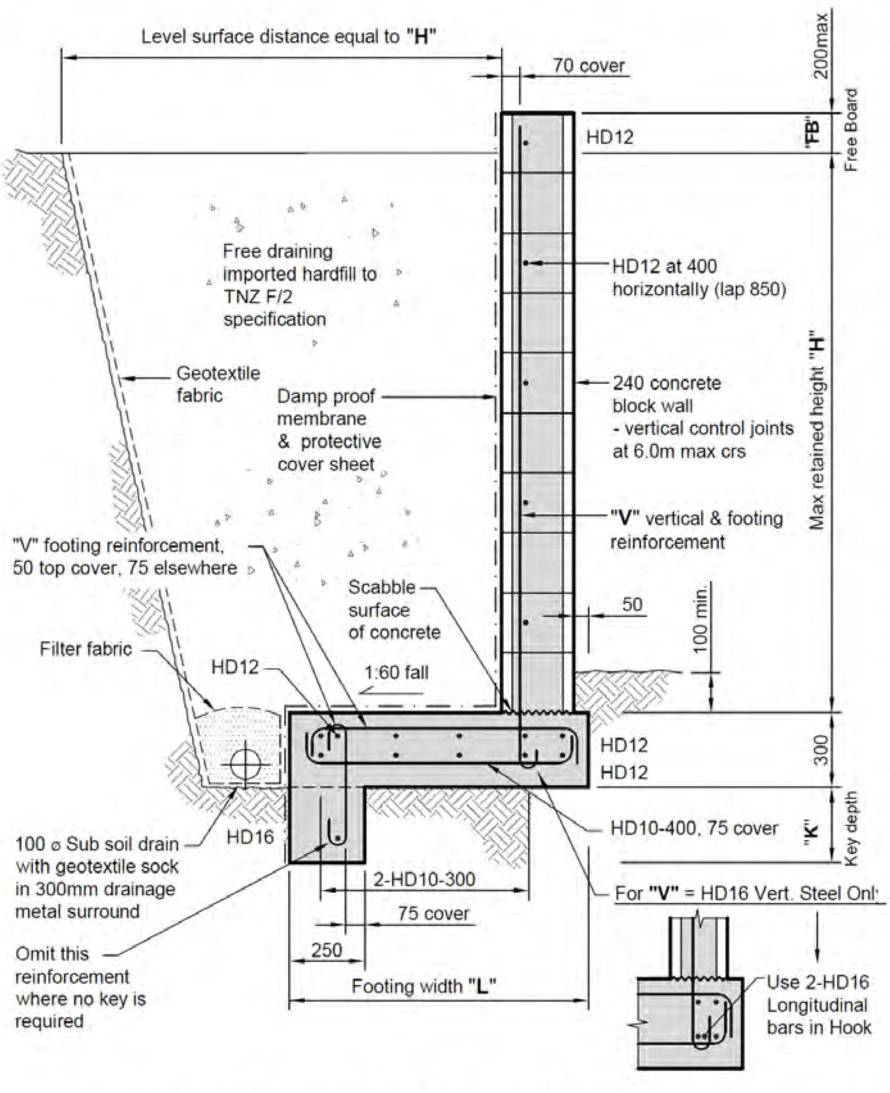
∗ Wall/foundation detailing considerations fall outside the scope of these standard designs

NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

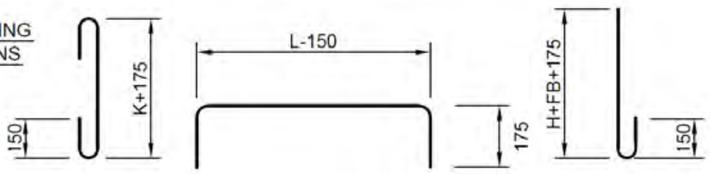
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *240mm Retaining Wall* - Without Surcharge



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



240

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
	"V"		"V"		"V"	
TABLE 1 ZONE 1 EARTHQUAKE						
1200	850	0	850	0	850	0
	HD12-400		HD12-400		HD12-400	
1400	950	0	950	0	950	0
	HD12-400		HD12-400		HD12-400	
1600	1050	0	1050	0	1050	0
	HD12-400		HD12-400		HD12-400	
1800	1250	0	1250	0	1250	0
	HD12-400		HD12-400		HD12-400	
2000	1350	0	1350	0	1350	0
	HD12-400		HD12-400		HD12-400	
2200	1450	0	1500	0	1500	0
	HD16-600		HD16-600		HD16-600	
2400	1550	0	1650	0	1600	0
	HD16-400		HD16-400		HD16-400	
2600	1650	0	1800	0	1650	0
	HD16-200		HD16-200		HD16-200	
TABLE 2 ZONE 2 EARTHQUAKE						
1200	1050	0	1050	0	1050	0
	HD12-400		HD12-400		HD12-400	
1400	1200	0	1200	0	1200	0
	HD12-400		HD12-400		HD12-400	
1600	1300	0	1300	0	1300	0
	HD12-400		HD12-400		HD12-400	
1800	1450	0	1450	0	1450	0
	HD12-400		HD12-400		HD12-400	
2000	1600	0	1550	0	1550	0
	HD12-400		HD12-400		HD12-400	
2200	1750	0	1700	0	1700	0
	HD16-400		HD16-400		HD16-400	
2400	1850	0	1850	0	1850	0
	HD16-400		HD16-400		HD16-400	
2600	2000	0	2000	0	2000	0
	HD16-200		HD16-200		HD16-200	
TABLE 3 ZONE 3 EARTHQUAKE						
1200	1350	0	1250	0	1250	0
	HD12-400		HD12-400		HD12-400	
1400	1450	0	1500	0	1500	0
	HD12-400		HD12-400		HD12-400	
1600	1700	0	1650	0	1650	0
	HD12-400		HD12-400		HD12-400	
1800	1850	0	1850	0	1850	0
	HD12-400		HD12-400		HD12-400	
2000	2050	0	2000	0	2000	0
	HD16-600		HD16-600		HD16-600	
2200	2150	0	2200	0	2200	0
	HD16-400		HD16-400		HD16-400	
2400	2350	0	2350	0	2350	0
	HD16-200		HD16-200		HD16-200	
2600	2550	0	2550	0	2550	0
	HD16-200		HD16-200		HD16-200	

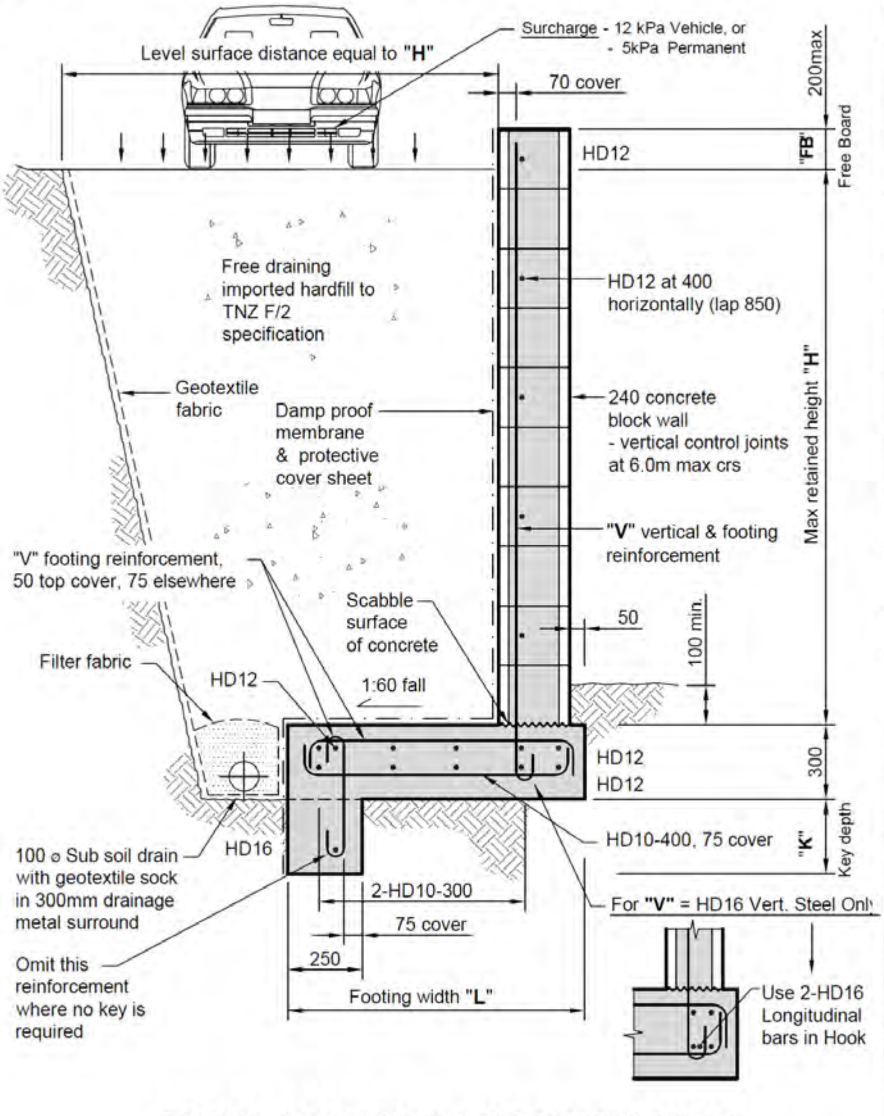
NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

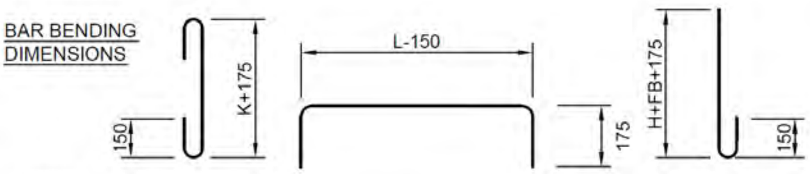
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *240mm Retaining Wall* - With Surcharge

240



CROSS-SECTION OF RETAINING WALL



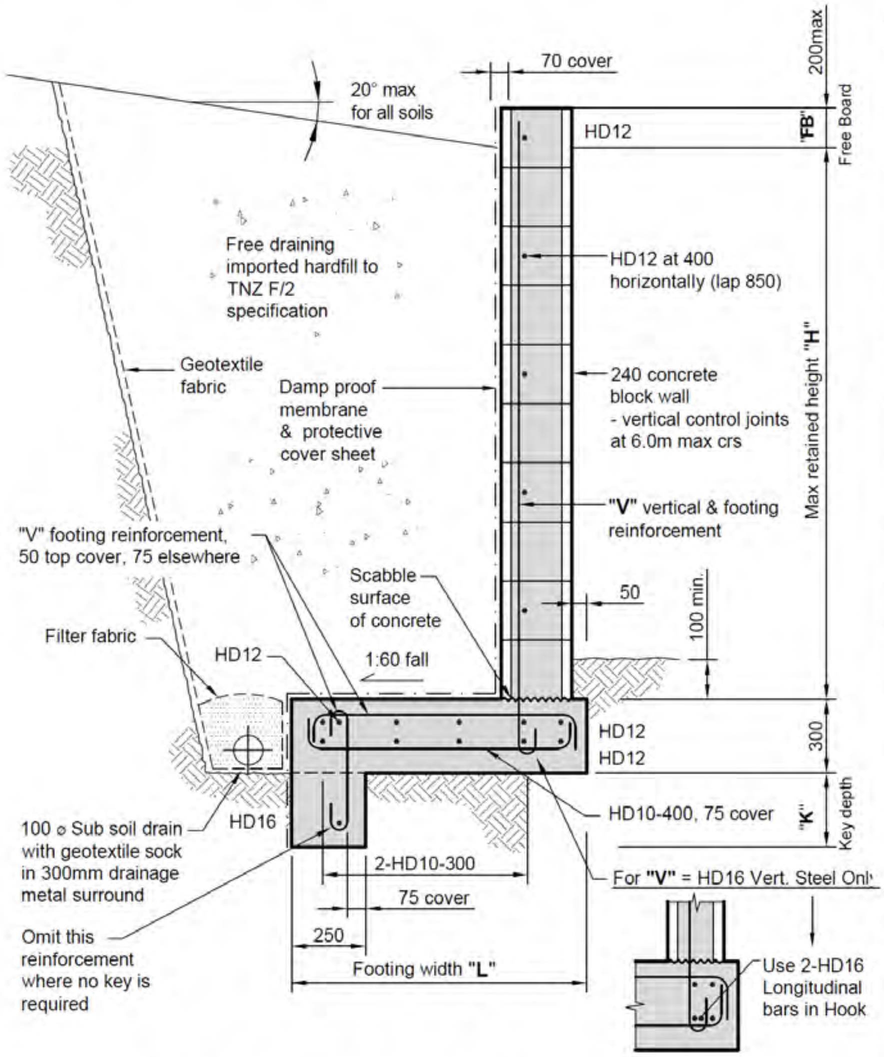
Maximum Height "H"	Soil A "L"	Soil A "K"	Soil A "V"	Soil B "L"	Soil B "K"	Soil B "V"	Soil C "L"	Soil C "K"	Soil C "V"
TABLE 1 ZONE 1 EARTHQUAKE									
1200	1050	0	HD16-600	1150	0	HD16-600	1000	0	HD16-600
1400	1150	0	HD16-600	1300	0	HD16-600	1150	0	HD16-600
1600	1250	0	HD16-600	1450	0	HD16-600	1250	0	HD16-600
1800	1350	0	HD16-400	1600	0	HD16-400	1350	0	HD16-400
2000	1450	0	HD16-400	1800	0	HD16-400	1450	0	HD16-400
2200	1600	0	HD16-400	2000	0	HD16-400	1550	0	HD16-400
2400	1700	0	HD16-200	2100	0	HD16-200	1650	0	HD16-200
2600	1850	0	HD16-200	2300	0	HD16-200	1700	0	HD16-200
TABLE 2 ZONE 2 EARTHQUAKE									
1200	1100	0	HD16-600	1150	0	HD16-600	1100	0	HD16-600
1400	1250	0	HD16-600	1300	0	HD16-600	1250	0	HD16-600
1600	1350	0	HD16-600	1450	0	HD16-600	1350	0	HD16-600
1800	1500	0	HD16-600	1600	0	HD16-600	1500	0	HD16-600
2000	1650	0	HD16-400	1800	0	HD16-400	1650	0	HD16-400
2200	1800	0	HD16-400	2150	0	HD16-400	1800	0	HD16-400
2400	1900	0	HD16-200	2100	0	HD16-200	1900	0	HD16-200
2600	2050	0	HD16-200	2300	0	HD16-200	2050	0	HD16-200
TABLE 3 ZONE 3 EARTHQUAKE									
1200	1350	0	HD16-600	1350	0	HD16-600	1350	0	HD16-600
1400	1500	0	HD16-600	1500	0	HD16-600	1500	0	HD16-600
1600	1700	0	HD16-600	1700	0	HD16-600	1700	0	HD16-600
1800	1850	0	HD16-400	1850	0	HD16-400	1850	0	HD16-400
2000	2000	0	HD16-400	2000	0	HD16-400	2050	0	HD16-400
2200	2200	0	HD16-200	2200	0	HD16-200	2200	0	HD16-200
2400	2400	0	HD16-200	2350	0	HD16-200	2400	0	HD16-200
2600	2550	0	HD16-200	2550	0	HD16-200	2550	100	HD16-200

NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

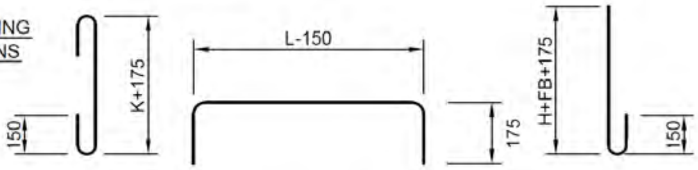
		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

Type II. *240mm Retaining Wall* - **With Backslope**



CROSS-SECTION OF RETAINING WALL

BAR BENDING DIMENSIONS



240

TABLE 1 — ZONE 1 EARTHQUAKE

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
1200	900	0	900	0	900	0
	HD12-400		HD12-400		HD12-400	
1400	1000	0	1050	0	1000	0
	HD12-400		HD12-400		HD12-400	
1600	1150	0	1250	0	1150	0
	HD12-400		HD12-400		HD12-400	
1800	1250	0	1450	0	1250	0
	HD12-400		HD12-400		HD12-400	
2000	1400	0	1600	0	1400	0
	HD16-600		HD16-600		HD16-600	
2200	1500	0	1800	0	1500	0
	HD16-400		HD16-400		HD16-400	
2400	1650	0	2000	0	1650	0
	HD16-200		HD16-200		HD16-200	
2600	1750	0	2150	0	1750	0
	HD16-200		HD16-200		HD16-200	

