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.

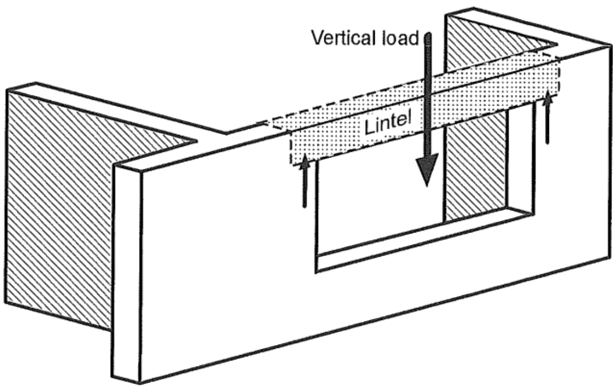


Figure 1.2: Lintel Load Actions

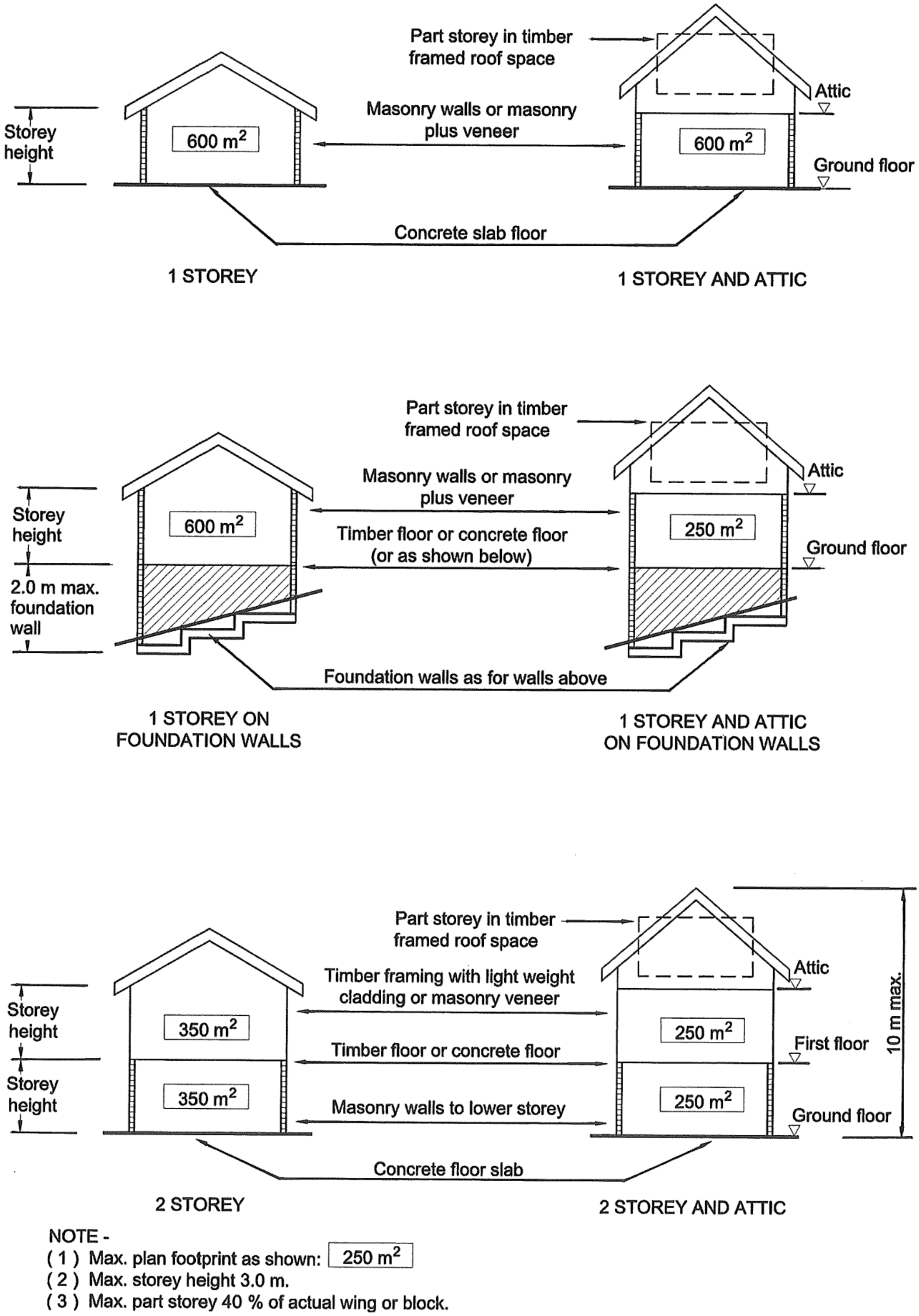


Figure 1.1: Building types covered by NZS 4229

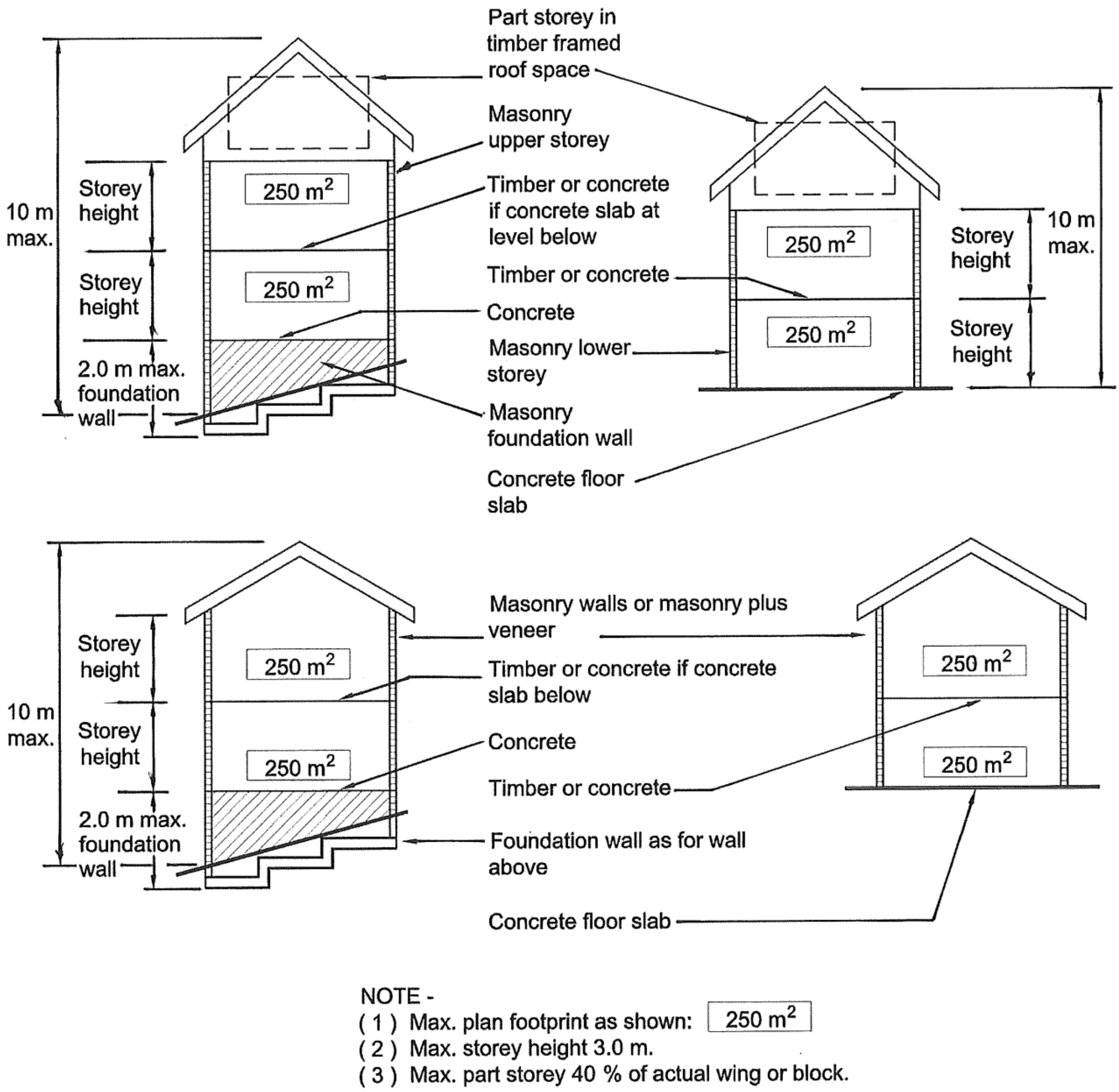


Figure 1.1: Building types covered by NZS 4229 (continued)

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.

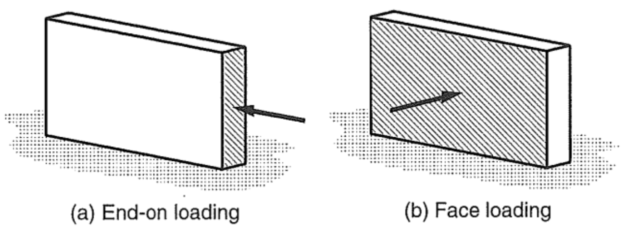


Figure 1.3: Wall Load Actions

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.

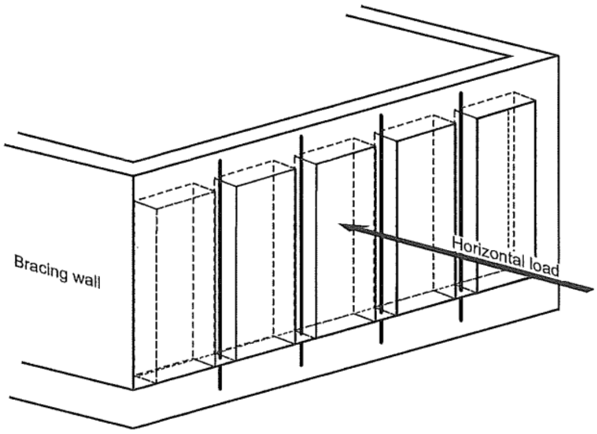


Figure 1.4: Wall Load Actions

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.