(Second row of each height: "V")

TABLE 2 — ZONE 2 EARTHQUAKE

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
1200	1100	0	1050	0	1050	0
	HD12-400		HD12-400		HD12-400	
1400	1300	100	1200	0	1200	0
	HD12-400		HD12-400		HD12-400	
1600	1500	150	1350	0	1250	100
	HD12-400		HD12-400		HD12-400	
1800	1700	200	1500	0	1350	150
	HD16-600		HD16-600		HD16-600	
2000	1900	250	1650	100	1400	200
	HD16-400		HD16-400		HD16-400	
2200	2100	300	1850	200	1450	300
	HD16-200		HD16-200		HD16-200	
2400	2350	400	2100	300	1600	400
	HD16-200		HD16-200		HD16-200	
2600	*		2350	400	1800	400
			HD16-200		HD16-200	

TABLE 3 — ZONE 3 EARTHQUAKE

Maximum Height "H"	Soil A "L"	Soil A "K"	Soil B "L"	Soil B "K"	Soil C "L"	Soil C "K"
1200	*		1250	0	1250	0
			HD12-400		HD12-400	
1400	*		1400	0	1250	100
			HD12-400		HD12-400	
1600	*		1450	150	1300	200
			HD12-400		HD12-400	
1800	*		1550	350	1450	300
			HD16-400		HD16-400	
2000	*		1750	400	1600	400
			HD16-200		HD16-200	
2200	*		2000	500	1800	500
			HD16-200		HD16-200	
2400	*		2250	600	2050	600
			HD16-200		HD16-200	

* Wall/foundation detailing considerations fall outside the scope of these standard designs

NOTES

1. Construction to be in accordance with NZS4210.
2. Masonry for walls up to 1.5m high may be Observation type C, otherwise to be Observation Type B.
3. Concrete for foundation to be 25MPa at 28 days
4. Reinforcement is deformed 500E grade.
5. Drainage metal shall be a layer of suitable granular aggregate with perforated pipe to discharge as required by Territorial Authority
6. Compaction forces from machinery are not included in the design
7. Foundation soil properties

		γ kN/m³	ϕ	C' kN/m²
Soil A	Medium Dense Gravel & Sand	19.0	32°	N/A
Soil B	Firm/Med Dense Silt & Sandy Silt	20.0	25°	2
Soil C	Firm Clay & Silty Clay	19.0	17°	25

6.2 Garden Walls

Introduction

Concrete masonry garden walls can provide many useful functions – privacy, separation, protection, ornamentation, shade and shelter from wind.

With a wide variety of concrete masonry shapes sizes and finishes available the design possibilities for walls are virtually unlimited.

Garden walls designed to this section should not form part of either a building structure or an earth retaining structure.

Use of this Document

This publication provides design charts and construction details for two main types of concrete masonry garden walls:

- cantilevered walls; and
- reinforced masonry pilasters supporting infill panels.

Options are provided for either symmetrical spread footings, or for narrower pile footings where there is a requirement to construct close to boundaries or bounding features.

Walls of up to 2.0 m height are considered. Where walls higher than this are required it is recommended that specialist engineering advice be sought.

Structural Design

These concrete masonry walls have been structurally designed in accordance with the appropriate New Zealand Standards and recognised codes of practice. To allow designers and Territorial Authorities to choose and confirm the structural adequacy of designs, options are provided aligning wind and earthquake actions with the *Timber Framed Buildings* Standard NZS 3604. Wind actions in the Extra High wind category are excluded as is Earthquake Zone 4.

Foundation conditions are also aligned to the requirements of NZS 3604.

The strength of masonry is very sensitive to the quality of workmanship. Therefore, every effort must be made to ensure the workmanship is of the highest standard. It is recommended that construction be carried out by a *Licenced Building Practitioner, Bricklaying and Blocklaying.*

Building Consents

The circumstance when a building consent is required is set out in Schedule 1 of the Building Act 2004. That clause explains that a consent is not required for a fence or garden wall provided:

1. It is not a retaining wall.
2. The height does not exceed 2.5 m.
3. It is not a fence as defined in section 2 of the Fencing of Swimming Pools Act 1987.

The height is to be measured from the lowest point of the adjoining ground to the highest level of the fence. If a capping is to be incorporated in the design, then it too should be within the total height of 2.0 m.

If there is any doubt regarding the interpretation of these requirements in relation to a particular proposal, the matter should be discussed with the territorial authority.

Design Charts

Selection of Environmental Design Actions

The principal actions governing the design of garden walls arises from wind or earthquake effects. The wall must be designed to withstand the forces arising from these phenomena.

Both wind and earthquake actions vary throughout the country. Design tables have been produced to enable designers to assess these actions by the methods used in the *Timber Framed Buildings* Standard, NZS 3604. Some territorial authorities have web maps available with NZS 3604 classification Wind speed categories identified for specific areas or on a property by property basis. This will enable rapid identification of the appropriate factor. Extra High and Specific Design categories have been excluded from the design tables and it is recommended professional engineering design advice should be sought in these cases.

Figure 1 (pages 6-7) provides a map of the seismic zones given in NZS 3604. Amendment 10 of compliance document B1/AS1 alters zoning to the Canterbury region which now extends the Zone 2 status to Waimakariri and Selwyn districts including

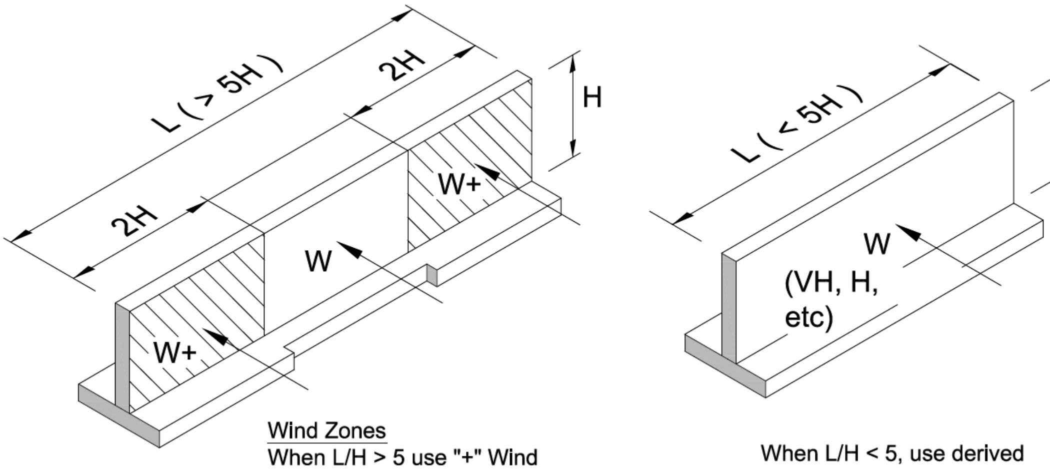
Banks Peninsular. Users of the designs in these areas are advised to contact the appropriate Territorial Authority for final confirmation of the seismic zone for their project.

Users should seek professional engineering advice should they wish to design in the highest earthquake risk Zone 4.

Design for the garden walls has included for the appropriate Importance Level from the *Structural Design Actions* Standard, AS/NZS 1170, this being a lesser risk than that of a habitable structure. The wall designs provided herein should not be used where the wall is intended to form part of a building structure.

Choice of Wall Type

Having identified the Environmental Design Actions, the tables will provide details for either of the wall types, i.e. cantilevered walls, or reinforced pilasters with infill panels. Designers must ***choose the worst case*** of either Wind or Earthquake actions for the particular site, within each table.



Wind Zones
When L/H > 5 use "+" Wind Zones, eg. L+, M+ etc, for 2H End Zone wall and footing selection. Use site derived Wind Zone, eg. L, M etc, for mid section wall and footing selection.

When L/H < 5, use derived site Wind Zone for wall and footing selection throughout

Wind actions on end regions of longer walls can be higher than those on shorter walls and on mid-regions of longer walls. End regions extend for the length equivalent of 2 x wall height, and apply at each end of longer walls. To account for this the tables contain '+' wind zones, e.g. L+, M+, H+ etc., which should be used wherever a continuous wall length is greater than five times its height above ground. On such walls find the wind zone applicable for the site, e.g. H, and use the table entry for the corresponding 'W+' zone, e.g. H+, for the design of the end regions. The mid-region of the wall can either be configured for the derived site wind zone, e.g. H, for economy especially if the wall is long, or left at the 'W+' detailing for ease of interpretation by the builder. These provisions are shown pictorially in the following diagram and an example of applicability is also provided at the end of the commentary.

As well as providing shallow foundation design details for concentric/symmetrical walls, options are now given to build walls close to boundaries, or other obstructions, by the use of bored concrete piles supporting offset wall construction for both wall types. These are show in Figures 3 and 10, for cantilever and pilaster wall types, respectively.

Cantilevered Walls

These are free-standing walls on a symmetrical concrete strip footing. They must be reinforced and be of all cells filled construction.

Reinforced Walls are made of plain concrete masonry blocks, uniformly reinforced vertically and horizontally. In this respect they are similar to a bearing wall or retaining wall.

Refer to Figures 2 to 4 (pages 8-10) for construction details.

Reinforced Masonry Pilasters Supporting Infill Panels

These walls consist of reinforced pilasters supporting masonry infill panels, refer to Figures 5 and 6 (pages 11-12) for layout details. The pilasters may also be used to support other types of lightweight panel such as timber, profiled sheet steel, insulated sandwich panel, etc., the design of which is not covered in this manual.

Three pilaster types, which are shown in Figures 7, 8 and 9 (pages 13-15), have been grouped according to their strength characteristics. Type C, being the strongest, can be used in any situation. Types B and A are of lesser capacity and are restricted to less demanding applications shown in the tables.

The infill panels may be constructed from a wide range of masonry types, or other materials such timber or fibre cement board.

Masonry panels require a minimum reinforcing as indicated and should be all cells filled construction.

Construction Details

Reference should be made to the tables and diagrams appropriate to the particular zone and type of wall.

General

These tables and diagrams provide recommendations for fences and garden walls up to 2.0 m in height. They do not apply to structural bearing walls, nor to retaining walls.

Footings

The base for all footings must be not less than 300 mm below the surrounding ground. The soil must be equivalent to “Good Ground” having an ultimate rupture capacity of no less than 300 kPa as defined in the *Timber Framed Buildings* Standard NZS 3604. Ground should not slope away from the front of the footing more than 10º from the horizontal for a distance equal to two times the height of the wall.

Cantilevered walls, refer Figure 2 (page 8), are constructed on continuous strip footings of minimum depth 200 mm. The footings must be symmetrical about the centre of the wall and the width not less than indicated in the Tables. Figure 3 (page 9), provides details for incorporating screen blocks into the top section of panels.

Alternative offset wall and pile footings are show in Figure 3, and Tables 2 and 3).

For pilaster and infill panel type walls (Figure 5 on page 11 and Figure 10 on page 16), the footing should be sufficient to support the infill panel selected and to act as a tie between pilasters. It should be not less than 200 mm deep and reinforced with two D12 bars which extend into the pilaster footings. The width of the strip footing should be at least 100 mm wider than the blockwork it supports.

Footings for pilasters may be either "pad" type or "bored pile" type. Dimensions and reinforcing details must be in accordance with Figures 5, 6 ,10 and Table 5 (page 12).

All concrete used in footings shall be 25 MPa strength and 80 mm slump. It must be thoroughly compacted and the finish on the upper scabbled, surface suitable for keying the blockwork.

Masonry

Concrete Masonry units shall comply with AS/NZS 4455. Masonry construction shall comply with NZS 4210, and in particular:

(a) Mortar shall consist of one part Portland cement thoroughly mixed with 3-4 parts of building sand.

(b) Grout infill to all cells shall have a spread of 340-530 mm and a strength of 20 MPa.

(c) All joints shall be adequately filled with mortar which shall be compacted by joint tooling after the initial set.

(d) Masonry walls should follow a running bond pattern.

Pilasters and Infill Panels

Pilaster types A, B or C should be selected for the strength requirements indicated in Table 4 on page 12.

Within each type alternative construction details are given. These are minimum requirements, but one of the stronger types can be selected if desired. Thus, where a type A pilaster is recommended any of the details of types A, B or C can be selected.

For a reference type B, the choice is either type B or C, but only type C can be used where this is specified in the Tables. The infill panel between pilasters must have reinforcing as shown in the tables and shall be of all cells filled construction.

Dimensions given for infill panels refer to the length of the panel itself between pilasters.

It should be noted that masonry units are manufactured to a 400 mm module (half block 200 mm) and screen blocks may well be some other dimension.

Reinforcing

All reinforcing shall be deformed bars of the diameters indicated, except for bars designated ‘R’, which are to be plain round.

In cantilevered walls, the vertical reinforcing should be full height in a single length, being located under the horizontal bars of the strip footing and extending into the top bond beam. A D12 horizontal bar shall be located in the top course, with the remaining bars being uniformly distributed through the height of the wall.

Reinforcement for screen blocks shall be hot dip galvanised 2/R4 lattice, e.g. Eagle Wire Products Limited *Bricklock STR 1000/2000,* laid in the fresh mortar in the centre of horizontal joints where indicated in the diagrams or Table 1. Rods shall be continuous through pilasters and lapped as indicated on Figure 4.

Control Joints

Vertical control joints should be provided in the following positions:

(a) in reinforced cantilevered walls the spacing should not exceed 6.0 m.

(b) in pilaster and infill panel walls, a vertical control joint should be provided at the junction of panel and pilaster, at no more than 6.0 m centres.

Control joints should be formed to the provisions of NZS 4210.

Example 1(A): Cantilever Walls

An 11.0 m long by 2.0 m high cantilever wall is to be constructed on a level building site in Hamilton. Site testing in accordance with NZS 3604 has been carried out and it was established that 'good ground' conditions exist at the location of the wall.

By reference to Figure 1 the site is in Zone 1 earthquake zone. By reference to the territorial authority GIS it is determined the site is located in an M medium wind zone. As the wall length is greater than five times its height the design must be for M+ in the end regions (extending for 2 x 2.0 m height = 4.0 m at each end).

With reference to Table 1/Figure 2, (page 8) M+ wind zone requirements are greater than earthquake Zone 1 and therefore take precedence. Thus a 190 thick wall is required, with D12-400 vertical reinforcing and D10-600 horizontal reinforcing (D12 top trim) in end regions. The concentric strip footing is 850 mm wide by 200 mm thick, reinforced with D12-500 both ways, plus SE62 mesh. The central wind zone, 3.0 m in length, could be designed for M wind zone requirements, these being the same as earthquake Zone 1, and would result in a footing width reduction to 650 mm overall width, if desired.

Example 1(B): Piled Cantilever Walls

Alternatively, a piled foundation can be used by reference to Figure 3/Table 3 (wall is in the range 1,500-2,000 high), using a 450 by 350 capping beam, 400 mm diameter piles at 1.5 m centres (because the wall is 190 thick/D12-400 vertical reinforcing) and all to the reinforcing details shown.

Example 1(C): Cantilever Walls, Screen Block Infill Option

Screen blocks can be incorporated into the cantilever wall by following the detailed provisions of Figure 4 (page 10).

Example 2(A): Pilaster Walls

An 18.0 m long by 1.8 m high pilaster wall is to be constructed along a site boundary on a level building site in Napier. The adjacent neighbours have indicated they do not want footings projecting into their property outside the bounds of a normal post footing. Site testing in accordance with NZS 3604 has been carried out and it was established that 'good ground' conditions exist at the location of the wall.

By reference to Figure 1 the site is in Zone 3 earthquake zone. By site-specific analysis to the provisions of Section 5.2 in NZS 3604, or in some areas, by reference to the territorial authority GIS, it is determined the site is located in a Very High wind zone. As the wall length is greater than five times its height the VH+ wind zone must be referenced in the tables for the end regions (but may be carried through the mid-region for consistency of construction and appearance).

With reference to Table 4, Earthquake Zone 3 (Z3-Z1) and VH+ govern this case. Thus, for the 140 mm thick block infill panel option the maximum panel width is 2.2 m for end zones, and it can be supported by Type C pilasters.

For mid-regions of the wall the VH wind zone applies and as Earthquake Zone 3 is also still covered under this lower wind zone the panel width can be increased to 2.6 m.

The foundation for this example where boundary conditions were specified, is a piled foundation Figure 10 (page 16) for 1800 mm high Walls with spacing at 2.2 m in the end zone and 2.6 m centres in the middle section. Pilaster starter bars are required to be cast in the top of the pile/pile cap.

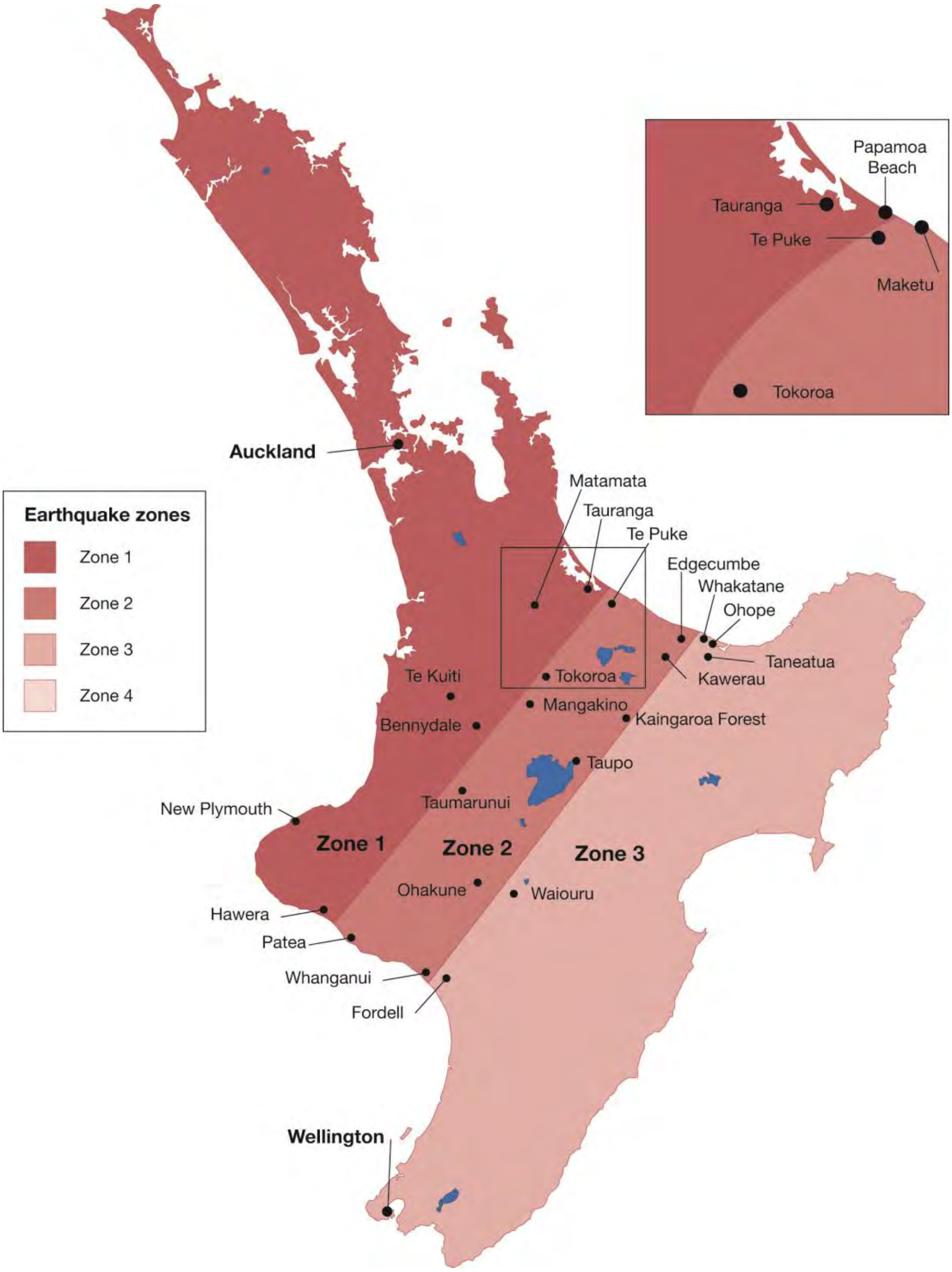
Example 2(B): Pilaster Walls

If the pilaster wall was not built on the boundary then an alternative foundation could be used following the provisions for Type C Pilaster Table 5 with details in Figure 6 (page 12) and connecting footing details in Figure 5 (page 11).

Example 3: Screen Block/Pilaster Walls

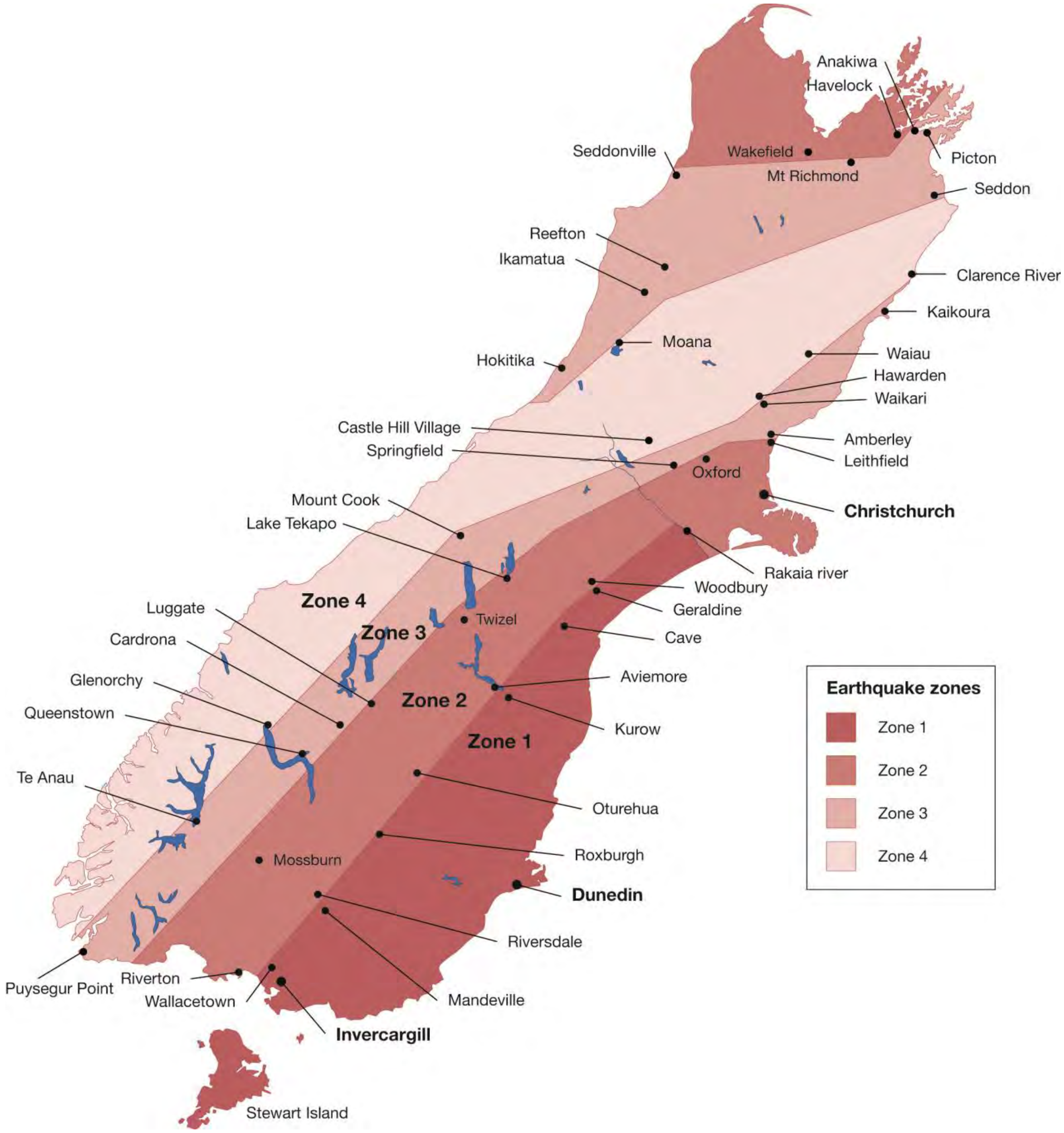
The determination of wind and earthquake requirements follows the previous examples.

For screen block walls the selection is limited to the first row of Table 4 (page 12). Horizontal reinforcement is included in the mortar joint with the Wind/EQ combination determining pilaster spacing from 1.5 m to 2.4 m. Foundations for pilasters can follow either pad footings or piled foundations using A, B or C pilaster details as determined from Table 4.



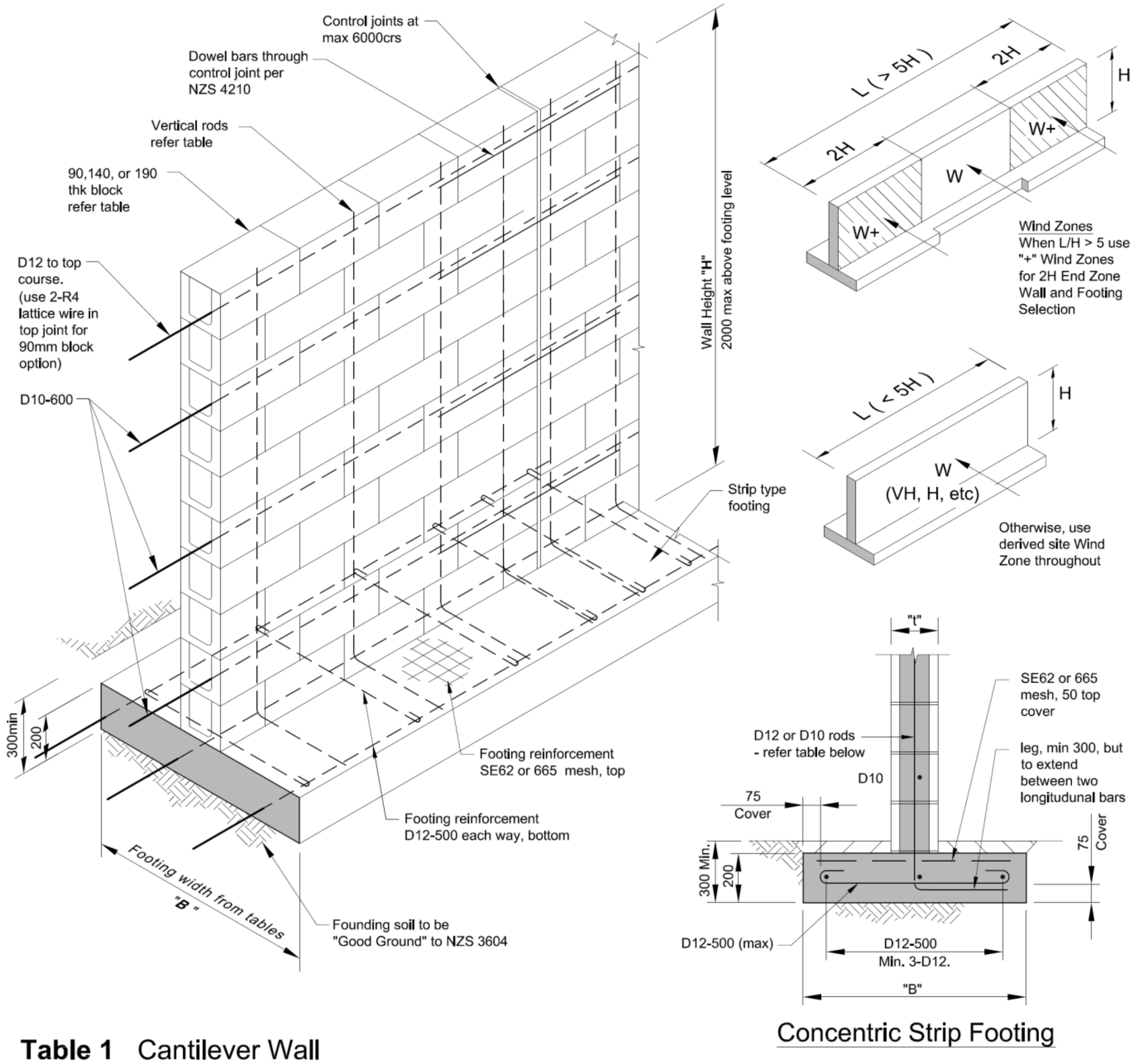
NOTE – This figure is taken from NZS 3604:2011 and incorporates a correction to the zoning of Whakatane.

Figure 1: Earthquake Zones



NOTE – This figure is taken from NZS 3604:2011 and incorporates changes to the Canterbury earthquake region.

Figure 1: Earthquake Zones (continued)



Concentric Strip Footing

Table 1 Cantilever Wall

		Wind: VH+ / H+ E/Q: Z3		Wind: M+ / VH		Wind: H / L+ E/Q: Z2		Wind: M / L E/Q: Z1	
Wall Height (mm) "H"	**Block Thickness** (mm) "t"	**Footing Width** (mm) "B"	**Wall Vertical Reinforcing**	**Footing Width** (mm) "B"	**Wall Vertical Reinforcing**	**Footing Width** (mm) "B"	**Wall Vertical Reinforcing**	**Footing Width** (mm) "B"	**Wall Vertical Reinforcing**
600	90	450	D10-600	400	D10-600	400	D10-600	400	D10-600
800	90	600	D10-600	600	D10-600	600	D10-600	600	D10-600
1000	140	700	D12-600	600	D12-600	600	D12-600	600	D12-600
1200	140	850	D12-600	650	D12-600	650	D12-600	650	D12-600
1400	140	1000	D12-400	650	D12-600	650	D12-600	650	D12-600
1600	190	1050	D12-400	700	D12-400	700	D12-400	650	D12-400
1800	190	1200	D12-400	750	D12-400	750	D12-400	650	D12-400
2000	190	1400	D12-400	850	D12-400	850	D12-400	650	D12-400

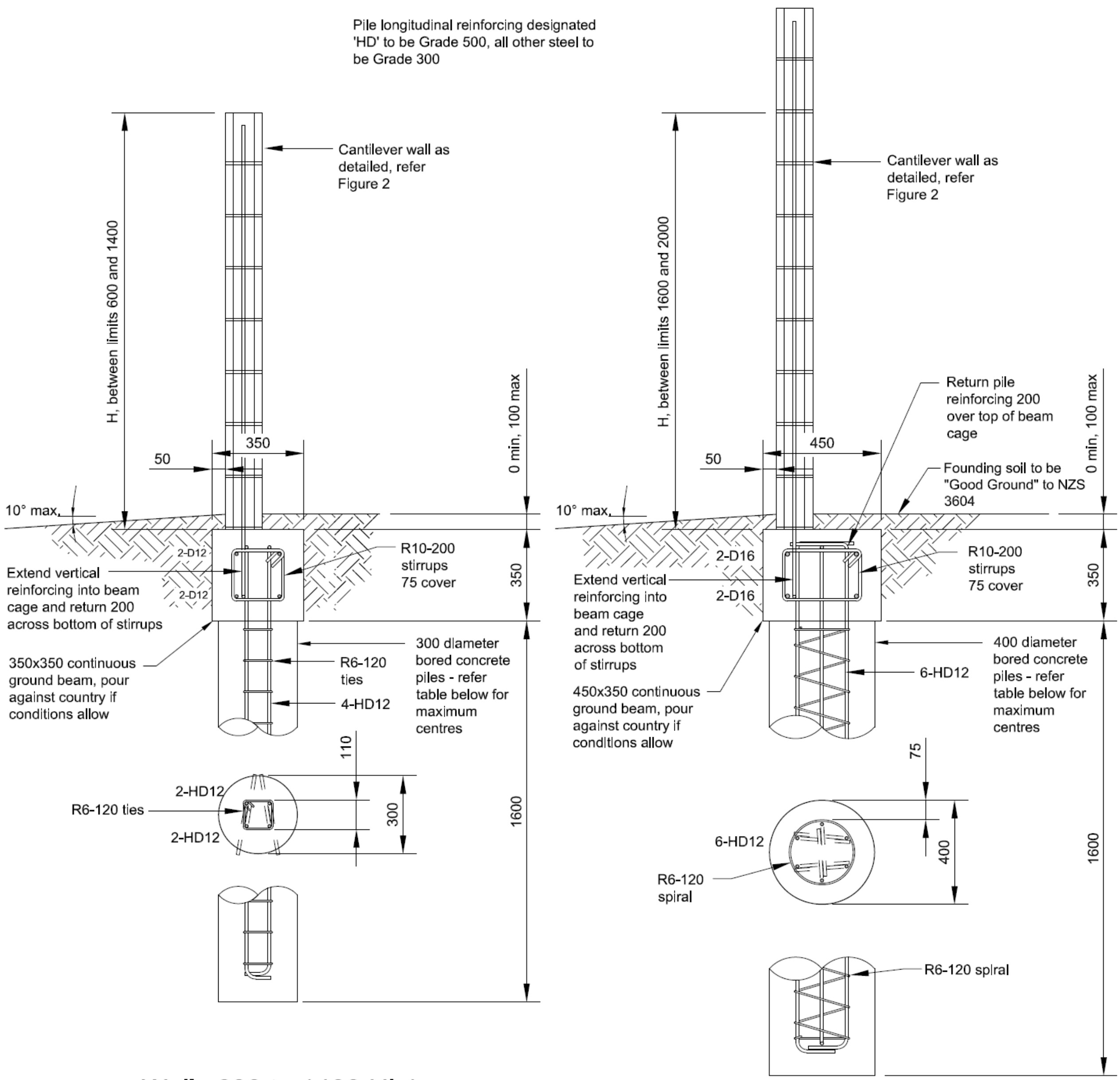
NOTE: Select the greater requirement of either Wind or Earthquake