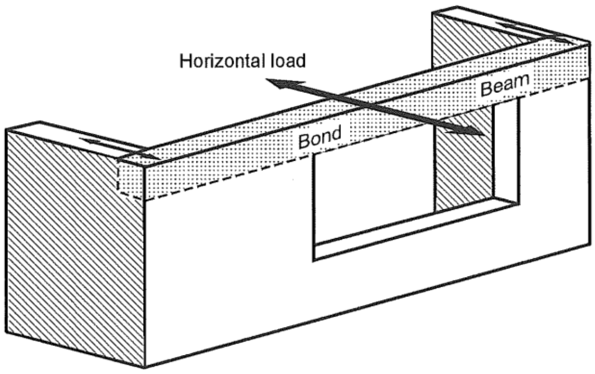


Figure 1.5: Bond Beam Load Actions

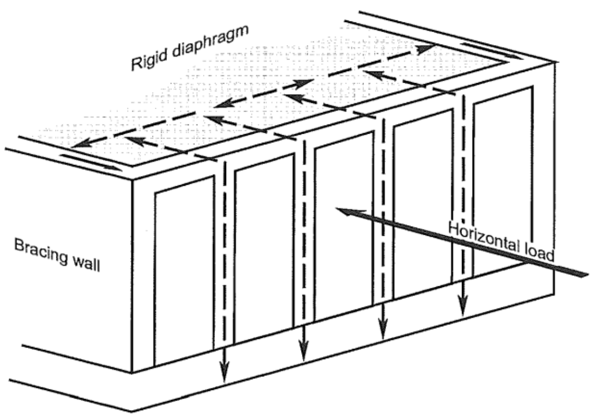


Figure 1.6: Diaphragm Load Actions

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.

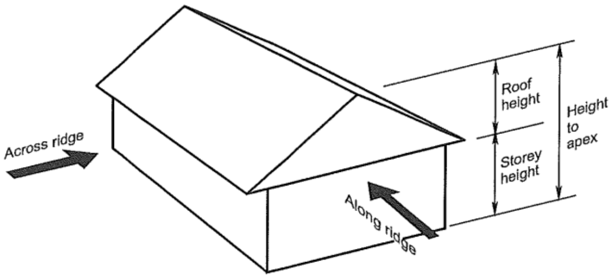


Figure 2.3: Wind Loading

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.