Figure 2: Cantilever Wall

Note:

Ground conditions to meet minimum 'Good Ground' criteria per NZS 3604

Pile longitudinal reinforcing designated 'HD' to be Grade 500, all other steel to be Grade 300



Walls 600 to 1400 High

Walls 1500 to 2000 High

Table 2 300Ø Pile Spacing

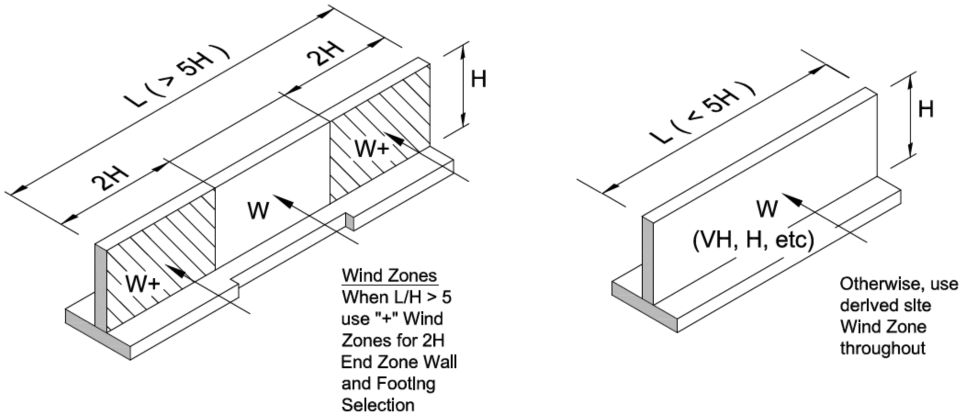
300 Diameter Piles	
Wall / Vertical Reinforcing (from Fig. 2)	**Pile Spacing** (m) **"S"**
90 - D10-800	3.2
90/140 - D10-600	2.5
140 - D12-600	1.5

Table 3 400Ø Pile Spacing

400 Diameter Piles	
Wall / Vertical Reinforcing (from Fig. 2)	**Pile Spacing** (m) **"S"**
140 - D12-600	2.2
190 - D12-400	1.5

Note: Wind and earthquake zone definitions and "Good Ground" foundation soil conditions, are to be assessed in accordance with NZS 3604:2011, Timber Framed Buildings Standard.

Figure 3: Piled Cantilever Wall Details



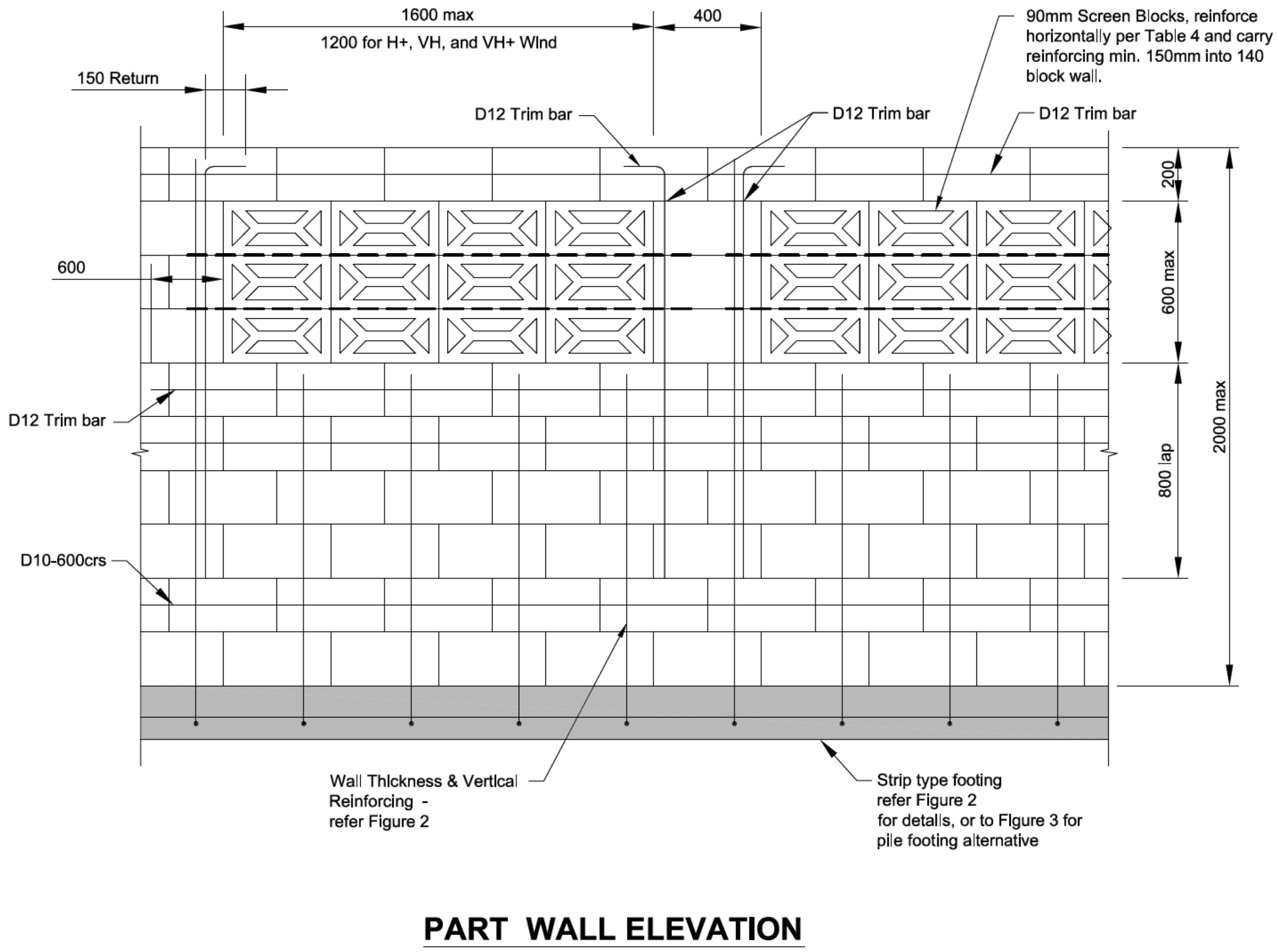


Figure 4: Cantilever Wall, Screen Block Infill Option

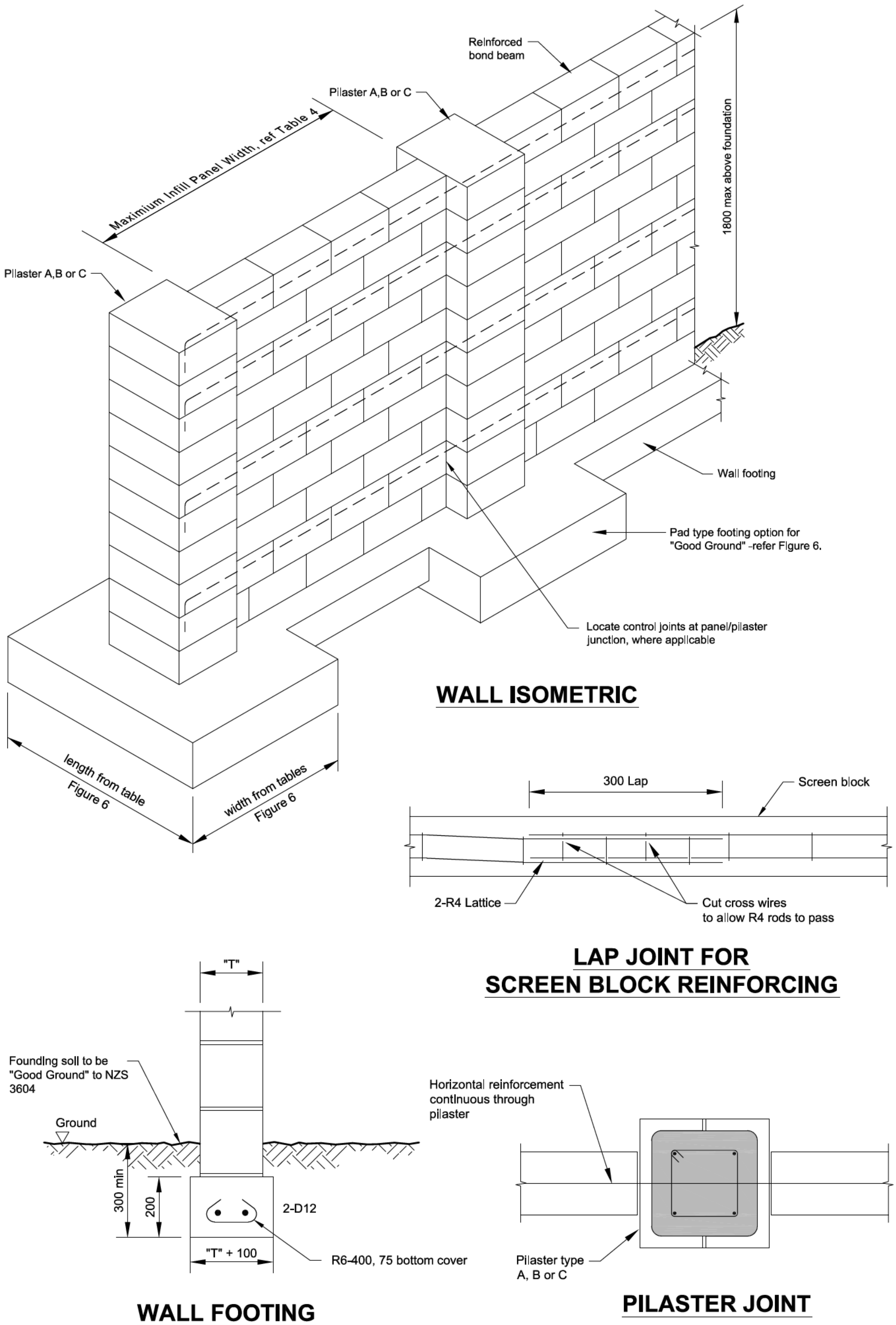


Figure 5: Infill Panel Wall

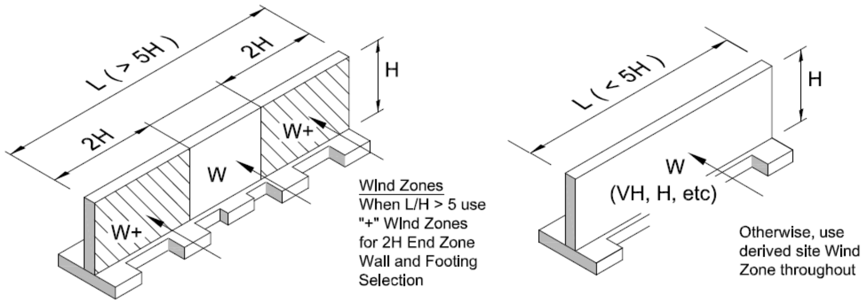


Table 4 Pilaster Wall Details (1.8m High, Maximum)

Infill Type	Block Thickness (mm)	Wind / E/Q Zone		Wind / E/Q Zone		Wind / E/Q Zone		Wind / E/Q Zone	
Horizontal Reinforcing		Max. Panel Width	Pilaster Type	Max. Panel Width	Pilaster Type	Max. Panel Width	Pilaster Type	Max. Panel Width	Pilaster Type
Screen	90	VH+ / Z3 - Z1		H+ / Z3 - Z1		VH / H / M+ / Z3 - Z1		M / L+ / L / Z3 - Z1	
2/R4 Galv. - 400		1.5	B/C	1.5	A/B/C	2.1	A/B/C	2.4	A/B/C
90 Block	90	VH+ / H+ / Z3 - Z1		VH / M+ / Z3 - Z1		H / L+ / Z3 - Z1		M / L / Z2 & Z1	
D10 - 600		2.0	C	2.6	B/C	3.0	B/C	3.6	A/B/C
140 Block	140	VH+ / H+ / Z3 - Z1		VH / M+ / Z3-Z1		H / L+ / Z2 & Z1		M / L / Z1	
D10 - 600		2.2	C	2.6	C	3.2	B/C	3.8	B/C
190 Block	190	VH+ / Z3 - Z1		H+ / Z2 & Z1		VH / M+ / Z2 & Z1		H / L+ / M / L / Z1	
D12 - 600		2.8	C	3.6	C	4.0	C	4.6	C
240 Block	240	Z3 - Z1		VH+ / Z2 & Z1		H+ / M+ / Z2 & Z1		VH / H / L+ / M / L / Z1	
D12 - 600		2.4	C	2.8	C	3.8	C	4.6	C

For any Infill type choose the lower maximum panel width given for the Wind and Earthquake zones applicable for the chosen site.
Refer to Figures 7 to 9 for definitions and construction details for the various pilaster types.
2/R4 in the table refers to double R4 rod lattice as per Eagle Wire Products *Bricklock STR 1000/2000 Lattice*, or equivalent.
Refer to Figure 5 and details below for pad footing schedule and details, or otherwise to Figure 10 for piled foundation details.

1.2m High Pilaster Wall Option

Seclect panel type and maximum width from table above, according to wind and earthquake requirements. Pilaster and foundation may be Type A for all Wind and Earthquake zones.

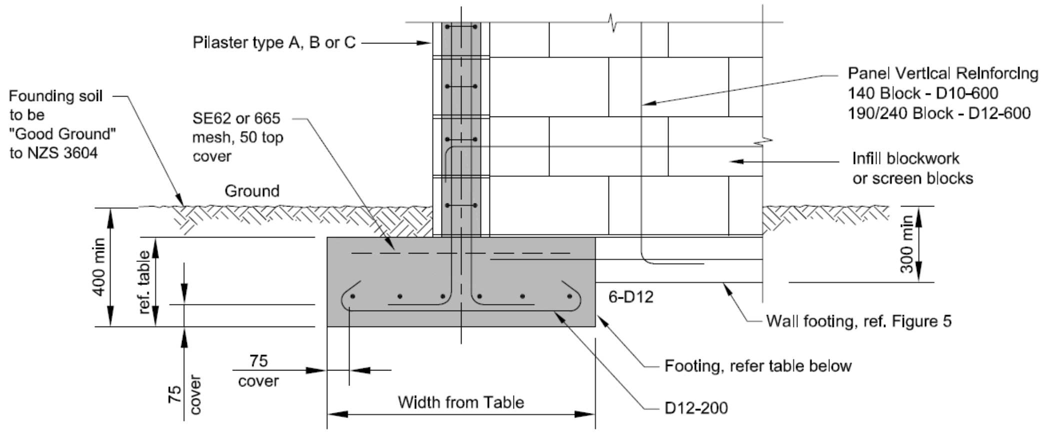
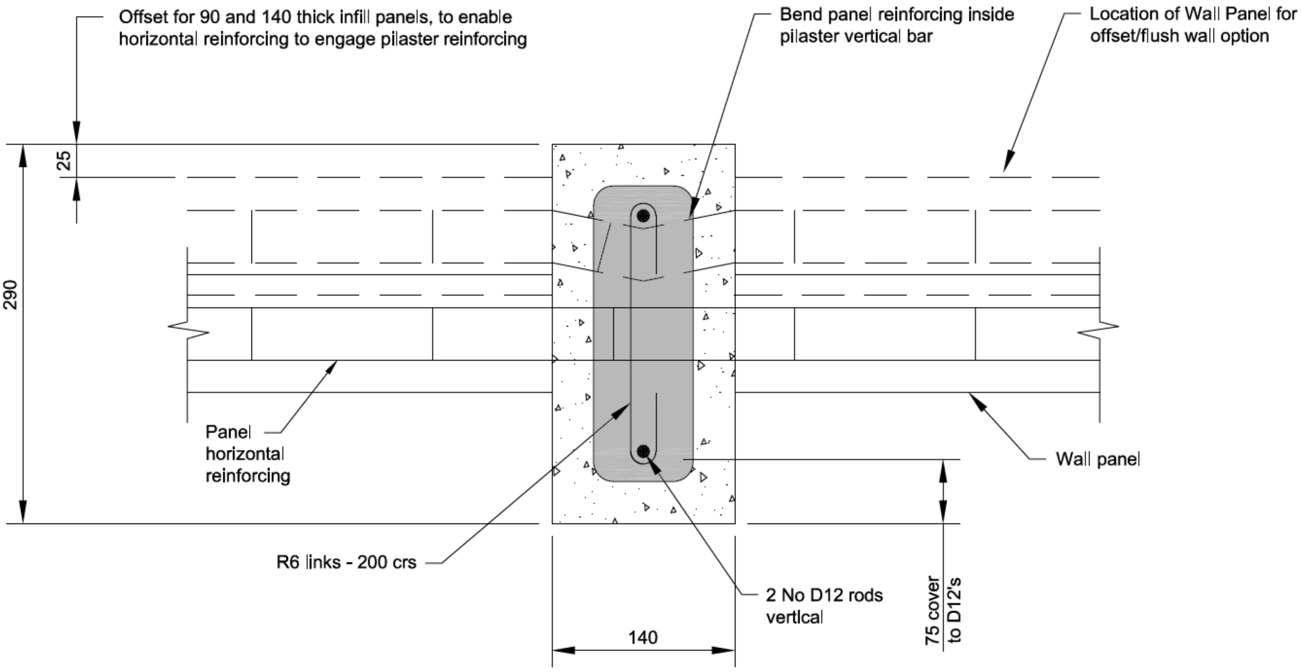


Table 5 Pilaster Wall Foundation Details

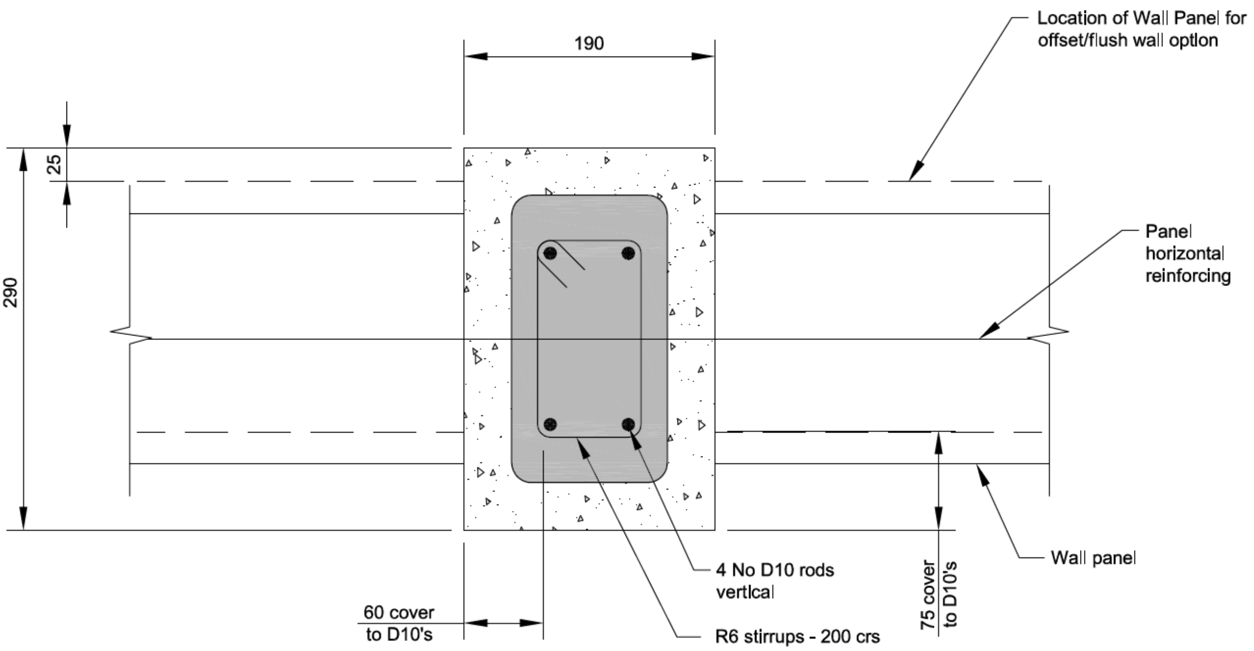
Pilaster Type	Dimensions mm			D12 Reinforcing	
	Length (across wall)	Width (along wall)	Thickness	Length Direction (normal to wall)	Width Direction (parallel with wall)
A	1200	1000	300	6 No	6 No
B	1400	1000	300	6 No	7 No
C	2000	1000	350	6 No	10 No