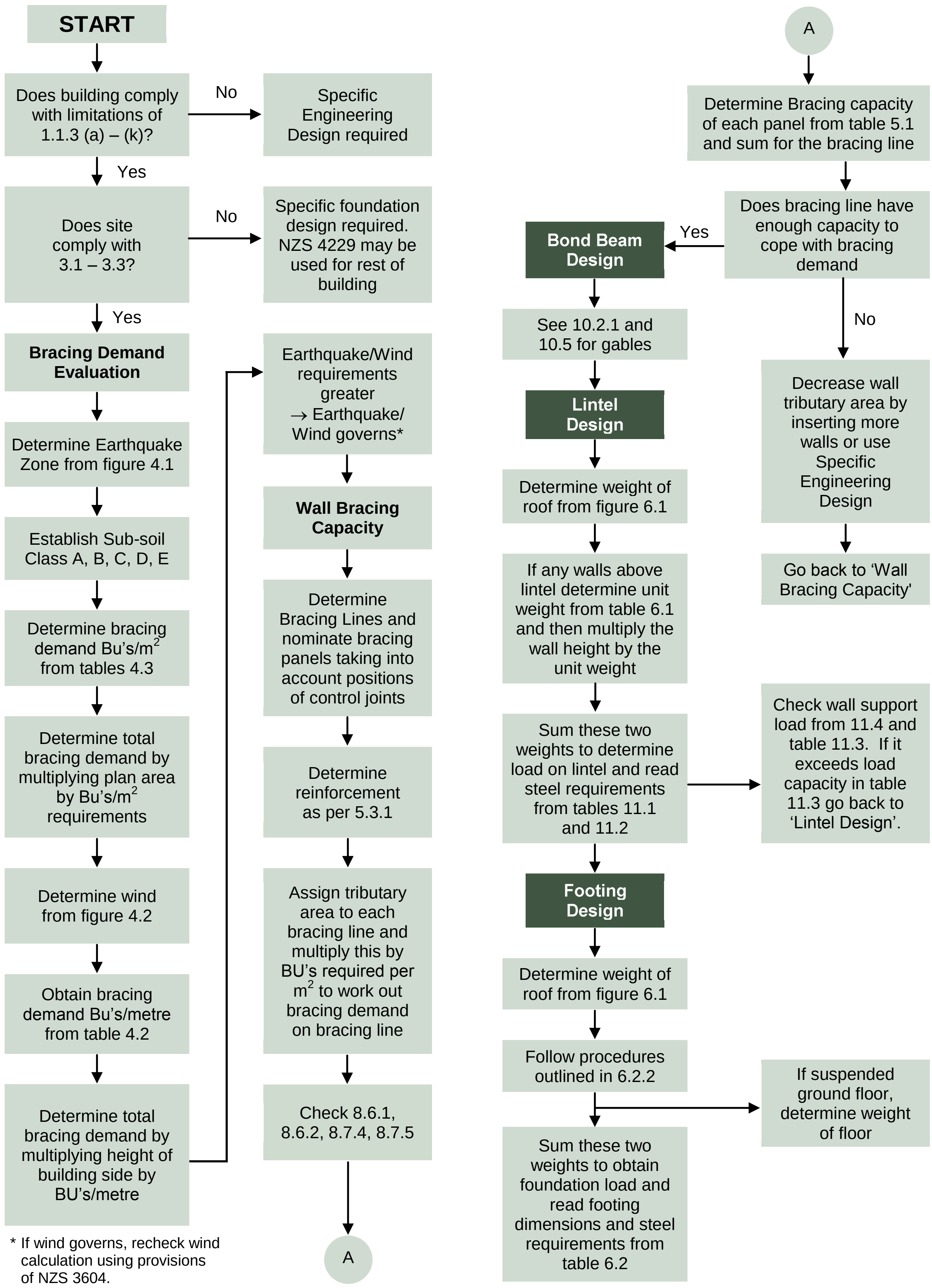


Figure 2.1: Flow Chart for a single storey design

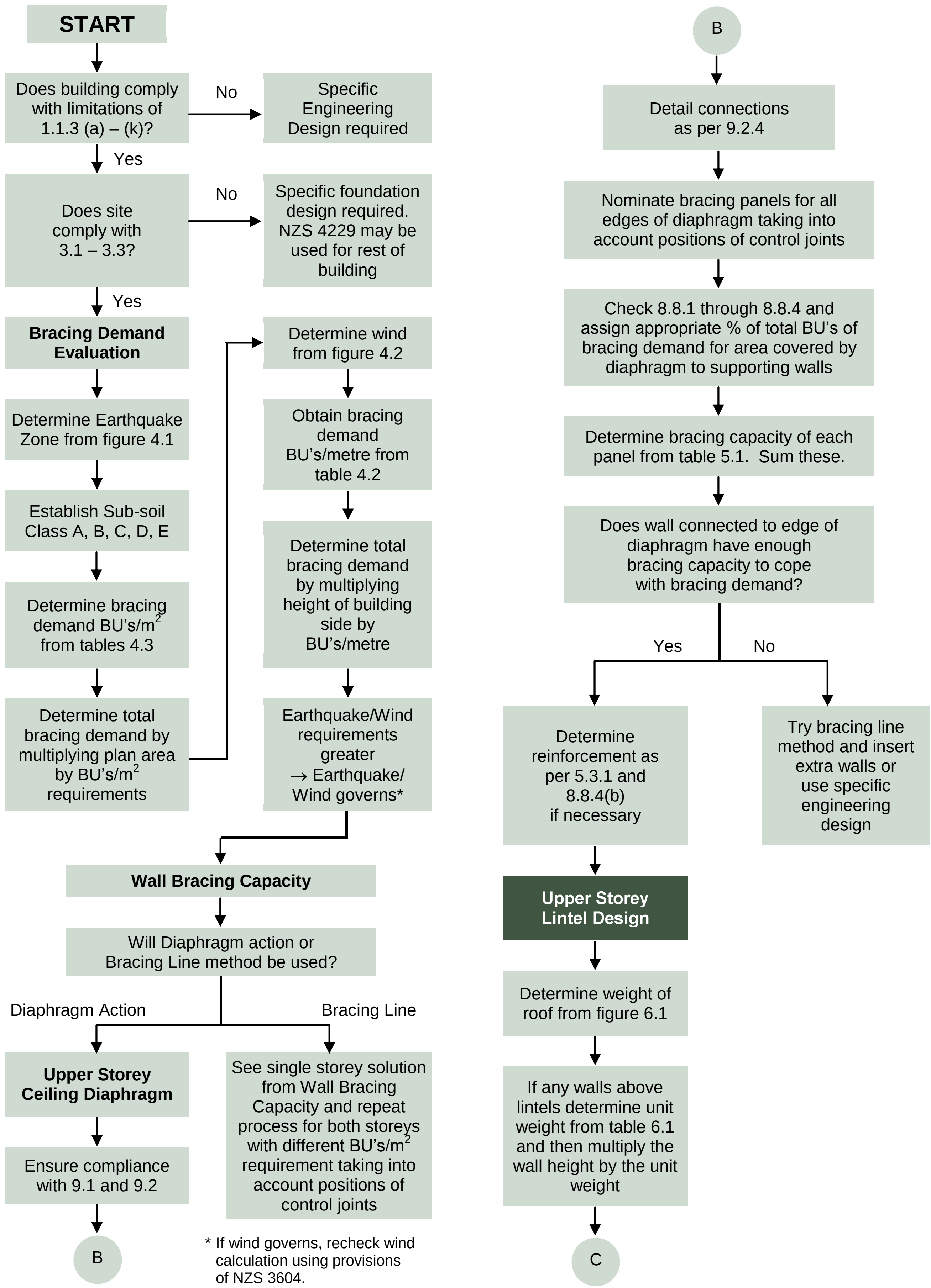


Figure 2.2: Flow Chart for a two storey design

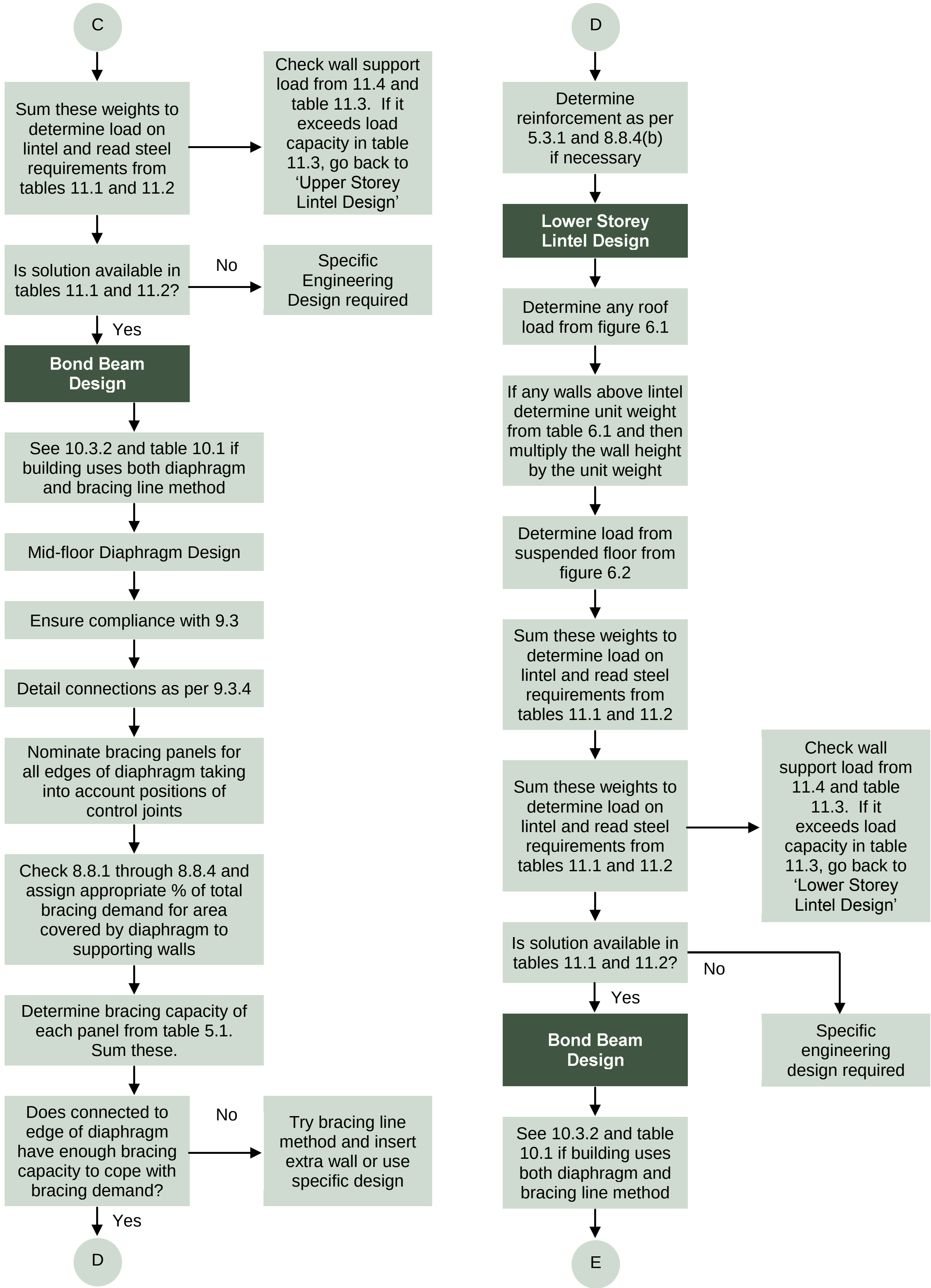


Figure 2.2: Flow Chart for a two storey design (continued)

E

↓

Footing Design

↓

Determine weight of roof from figure 6.1

↓

Determine unit weight of two storey wall from table 6.1 and then determine load by multiplying the wall height by the unit weight. (See 6.2.2).

↓

Determine load from suspended floor from figure 6.2

↓

Sum these loads to determine load on footing and read footing dimensions and steel requirements from table 6.2. (See 6.2.2).

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.

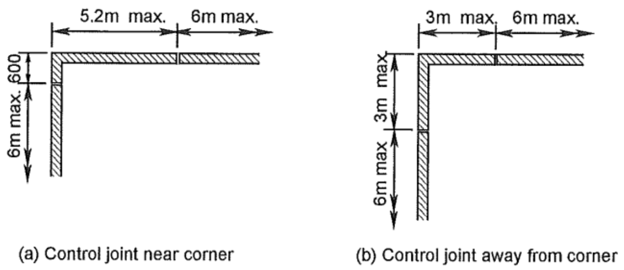


Figure 2.4: Location of Shrinkage Control Joints at Corners

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.

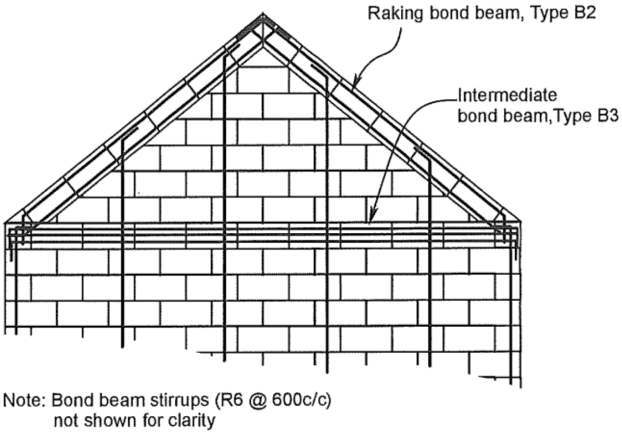


Figure 2.5: Gable End Reinforcing Details

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 Al 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.	