Figure 6: Pad Type Footing

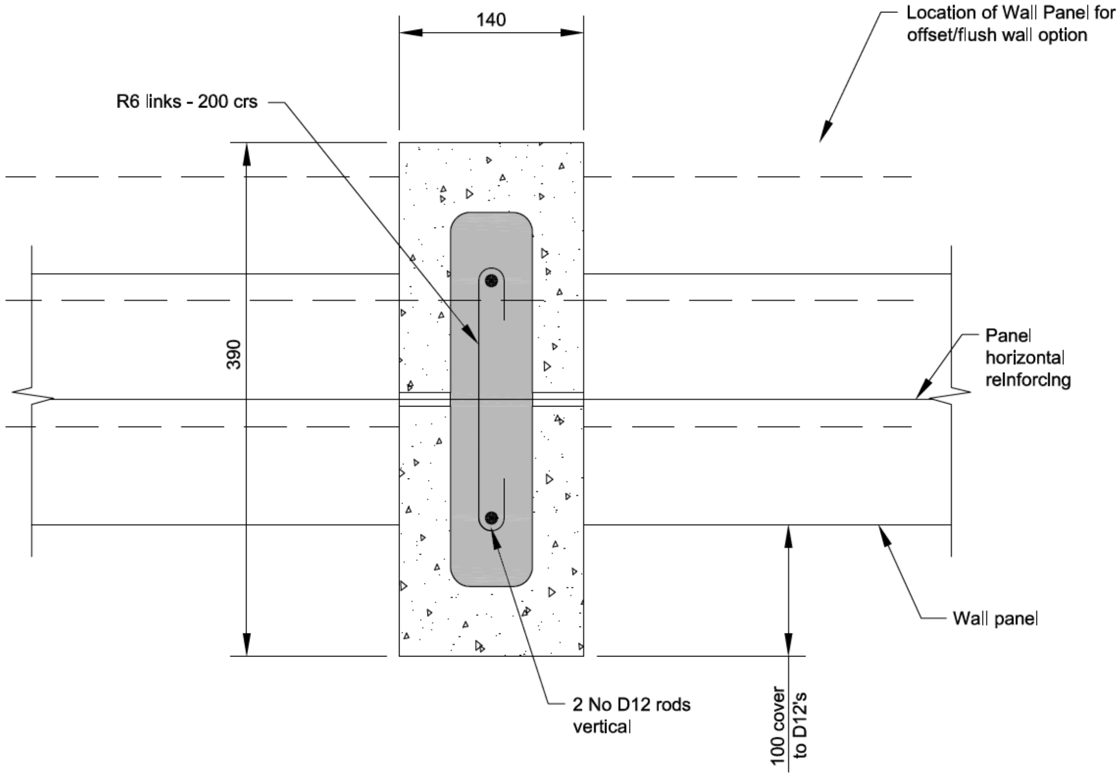


15.19 THREE QUARTER BLOCK

Figure 7: Pilaster Type 'A' Details

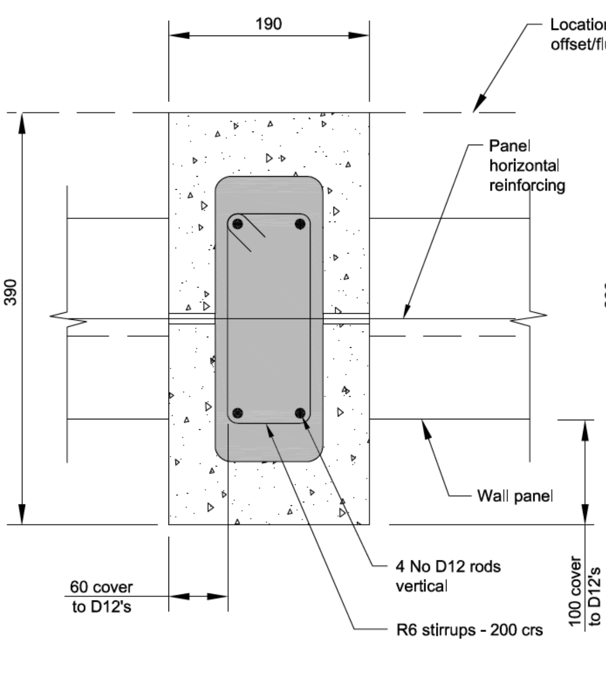


20.19 THREE QUARTER BLOCK

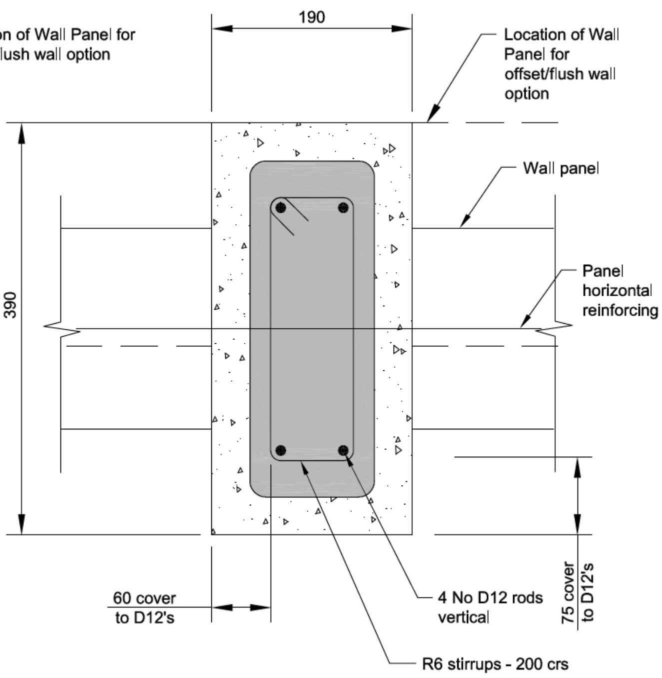


2 x 15.12 STANDARD LINTEL BLOCK

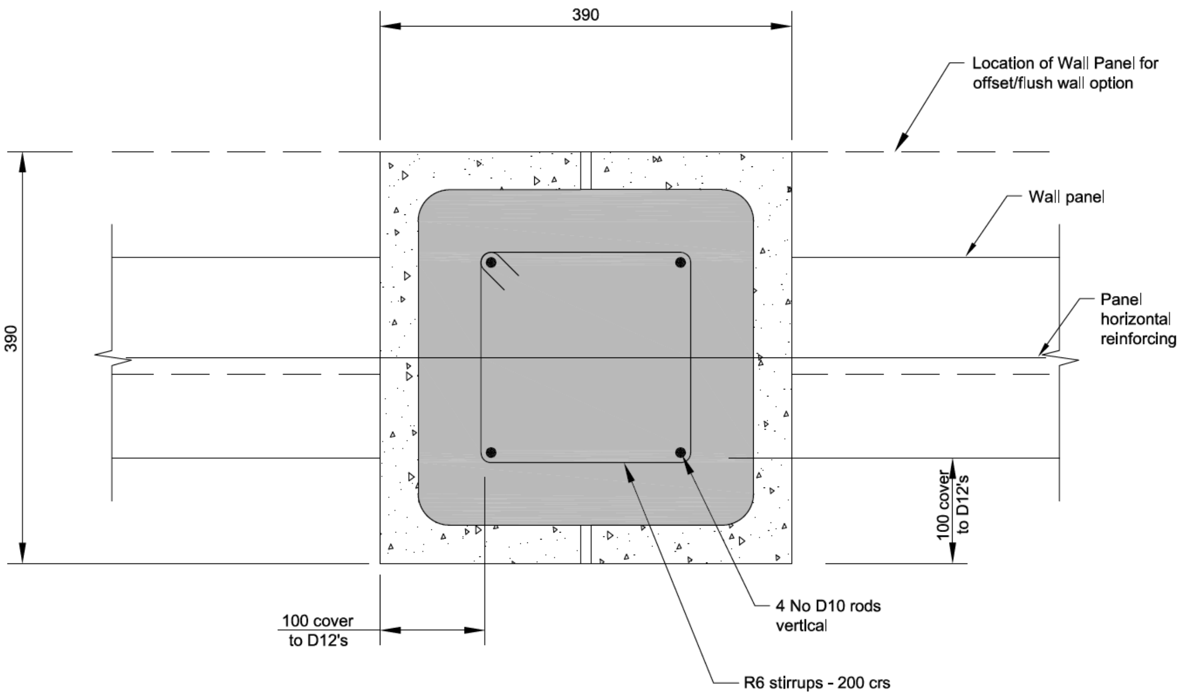
Figure 8: Pilaster Type 'B' Details



2 x 20.12 STANDARD LINTEL BLOCKS

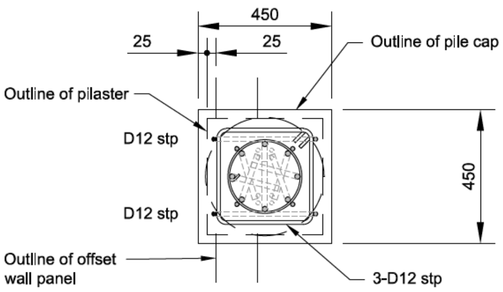


20.30 PIER BLOCK

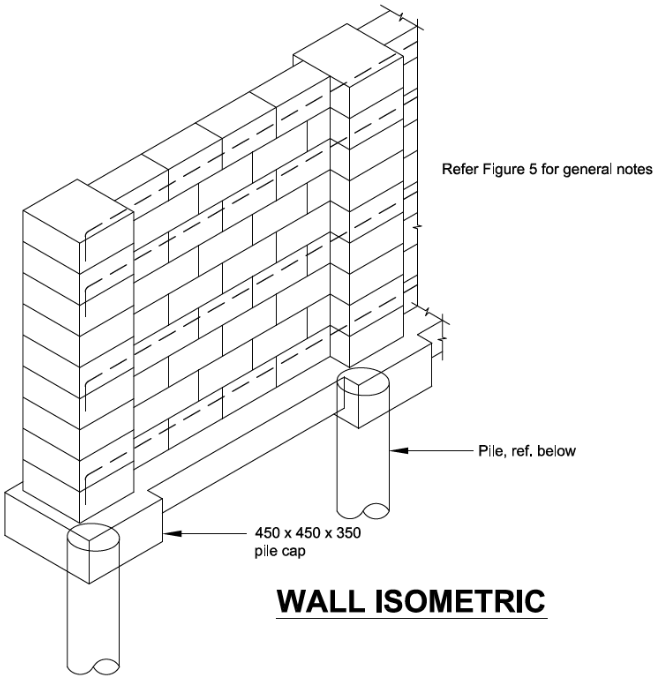


20.34 PILASTER C TYPE (ALSO KNOWN AS 20.36A)

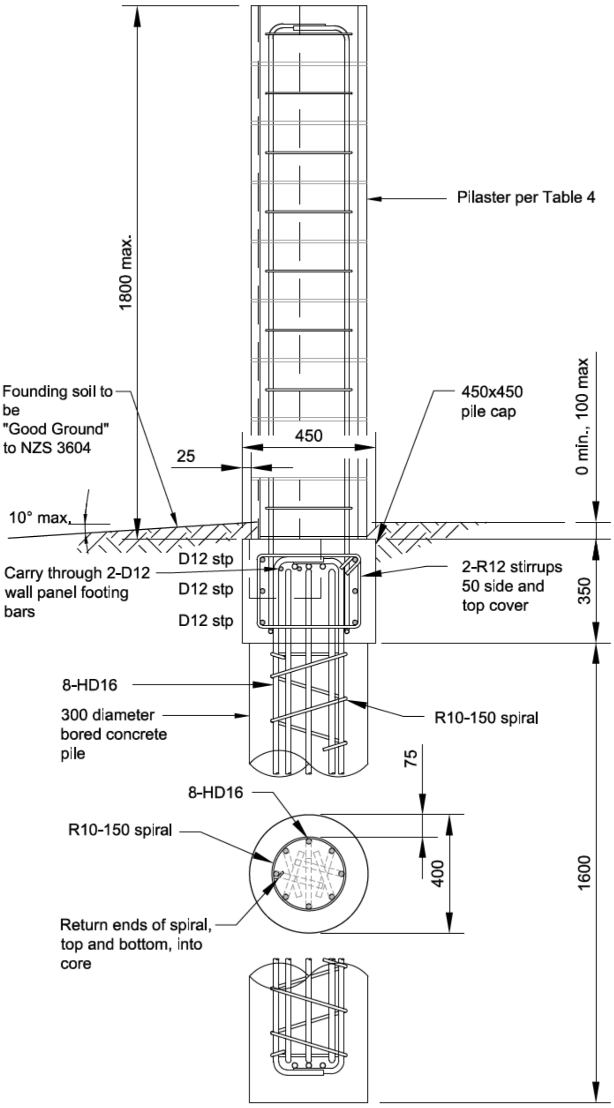
Figure 9: Pilaster Type 'C' Details



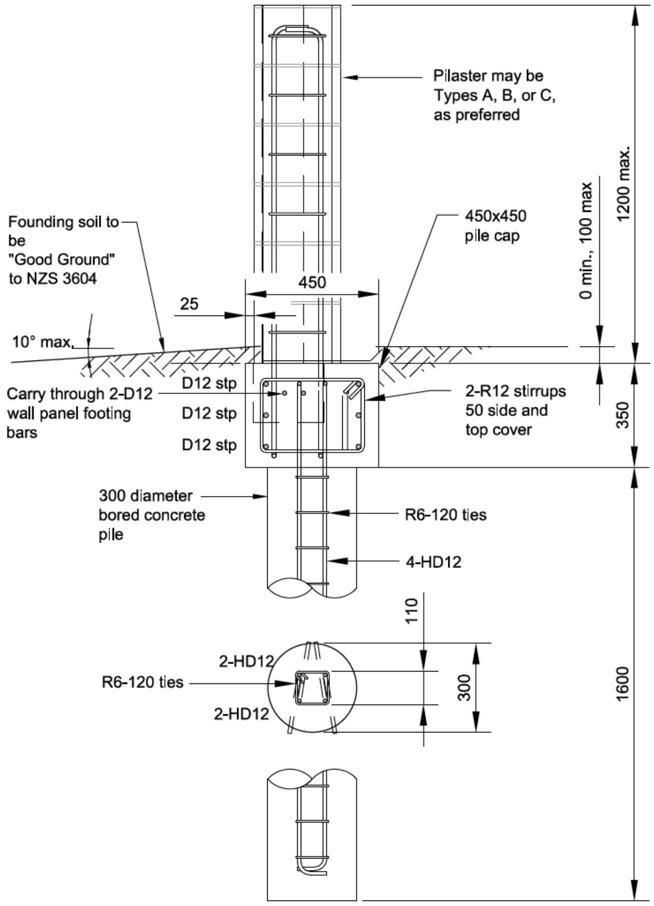
Plan on Pile Cap



WALL ISOMETRIC



Pile Foundation for Max.
1.8m High Pilaster Wall



Pile Foundation for Max.
1.2m High Pilaster Wall

Figure 10: Piled Pilaster Wall Details

6.3 Control Joints

Introduction

Concrete masonry walls, whether they be walls forming part of a building structure, retaining or garden walls, often require control joints to be built in to counter the effects of drying shrinkage and other movement effects not related to their primary structural purpose. The requirements for these are spelt out in most design and construction standards dealing with concrete masonry and that are referenced by the New Zealand Building Code. There are, however, differences in the presentation of control joints between these standards, and in some cases a lack of rationale in specific provisions.

This section is written with the aim of clarifying for both designers and tradespeople the reasons for providing control joints, at what locations they are needed, and details for their construction.

This section applies to reinforced concrete masonry but excludes masonry veneer construction. Refer to Section 5.1 for veneer construction requirements.

Regulations

Concrete masonry construction is covered under a number of standards cited by the New Zealand Building Code. These are as follows:

Acceptable Solutions

Clause B1/AS1:

- NZS 4229 *Concrete masonry buildings not requiring specific design*
- NZS 3604 *Timber-framed buildings* (specifically, masonry foundation walls)

Verification Methods

Clause B1/VM1:

- NZS 4230 *Design of reinforced concrete masonry structures*

Additionally, all the above standards reference the standard for masonry construction -

- NZS 4210 *Masonry Construction – Materials and Workmanship*

Where there is any conflict between linked standards these would normally be picked up and clarified within the Building Code. Where conflict has not been identified, by inference the provisions contained within the acceptable solutions and verification methods would take priority over those of subordinate standards, in this case NZS 4210. Any advice contained within this section of the Masonry Manual which does not follow the Building Code needs to be considered as an Alternative Solution, and be presented as such where approval for building consent is being sought.

An organizational chart of how the regulations are related is provided in Figure 1 on page 2.

At the time of writing NZS 4210:2001 precedes updates to all the other standards referenced above. It should be noted that NZS 3604 makes no specific provision for control joints and so the requirements of NZS 4210 (which is referenced by NZS 3604) should apply. NZS 4229 also references NZS 4210 however as it has its own specific requirement for control joints (refer *Section 12, Shrinkage*) they will take precedence over NZS 4210. Similarly, NZS 4230 references NZS 4210 but has its own control joint requirements, which should take precedence where applicable.

Reasons for Control Joints

Cracking in blockwork structures can be due to a combination of actions which may include drying and carbonation shrinkage, temperature fluctuations, moisture content, creep, and building and foundation movement. Cracking will only occur when the blockwork is **restrained** against such movement effects. Most blockwork structures contain rigid restraints and therefore require consideration for crack control.

Some consequences of uncontrolled cracking include:

- Loss of durability, potential for accelerated corrosion of reinforcing steel
- Loss of weather-tightness
- Unsightliness

Crack widths less than 0.5 mm in width are generally considered to be of cosmetic concern only. Applied finishes can usually bridge that order of width.

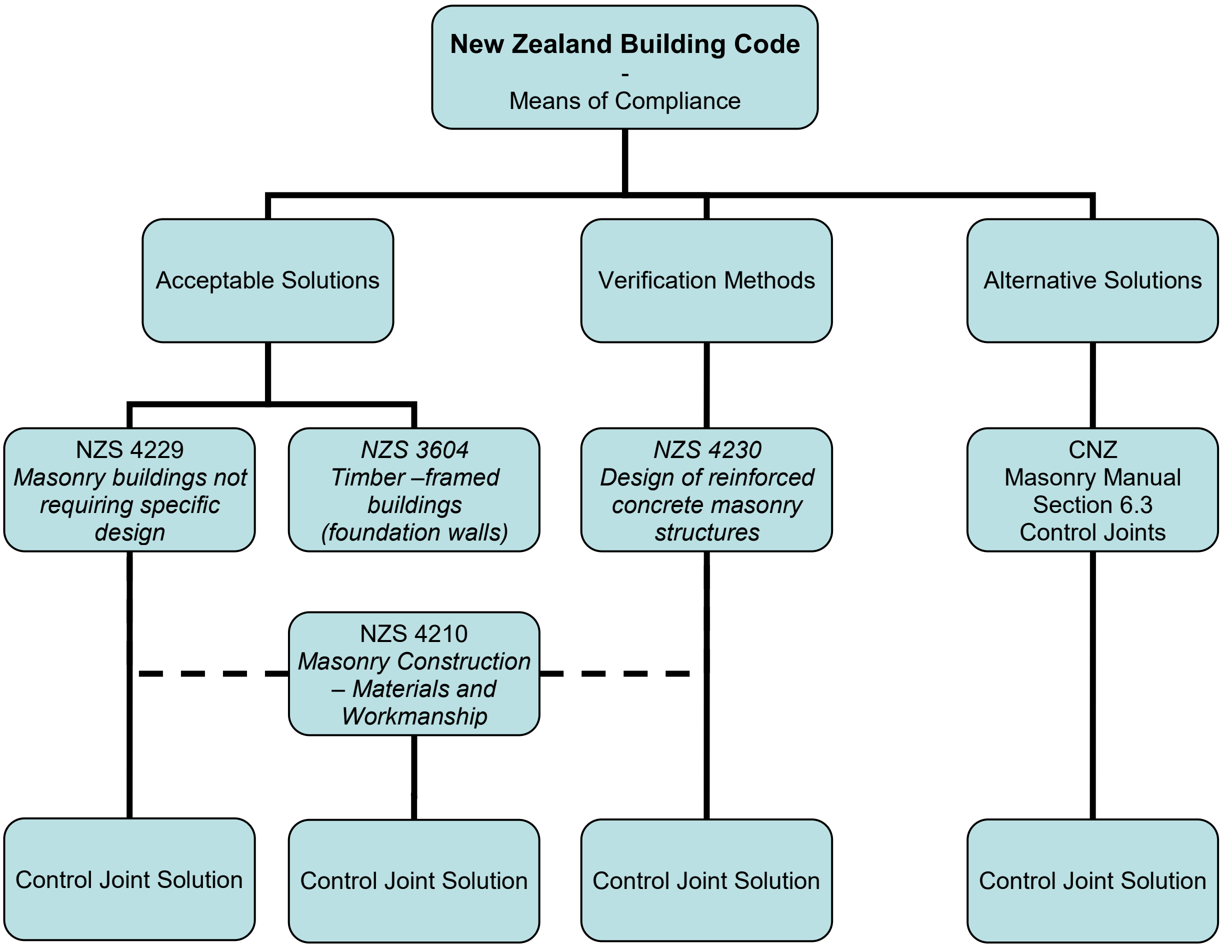


Figure 1: Regulatory Context for Masonry Buildings

Vertical action effects that might lead to cracking in blockwork are rare, principally because most structures are not restrained in that direction and the blockwork is therefore free to move. Where blockwork is in the form of infilling to other primary structure, say inside a rigid concrete or steel frame potentially providing vertical restraint, vertical crack control may need to be considered.

Horizontal movement in blockwork, on the other hand, is more likely to result in cracking because there is usually some form of restraint provided to resist such movement. This is often in the form of foundations and floors (including stairs), and to a lesser extent, connected walls. Cracking also tends to concentrate at abrupt reductions in horizontal strength of blockwork such as those occurring at changes of thickness, height, and around opening penetrations.

Horizontal reinforcing steel contents that are typically provided in block walls are not sufficient to evenly distribute movement strains so that cracks are spread evenly and consequently their widths minimized. Most concrete masonry standards therefore provide for appropriately positioned and detailed **control joints** to provide fuse points for horizontal movement strains to concentrate at. As an alternative to these, by significantly increasing the horizontal reinforcing steel contents cracks are more evenly distributed and the cracks are individually of narrower width. Control joint spacing can usually be increased in this instance. This is a matter for specific engineering design and judgement and is not discussed further in this section.

Drying Shrinkage

Constituent blockwork components comprising hollow block, mortar, and grout infilling all lose free water on setting and curing until equilibrium, or ambient condition, moisture content is reached. This loss of water is accompanied by a loss of overall volume in the constituents and results in irreversible drying shrinkage of the blockwork as a whole. The process is time-, material-, and environment-dependent. Typical values for linear drying shrinkage strains of masonry constructed from lightweight pumice aggregate masonry units in tests carried out by the University of Auckland[2] are as follows:

- Un-grouted masonry – 0.04%, or 0.4 mm/m length
- Grouted masonry – 0.07%, or 0.7 mm/m length, e.g. 3.5 mm in 5 m length

Carbonation

Concrete blockwork also undergoes irreversible shrinkage due to long term effects of carbonation, which is the result of a reaction between cementitious materials and carbon dioxide in the atmosphere. A suggested allowance for US concrete masonry[5] which might reasonably be considered applicable for New Zealand blockwork pending advice of more accurate data is in the range 0.02% to 0.045%, or between 0.2 mm/m length and 0.45 mm/m length.

Temperature

Temperature effects on blockwork can be significant, especially in climates where extremes of temperature are experienced, or where masonry is dark in colour and has a greater propensity to absorb solar radiation. The induced movements fluctuate and are largely reversible. Based on an assumed range of coefficient of linear expansion of between 5×10^{-6} / ºC and 12×10^{-6} / ºC, a temperature variation of, say, 50ºC could translate to between 0.25 mm/m and 0.6 mm/m movement range in un-restrained blockwork.

Moisture Fluctuations

Changes in moisture content of the blockwork may occur where no surface protective coatings have been provided and the blockwork is subject to the fluctuations in atmospheric moisture, and in some cases groundwater. Specific data quantifying likely effects of moisture on blockwork are difficult to source, but changes in dimension should be significantly less than that for drying shrinkage.

Building and Foundation Movement

The prime function of blockwork in buildings and structures is to resist building forces that may be imparted by gravity, wind, seismic, and other action effects. Unintended, or incidental, movements can occur due to unidentified elastic deflection in support structure, differential foundation soils consolidation, and time-dependent creep in primary structure. Resulting stresses in the blockwork can often be relieved at locations where control joints are provided for shrinkage and temperature relief.

Specialised Joints

Specialised joints are not provided for in structures considered within the scope of NZS 3604 and NZS 4229. They may occur in structures specifically designed to NZS 4230 but are not considered in detail in the section. A brief explanation of specialized joint types is provided in the following.

- **Expansion joints** might be required in particularly long, or specialized, structures where the likes of thermal fluctuation movements can be significant. Note that relatively small expansions resulting from the thermal, moisture, and movement effects in non-specific design structures can usually be accommodated by normal control joints.
- **Contraction joints** might be provided in larger specific design structures for reasons similar to expansion joints, principally thermal fluctuation effects. Control joints considered in this section do provide for limited building contraction in non-specific design structures.
- **Separation joints** are sometimes provided to provide a break in the transfer of actions for earthquake-resisting structures, or for other specialised reasons.

Provisions of Current Standards

On review of provisions for control joints in current materials and design standards several matters requiring consideration and further comment were identified. Briefly, these are as follows:

NZS 4210:2001 Masonry Construction, Materials and Workmanship

1. Clause 2.10.1.2(c): **Control joint centres** for foundation wall blockwork in **NZS 3604** structures are allowed at up to 24 m centres for low-height blockwork. The practical experience of the performance of semi buried foundation walls, by a Government Agency, was reflected in the decision to allow wider spacing between control joints than for fully exposed walls. This view is also similar to the views of NCMA (USA)who consider shrinkage to be of a lower overall value than in the exposed superstructure, with the foundation reinforcement able to satisfactorily to control the reduced shrinkage movement. Where users consider that foundation walls are subject to significantly greater exposure, then control joint centres should follow the practices of NZS 4229 and 4230.

2. Clauses 2.10.2(a) and 10.4.2: The **control joint default width of 10mm** does not fully provide for the extension capabilities of sealants that typically require sealant aspect ratios of 2:1 and width of bonded surface (sides only) of 8mm. A modification to existing detailing (refer Figure 4) or increasing the joint width by cutting the face of the block to the depth of the joint, say 10mm, would achieve a more resilient outcome.

3. Clause 2.10.5: These clauses apply to all referenced standards and require horizontal **bond beam and lintel reinforcement to be continuous** through control joints, with other horizontal reinforcement to be discontinuous using de-bonded dowels. From a structural continuity perspective this makes sense however the restraint provided by the combined effect of foundation and bond beam reinforcing will restrict overall movement and the effectiveness of the joint. It appears the provision does work in practice, however, and it is a structural compromise that must be accepted.

4. In Figure 2 of NZS 4210:

   - R16 dowels shown should coincide with horizontal bars provided
   - Dowel bars should be saw-cut at de-bonded ends, and should not be sheared as this tends to deform the bar and create undesirable bond
   - Dowels should be contact lapped with horizontal reinforcing and not be offset as shown (it is very difficult to accurately place and retain bars offset as shown)
   - The use of grease to de-bond bars is not in common usage. Polythene tape is the preferred method as it doesn't run the risk of inadvertently compromising other reinforcing, is more resilient, and is more readily identified and checked.

NZS 4229:2013 Concrete masonry buildings not requiring specific engineering design

5. **Control joint locations** in walls are purposely located away from door and window locations to allow the structural trim reinforcing to achieve its aim of transferring internal actions around the opening to adjacent structure, either side.