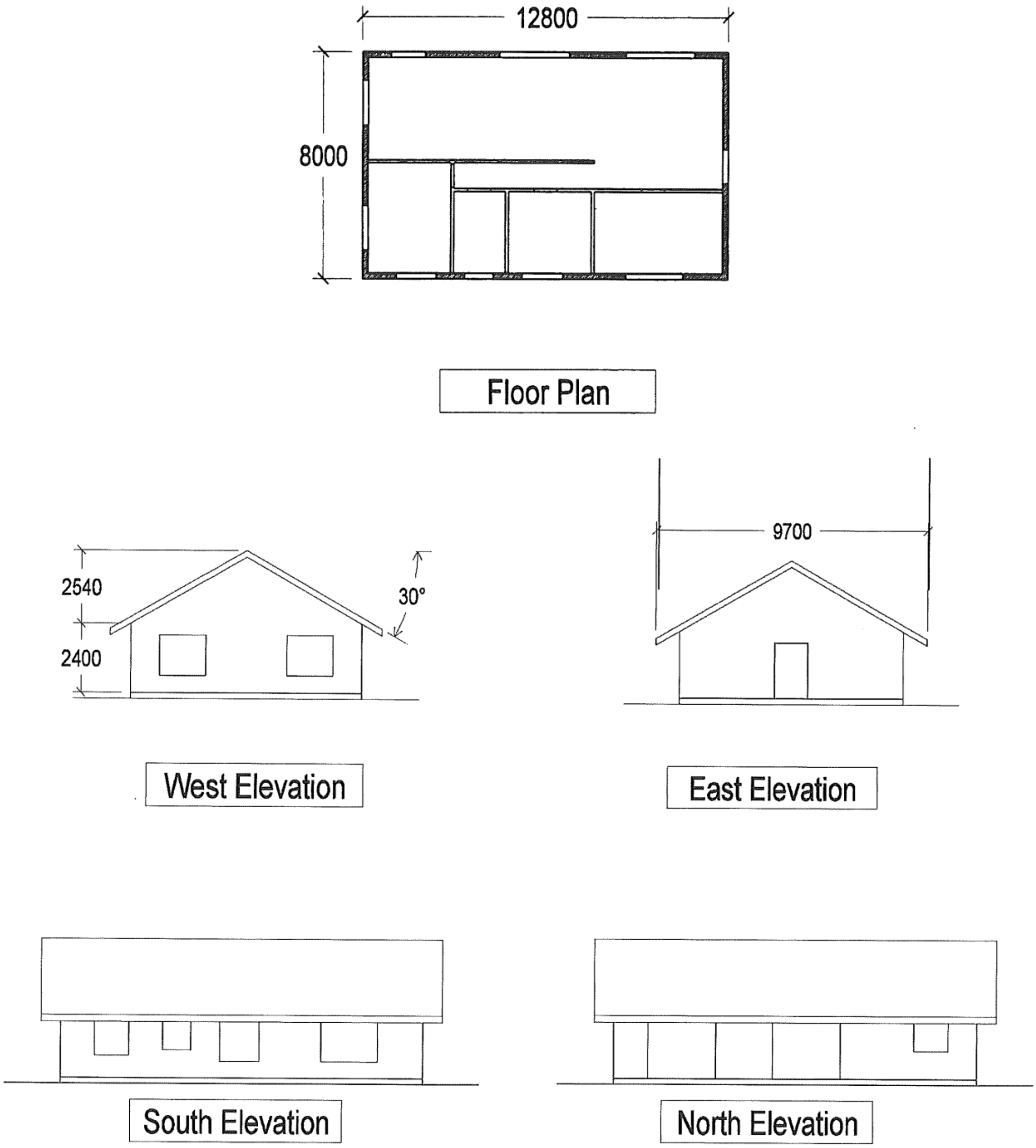


Figure 3.1.1: Design Example 1

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 102 BU/m along ridge
Table 4.2	→ Demand along ridge = 93 + 0.5 x (111 - 93) = 102 BU/m	
	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.

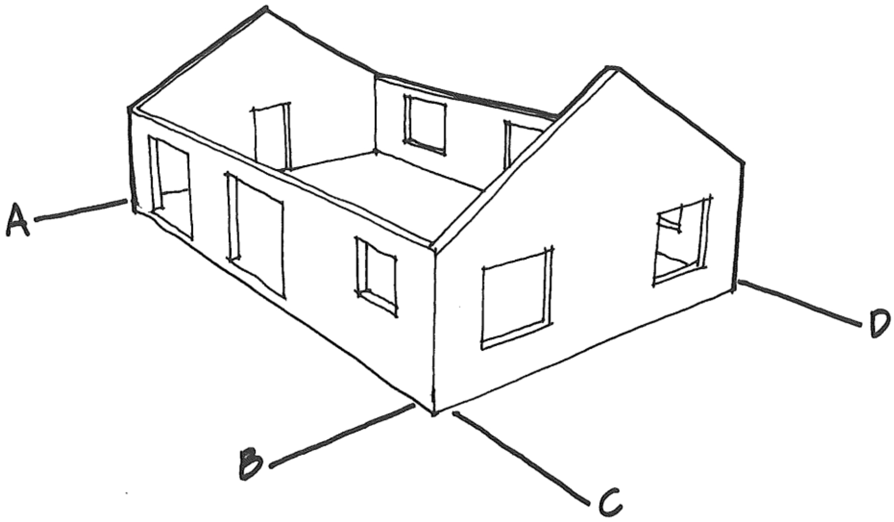


Figure 3.1.2: Bracing Lines for Design Example 1

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.

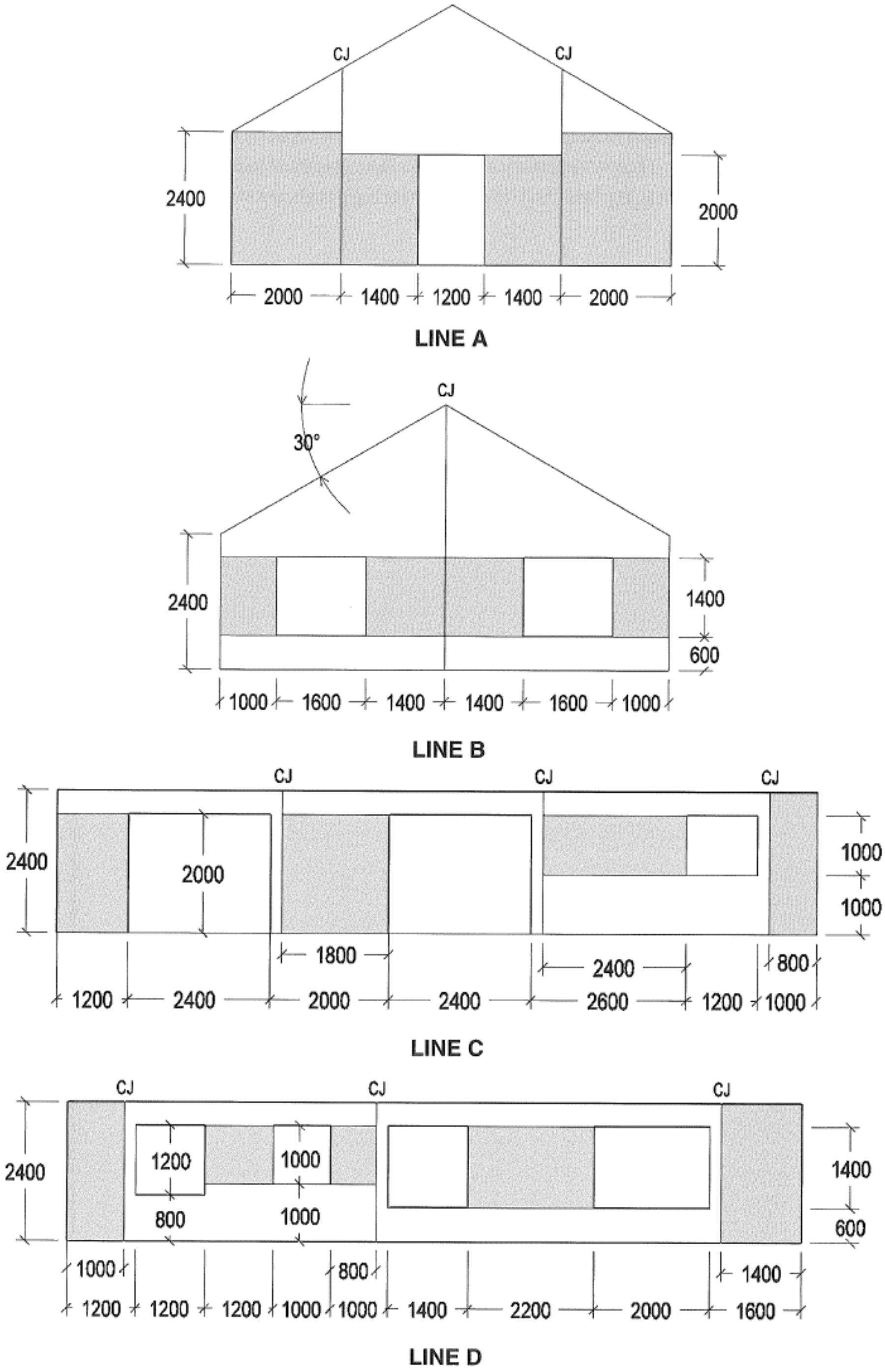


Figure 3.1.3: Bracing Panels for Design Example 1

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:

BU for 1.4 panel length = $\frac{305+480}{2}$ = 393 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.

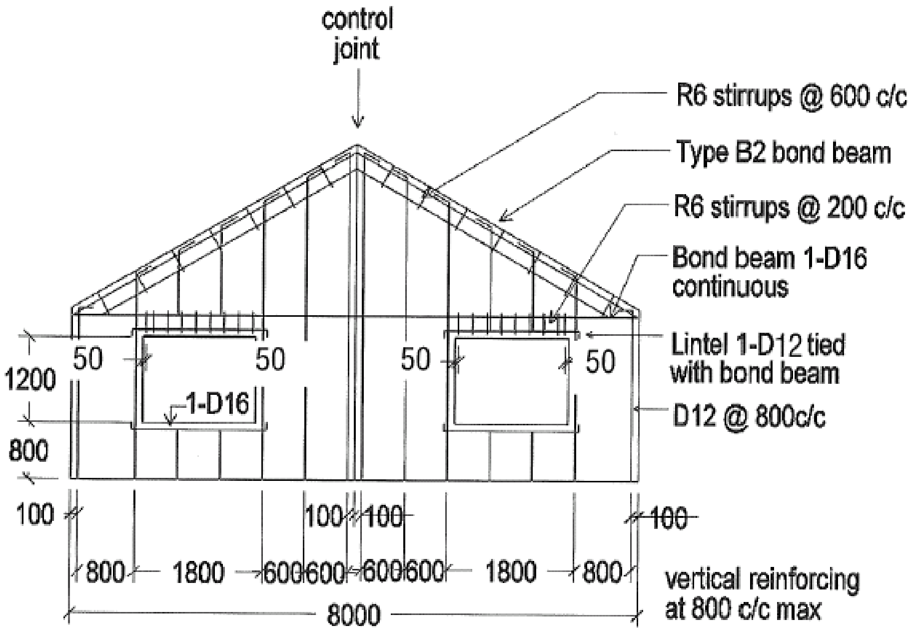


Figure 3.1.4: Bond Beam and Lintel Reinforcement for West Wall (Bracing Line B)

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.	