6. Clause 12.1.1(a), referring to masonry buildings having Tee and L shaped floor plans, may be difficult for users to interpret in the absence of diagrammatic representation. This is addressed in Figure 2 of this section.

NZS 4230:2004 Design of Reinforced Concrete Masonry Structures

This standard shows, in Figure 3.1, an alternative crack control joint having continuous reinforcing through the joint, de-bonded over a width of 300mm, and centred on the joint. There is little commentary on its usage other than that it has apparently performed well in in practice. A potential problem with this detail is that the strains induced in the reinforcing steel in the vicinity of the joint are likely to cause the reinforcing steel to yield, and this could have undesirable structural consequences in certain circumstances. It should only be used where designers are confident that blockwork movement effects on the reinforcing are not critical to structural performance. It will also only be properly effective where the bond break tape has sufficient bulk and deformability to prevent binding of reinforcing bar deformations, for example using grease-impregnated tape such as Denso.

Joint Locations

The table on page 5 summarises the wall control joint location provisions of the four standards considered. For context and specific application users are advised to refer to the relevant standard.

		NZS 4210 Masonry Construction Standard	**NZS 3604** (via NZS 4210) Foundation Walls	**NZS 4229** Non-Specific Design Structures	**NZS 4230** Specific Design Structures[3]
1	General Spacing	Per Design Standard over.	Walls < 2.0m – 8.0 m < 1.2m – 12.0 m[1] < 0.8m – 24.0 m[1]	5.0 m recommended, 6.0 m max	1.5H, 7.6 m max.
2	T and U-shaped floor structures	Per Design Standard over.	Per NZS 4229	Within 0.6 m of return angles [2]	To align with movement joints in floors and roofs
3	L shaped corners	Per Design Standard over.	Per NZS 4229	Within 0.6 m of junction, or 3.2 m from corner in both directions	Within 50% of general joint spacing, from the junction; in all walls connecting at the junction
4	Changes in Wall Height	Per Design Standard over.	Per NZS 4229	Yes, where the difference is > 0.6 m	Yes
5	Changes in Wall Thickness	Per Design Standard over.	Per NZS 4229	Yes	Yes (e.g. vertical duct chases, pilasters etc.)
6	At Door and Window Openings	Per Design Standard over.	No	No	Yes/No (both options can be designed for)
7	Movement Joints in Floors and Foundations	Per Design Standard over.	Per NZS 4229	N/A	Yes

1. *Explanation for these figures is contained in Review 1 of NZS 4210, pending at the time of publication of this document.*
2. *Refer to Figure 2 below for interpretation.*
3. *Provisions shown for NZS 4230 assume a minimum horizontal reinforcing content of 0.03%, which is very low. Through specific engineering design and using considerably higher reinforcing contents control joints can be spaced further apart or removed altogether.*

Figure 2 below provides an example structure and the interpretation of control joint locations to NZS 4229, using a T-shaped floor plan. The provisions may also largely apply for structures constructed to NZS 3604 and, for practical applications, to many structures specifically designed to NZS 4230.

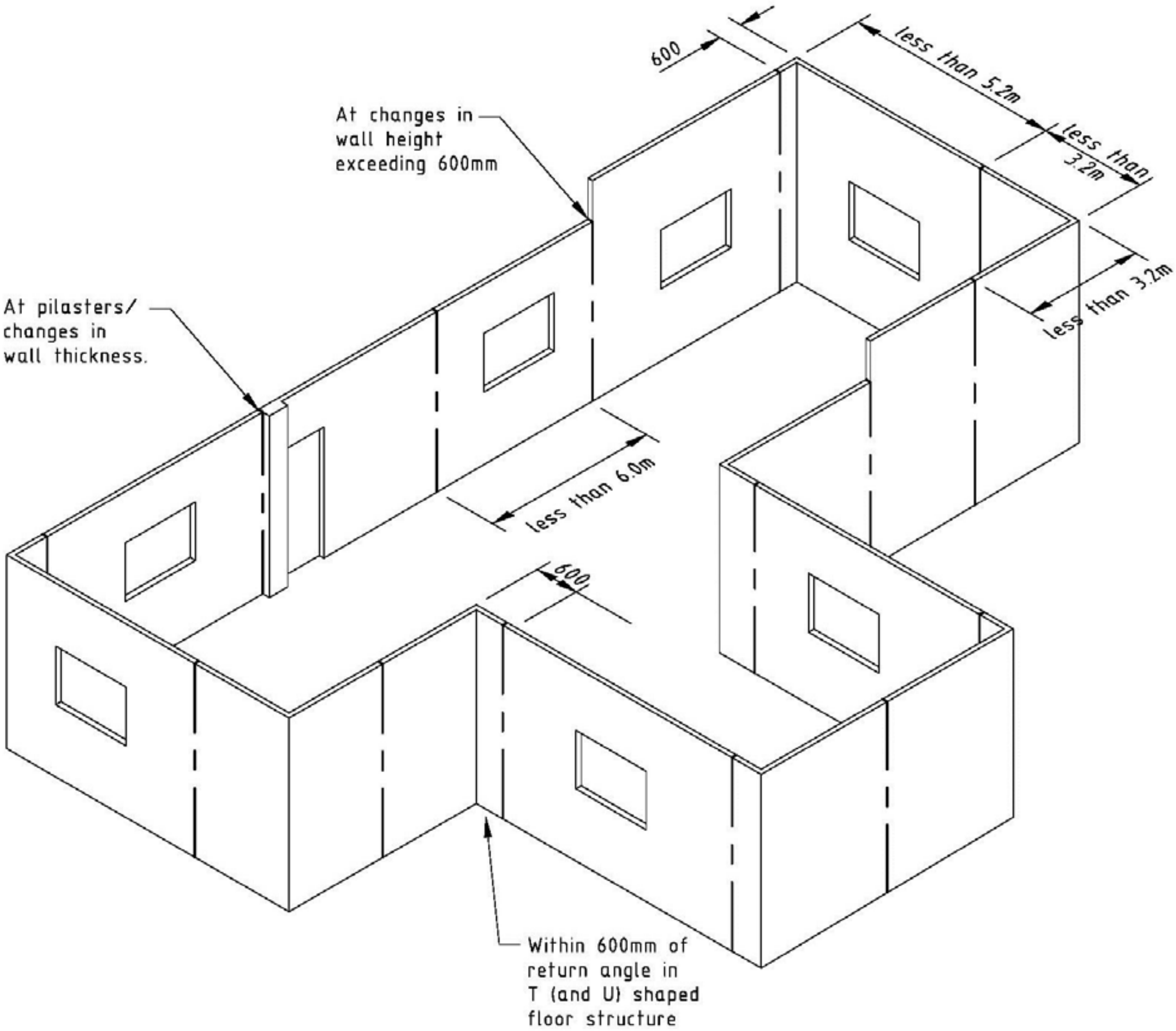


Figure 2 - Control joint locations (NZS4229)

Joint Detailing

Some of the typical details provided in the standards are perceived to contain shortcomings. These are explained in this section. Figures 3 (applicable for NZS 4229), 4, and 5 following this section have been produced to provide more clarity and consistency in interpretation.

Dowels

Dowels are typically provided through joints to transfer shear forces both in-plane and out of plane. For structures designed to NZS 3604 (NZS 4210) and NZS 4229 the forces involved are likely to be nominal, the action of the dowels will principally be restraining the walls to provide alignment both sides of the joint, and will also be contributing to bridging against differential foundation settlement. It is the intention of both standards that R16 dowels are provided to align with specified horizontal wall (not bond beam) reinforcement.

NZS 4229 requires lintel and bond beam reinforcing to be continuous through control joints, i.e. do not use dowels at lintels and bond beams. For NZS 3604 structures the reinforcing in foundation beams supporting foundation walls will also be continuous through joints. These factors may lead to reduced movements at wall joint locations.
Both non-specific design standards require vertical reinforcing to be provided in the first block cell either side of the joint. This will assist in tying the dowel bars in their required horizontal alignment however the likelihood of the bars being installed true, or remaining perfectly horizontal through the construction process when not in direct contact with the horizontal reinforcing, is questionable. Securely tying the dowel bars to the horizontal reinforcing is a more practical option.

The use of grease as a de-bonding agent is not widely practiced. Wrapping the de-bonded half of the plain round dowel with plastic tape (or a tight fitting plastic sleeve) is the preferred, and more common, option. It is important that the de-bonded end of the dowel is saw-cut and not sheared. Shearing is often undertaken by bar suppliers and induces a non-desirable deformation in the end of the bar which may inhibit slip.

A slightly modified detail for use with the two non-specific design standards is presented in Figure 4, on page 9.

NZS 4230 details provide for short-lapping and de-bonding the horizontal deformed reinforcing in lieu of providing plain round dowels. While this is a more practical detail than is provided for in the non-specific design standards it is necessary that the de-bonding tape possesses the ability to compress to allow the bar deformations to freely move without over-stressing the surrounding grout and masonry. The use of grease-impregnated tape such as Denso is considered capable of achieving that aim. NZS 4230 designers using that detail need to be aware the horizontal steel passing through the joint will not be developing bar strength in the same manner that a standard hooked or 90º returned bar would. This could be significant, particularly for short wall panels, if horizontal reinforcing is required to contribute to in-plane shear capacity.

Sealants

All standards require control joints on exposed external faces of walls to be weatherproofed. Typically, this is provided by using a flexible sealant to seal off the exposed joint.

The non-specific design standards call for this joint to be 10mm wide and for the sealant to have a movement capability of ±25%, equating to 2.5 mm total movement in the joint. Polysulphide or modified silicone sealants are generally capable of providing that level of movement, but typically require a sealant width/depth aspect ratio of 2:1 to achieve that. Commonly available sealants also require a minimum bonded surface of 8mm, applying to the two sides of the joint, and it is important that the rear face of the sealant is de-bonded from the block substrate. A modified detail applicable for the non-specific design standards is presented in Figure 5 on page 9.

Using the criteria discussed in the foregoing *shrinkage* and *carbonation* sections the total movement in a typical joint would be at least double the 2.5 mm accommodation allowance provided for in a 10 mm wide joint. In practice restraint provided by foundations, floor slabs, and bond beams are likely to reduce the total movement occurring in the joints (non-significant cracking of the blockwork might be experienced between control joints). Where these factors don't exist, or otherwise as a result of specific design consideration, a wider sealant joint may be required. This could be achieved by saw-cutting the face shells to provide the minimum width to depth ratio required, with a limitation of 20 mm wide by 10mm deep.

Sealants in control joints should also be used in sub-floor spaces and the like where humidity in the atmosphere might compromise the reinforcing steel passing through the control joint.

Sealant is generally not required for joints in interior spaces of buildings that are not seen, ordinary pointing can be used.

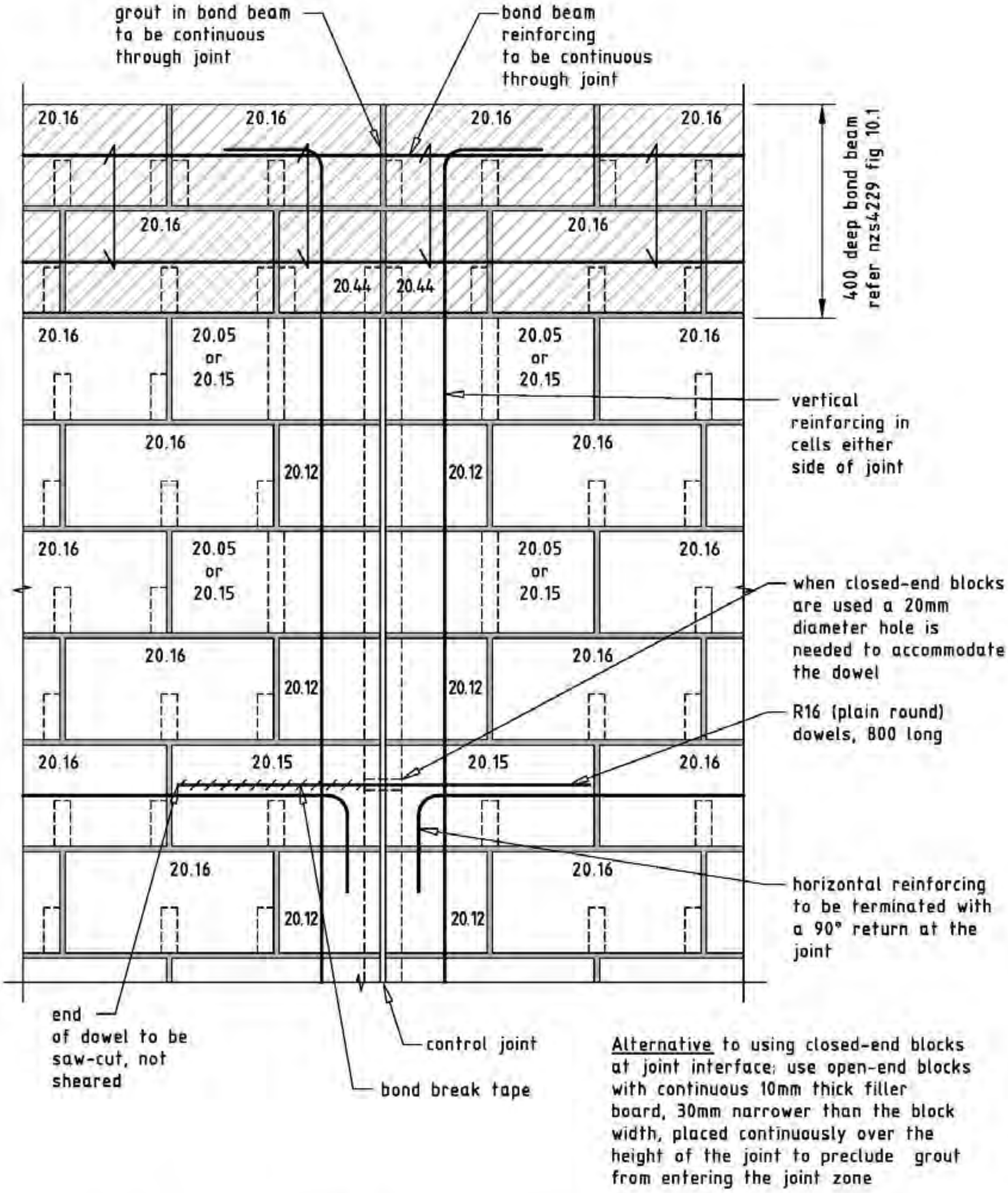


Figure 3 - Control joint elevation (NZS4229)

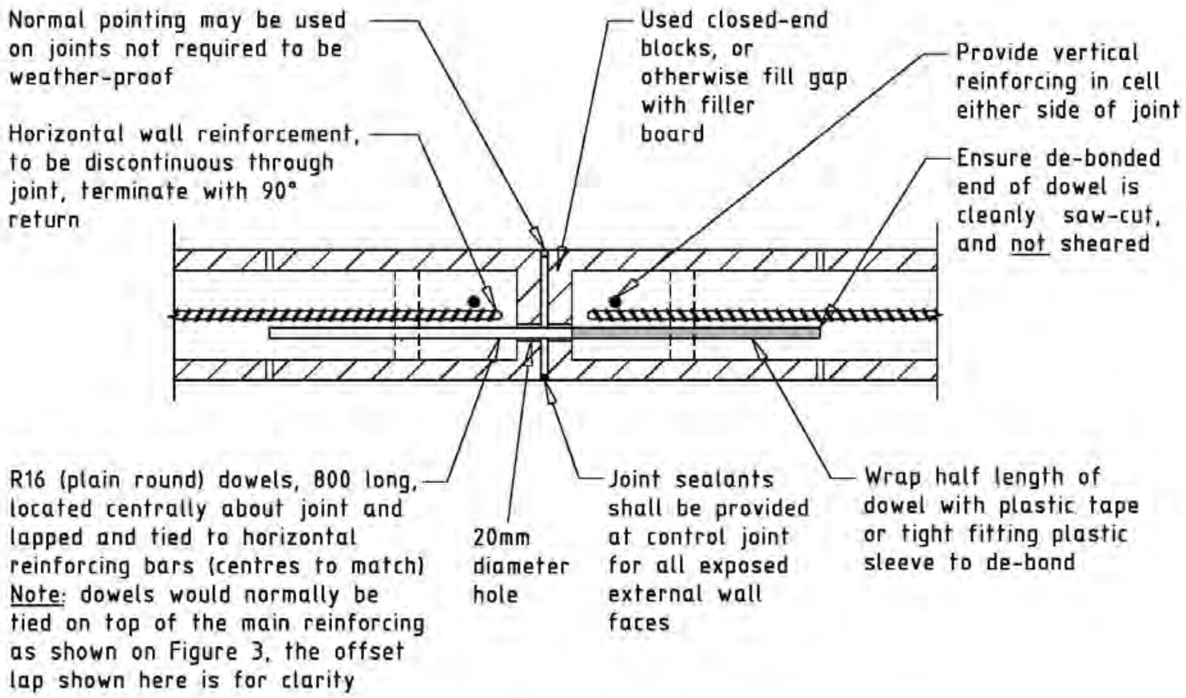


Figure 4 – Control joint detail for solid and partial fill walls (does not apply to bond beam and lintel reinforcing)

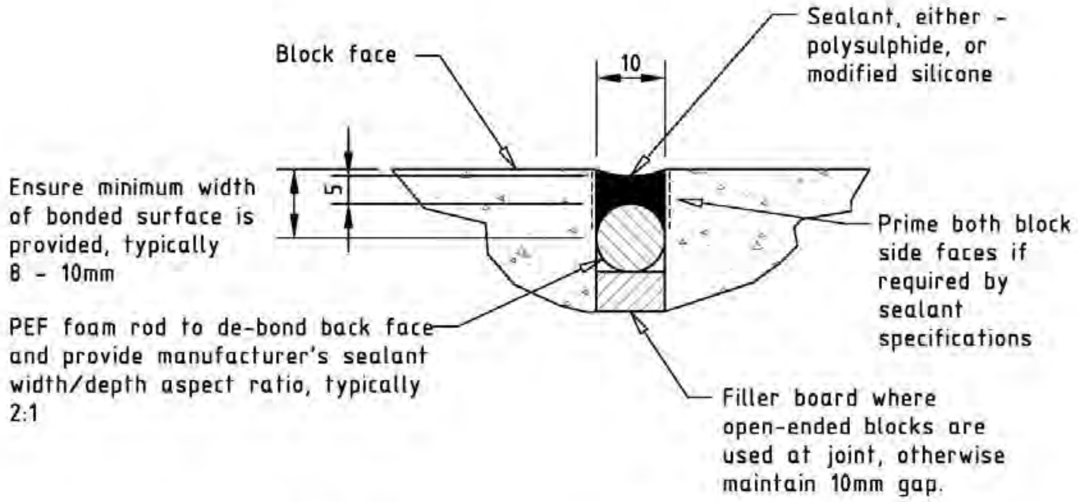


Figure 5 – Sealant Detail

References

1. NZS 4229 *Concrete masonry buildings not requiring specific design*

2. NZS 4230 *Design of reinforced concrete masonry structures*

3. NZS 3604 *Timber-framed buildings*

4. NZS 4210 *Masonry Construction – Materials and Workmanship*

5. National Concrete Masonry Association Tek 10-1A *Crack Control in Concrete Masonry Walls*

7.1 Concrete Segmental and Flagstone Pavers

Guide to Specifying

Introduction

To provide specifiers with an understanding of the product, this Guide sets out the requirements for the manufacture of concrete segmental pavers and flags. It takes account of the latest research and development and references:

- New Zealand Standard *NZS 3116 Concrete Segmental and Flagstone Paving*,
- Joint Standards AS/NZS*4456 Masonry Units, Pavers, Flags and Segmental Retaining Wall Units – Methods of Test,* and
- AS/NZS *4455.2 Masonry Units, Pavers, Flags and Segmental Retaining Wall Units - Part 2 Pavers and Flags.*

Industry design, detailing and construction guides should also be referenced when specifying concrete segmental and flagstone pavements.

These Standards are available from:

Standards New Zealand
Private Bag 2439
Wellington 6140

Freephone	0800 782 632 (New Zealand)
Phone	+64 4 498 5990
Fax	+64 4 498 5994
Email	enquiries@standards.co.nz
Website	www.standards.co.nz

Definitions

- **Abrasion Resistance**

  A measure of resistance to erosion of the surface of a paver or flag, expressed as an index, when tested in accordance with AS/NZS 4456.9.

- **Annual Average Daily Traffic (AADT)**

  The total volume of traffic passing a point in the pavement, in both directions, for one year divided by the number of days in the year.

- **Breaking Load**

  The failure load determined in accordance with AS/NZS 4456.5 as modified by NZS 3116.

- **Characteristic Value**

  The value that is exceeded by at least 95% of the units in the lot.

- **Commercial Vehicle (CV)**

  A vehicle, having a gross weight of 3 t or more, that complies with legislation for the axle load, tyre pressures and dimensions of vehicles permitted on public roads and streets.

- **Dimensional Deviation**

  The deviation from work size of paving units when determined in accordance with AS/NZS 4456.3.

- **Flagstone**

  Large format solid (non-cored) paver with a gross plan area greater than 0.08 m^2.

- **Light Vehicle (LV)**

  A vehicle which, when fully loaded, has a gross weight less than 3t.

  NOTE: This category includes cars, utilities, delivery vans and some light two-axle trucks.

- **Industrial Pavements**

  Pavements that may be subject to a range of unregulated vehicle types, axle configurations, wheel and tyre pressures.

- **Lot**

  A group of units of a single type with specific characteristics and dimensions presented for sampling at the same time.

- **Paver**

  Solid unit with a gross plan area less than or equal to 0.08 m^2 which is used to form a surfacing layer.

- **Salt Attack Resistance**

  Resistance to attack by the action of soluble salts, determined by the action of sodium sulphate or sodium chloride, in accordance with AS/NZS 4456.10.

- **Slip Resistance Class**

  A classification of slip resistance as determined in accordance with AS/NZS4586.

- **Traffic Loading Classification**

  NZS 3116 sets out the full details of the traffic loading:

  (a) Residential Paving:

  – No vehicles.

  (b) Residential Driveways:

  – Light traffic
  – Medium traffic

  (c) Public Footpaths:

  – Low impact
  – High impact (high volume, malls etc.)

  (d) Roads

  – Minor (up to 150 vehicles a day)
  – Local (150-400 vehicles a day)
  – Main (over 400 vehicles a day)

  (e) Industrial

  For full details see *NZS 3116 Concrete Segmental and Flagstone Paving*.

- **Work Size**

  The size of a unit specified for its manufacture, from which deviations are measured.

Concrete Paver Selection

Table 1: Maximum Requirements for Dimensions Breaking Load and Abrasion Resistance *(based on Table 1, NZS 3116)*

Applications	Characteristic breaking load(1) (kN) per 100 mm width	Minimum thickness(2) (mm)	Shape(3)	Dimensional tolerances(4)	Edge detail(5)	Abrasion resistance(6) at 56 days mean	Minimum slip resistance classification(8)
Relevant AS/NZS	**4456.5**	**-**	**-**	**4455/4456.3**	**-**	**4456.9**	**4586**
1. **Residential**							
Pedestrian	3.0	40	Any	DPB1	SQ/SC/R/CH	Not required	W
2. **Residential driveways**							
Light Traffic	5.0	50	Any	DPB2	CH/R	Not required	W
Medium Traffic	Follow provisions of application 4 Roads: Minor						
3. **Public Footpaths**							
Low Impact	5.0	50	Any	DPB2	SQ/SC/CH	6.0	W
High Impact	5.0	50	Any	DPB2	SQ/SC	3.5	W
4. **Roads**							
Minor	6.0	60	Rr/Dd	DPB2	CH	Not required	W
Local	12.0	80	Rr/Dd	DPB2	CH	Not required	W
Main	12.0	80	Rr/Dd	DPB2	CH	Not required	W
5. **Industrial Pavements**	Specific engineering design(7)	80	Rr/Dd	DPB3	CH	See Note (7)	W

NOTES to Table 1

(1) The characteristic breaking load to AS/NZS 4456.5, as amended by clause 202(b), is carried out on a 150 mm actual paver width in mm. The figures quoted are based on a 100 mm width, i.e. actual breaking load x the ratio of 100 mm divided by the actual paver width mm. The modulus of rupture value of any paver shall not be less than 4 MPa. Where pavers may be subject to chemical/environmental exposure e.g. marine, swimming pools, thermal pools etc., it is recommended that they be subjected to the resistance test contained in AS/NZS 4456.10 to demonstrate an acceptable performance at 50 cycles of test.

(2) In application 3 where pedestrian areas may be subject to service vehicles, a 60 mm SC paver is recommended.

(3) The principal shapes are:

(a) Rectangular 2:1 ratio (Rr);
(b) Rectangular 2:1 ratio (Dd) but dentated for additional interlock;
(c) Approximately square, see 304.1 for laying patterns.

(4) DPB is fully defined in AS/NZS 4455 and relates to dimensions (D) of paver (P) and specifies a method of measurement (B) with a tolerance (1 or 2). The method of measurement is contained in AS/NZS4456.3.

(5) Definitions:

SQ - square edge
SC - shallow chamfer no deeper than 2 mm and no wider than 7 mm.
R - rumbled
CH - chamfer no deeper than 4 mm in depth and no wider than 7 mm.

(6) The abrasion index figures quoted are values established as criteria for satisfactory performance of pavers going into service at an age of 56 days in areas subjected to pedestrian impact traffic. Typical abrasion test index values at 28 days rather than 56 days are 7 and 4 respectively. Where abrasion resistance is required, the drawings, specification, and/or purchase order for the pavers should specify the abrasion resistance value to be achieved.

(7) Industrial pavers may be required to have special strength requirements. Specific engineering design requirements need to be agreed between the specifier/designer and the producer. These may require a specified abrasion index. Alternative testing regimes may be arranged between the producer and the product user.

(8) Products with minimum slip resistance classification W when tested in accordance with AS/NZS 4586 and used in accordance with SAA HB 197 and AS/NZS 3661.2 provide an Alternative Solution for the Compliance Document for NZBC Clause D1.

Flagstone Selection

Table 2: Flagstone Selection: Minimum Requirements for Dimensions Breaking Load and Abrasion Resistance *(based on Table 1A, NZS 3116)*

Pavement Applications	Characteristic breaking load (kN) per 100 mm width(1)	Nominal Size (mm)	Minimum thickness (mm)	Dimensional tolerance(2)	Flatness tolerance (mm)	Edge Detail(3)	Abrasion resistance at 56 days (mean)(4)	Minimum slip resistance(5)
Relevant AS/NZS	**4456.5**	**-**	**-**	**4455/4456.3**	**-**	**-**	**4456.9**	**4586**
1. **Residential**	2.6	600 x 600	40	DPB1	2.5	SQ/SC/CH	Not required	W
Pedestrian		500 x 500	40		2.2			W
		450 x 450	40		2.0			W
		400 x 400	40		1.5			W
		300 x 300	40		1.0			W
2. **Residential driveways**	3.8	300 x 300	60	DPB2	1.0	CH	Not required	W
Light Traffic								
3. **Public Footpaths**	3.8	450 x 450	60	DPB2	2.0	SQ/SC/CH	Low Impact 6.0	W
Low Impact		400 x 400	60		1.5		or	W
High Impact		300 x 300	60		1.0		High Impact 3.5	W

NOTES to Table 2

(1) Breaking loads are characteristic on a 250 mm span by 100 mm nominal width. For large sizes, specimens may be cut, tested, and compared with the results above.

(2) DPB is fully defined in AS/NZS 4455 and relates to dimensions (D) of paver (P) and specifies a method of measurement (B) with a tolerance (1 or 2). The method of measurement is contained in AS/NZS 4456.3.

(3) Definitions:

SQ - square edge
SC - shallow chamfer no deeper than 2 mm and no wider than 7 mm.
CH - chamfer no deeper than 4 mm in depth and no wider than 7 mm.

(4) The abrasion index figures quoted are values established as criteria for satisfactory performance of flagstones going into service at an age of 56 days in areas subjected to pedestrian impact traffic. Typical abrasion test index values at 28 days rather than 56 days are 7 and 4 respectively. Where abrasion resistance is required, the drawings, specification, and/or purchase order for the pavers should specify the abrasion resistance value to be achieved.

(5) Products with minimum slip resistance classification W when tested in accordance with AS/NZS 4586:2004 and used in accordance with SAA HB 197 and AS/NZS 3661.2 provide an Alternative Solution for the Compliance Document for NZBC Clause D1.

Pavement Thickness Design and Construction

See NZS 3116, Section 3.

Bedding Sand Abrading Stability

See NZS 3116, Appendix A.

Table 3: Maximum Dimensional Deviations Determined for Pavers and Flagstones by Individual Measurements *(from Table 2.2(B) AS/NZS 4455.2)*

Category	**Work Size Dimensions, mm**			
	Plan		**Height**	
	Standard Deviation	**Mean**	**Standard Deviation**	**Mean**
DPO	No requirement			
DPB1	2.0	±3.0	3.0	±2.5
DPB2	2.0	±2.5	3.0	±2.0
DPB3	Values declared by the supplier or by agreement between supplier and purchaser			
DPB4	1.5	±2.0	2.0	±2.0

All paving and flag units will be categorized in accordance with *AS/NZS 4455.3 Method for Determining Dimensions*.

7.2 Segmental Paving Design and Construction

Introduction

Section 7.1 has dealt with the specification of the segmental paving unit.

This Section 7.2 gives some guidance to matters of design and construction.

Design

For most applications in New Zealand, whether it is segmental paving or flagstones, NZS 3116 provides sufficient guidance on design and construction thickness of the overall pavement application. Where a more detailed study is required then the Concrete Masonry Association Australia (CMAA) has several publications that provide information on design:

- *Concrete Segmental Pavements – Detailing Guide* (T46).
- *Concrete Flag Pavements – Design and Construction Guide* (MA44).

These publications are free for download from: http://www.cmaa.com.au/html/TechInfo/TechInfoPaving.html.

An alternative New Zealand reference is IB 67 *Interlocking Concrete Block Road Pavements* free for download from the Cement & Concrete Association of New Zealand (CCANZ) website: http://www.ccanz.org.nz/files/documents/135d9e0c-2b78-4270-9dcb-c22c3d7086ea/IB%2067.pdf. This IB does not deal with flagstone pavements

Please note, however, that the NZCMA does *NOT* recommend the use of flagstones (flags) where vehicular traffic used the paving. This is based on practical experience in New Zealand.

Please also note that NZS 3116 has a requirement and test procedure for determining the suitability of bedding sands.

In the more heavily trafficked applications, it is imperative to use a bedding sand that does not abrade. Sands containing pumice are particularly susceptible to the abrading failure.

Experience has shown that many earlier problems with segmental paving related to a failure to understand the importance of the bedding sand.

To summarise:

(a) Follow grading provisions for the applications.

(b) Check that the sand has the appropriate abrading resistance (See NZS 3116).

(c) Do not use crusher dust fines because after time the dust forms an impervious layer trapping water on the underside of the segmental paving. Water must be able to 'flow' in the bedding sand.

Construction Practices

Two construction methods are outlined in NZS 3116.

For flagstones, the pre-compacted method is one recommended.

A useful construction detail reference is:

- *Concrete Segmental Pavements – Detailing Guide* (T46).

Together with:

- *Concrete Segmental Pavements – Maintenance Guide* (MA48).

These publications are free for download from: http://www.cmaa.com.au/html/TechInfo/TechInfoPaving.html.

An alternative reference is IB 68 *Construction of Concrete Block Paving,* free for download from the Cement & Concrete Association of New Zealand (CCANZ) website: http://www.ccanz.org.nz/files/documents/a20d7e5d-b998-4397-b2ca-4e9aacfa5516/IB%2068.pdf.

7.3 Permeable Interlocking Concrete Pavements

Introduction

More recent developments in uses for interlocking pavements have seen pavements used to absorb water rather than shed it. The broad outlines of the technique are shown in Section 1.1 of the Masonry Manual.

NZCMA Company member, Firth Industries (www.firth.co.nz) has special access to permeable paving designs. See *Firth Ecopave System: Installation Guide* available from the following link: http://www.firth.co.nz/assets/Uploads/Brochures/Firth-ecopave-system-installation-guide.pdf.

SUDS Seminar

The SUDS seminar shown on the NZCMA website front page (http://www.nzcma.org.nz/home.aspx) highlights detailed presentations on the latest use of permeable paving.

Suggested use of the information is to open the PowerPoint presentation and evaluate the information. The verbal presentation of the PowerPoint slides can be obtained by scrolling through the appropriate video presentation.

Further information on Permeable Paving and SUDS can be found on the Interpave (Precast Concrete Paving and Kerb Association) website – http://www.paving.org.uk/commercial/permeable.php

Information on the Permeable Design Pro Software, along with a ten day free trial, can be found on the Interlocking Concrete Paving Institute website – http://www.permeabledesignpro.com/free-trial-registration.aspx.