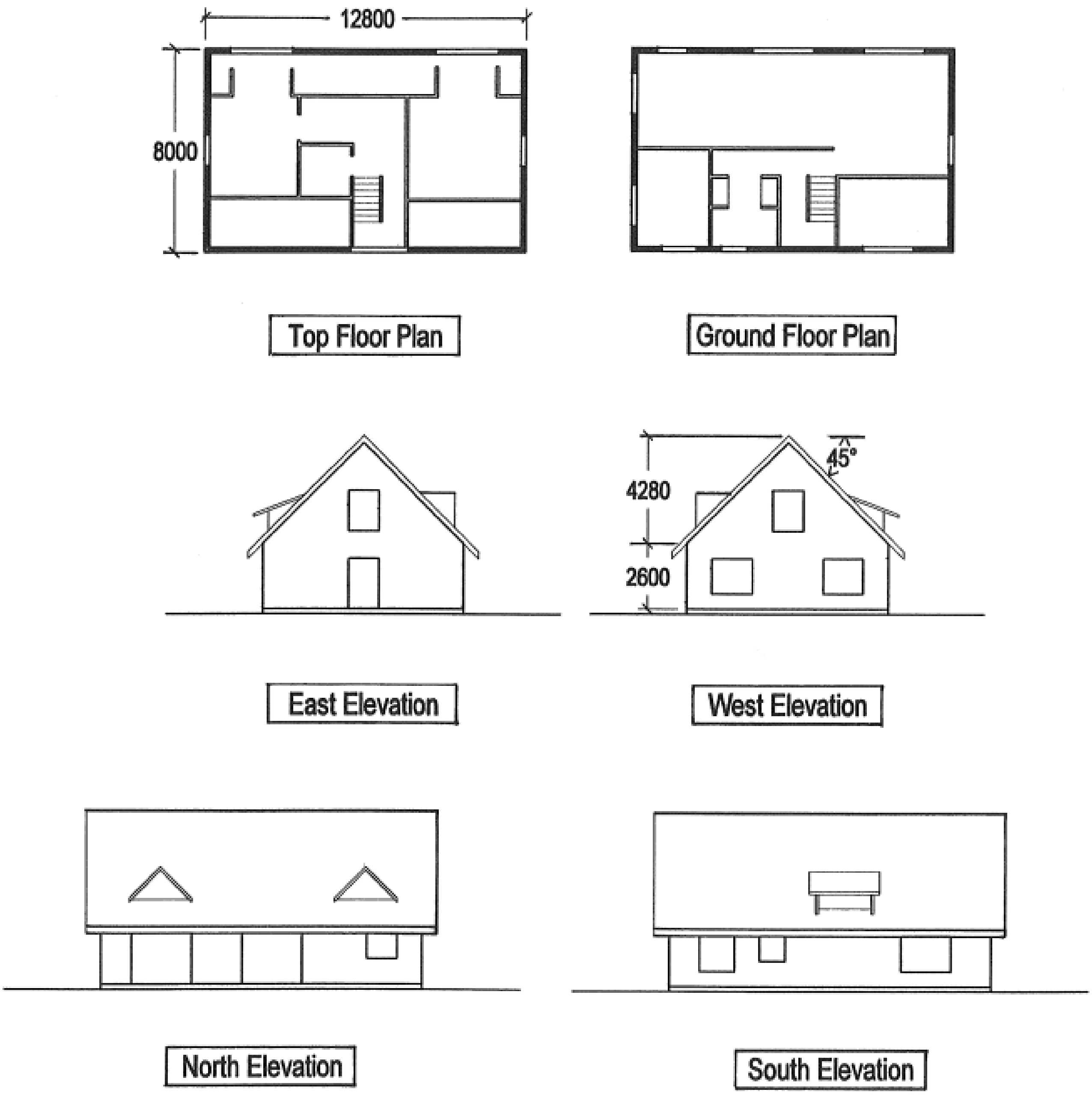


Figure 3.2.1: Design Example 2

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.

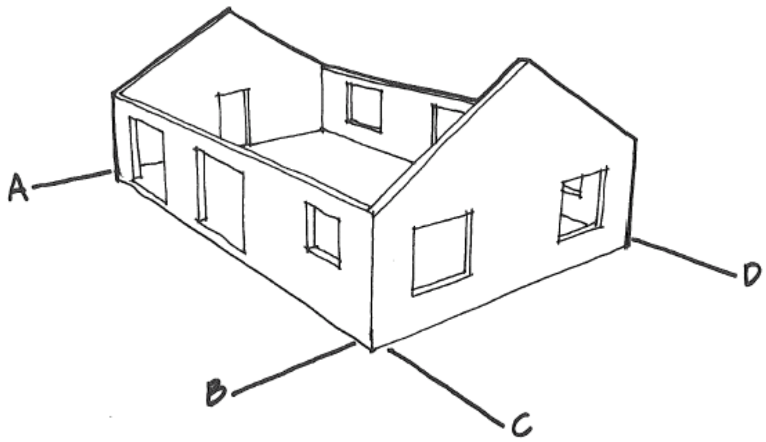


Figure 3.2.2: Bracing Lines for Design Example 2

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.

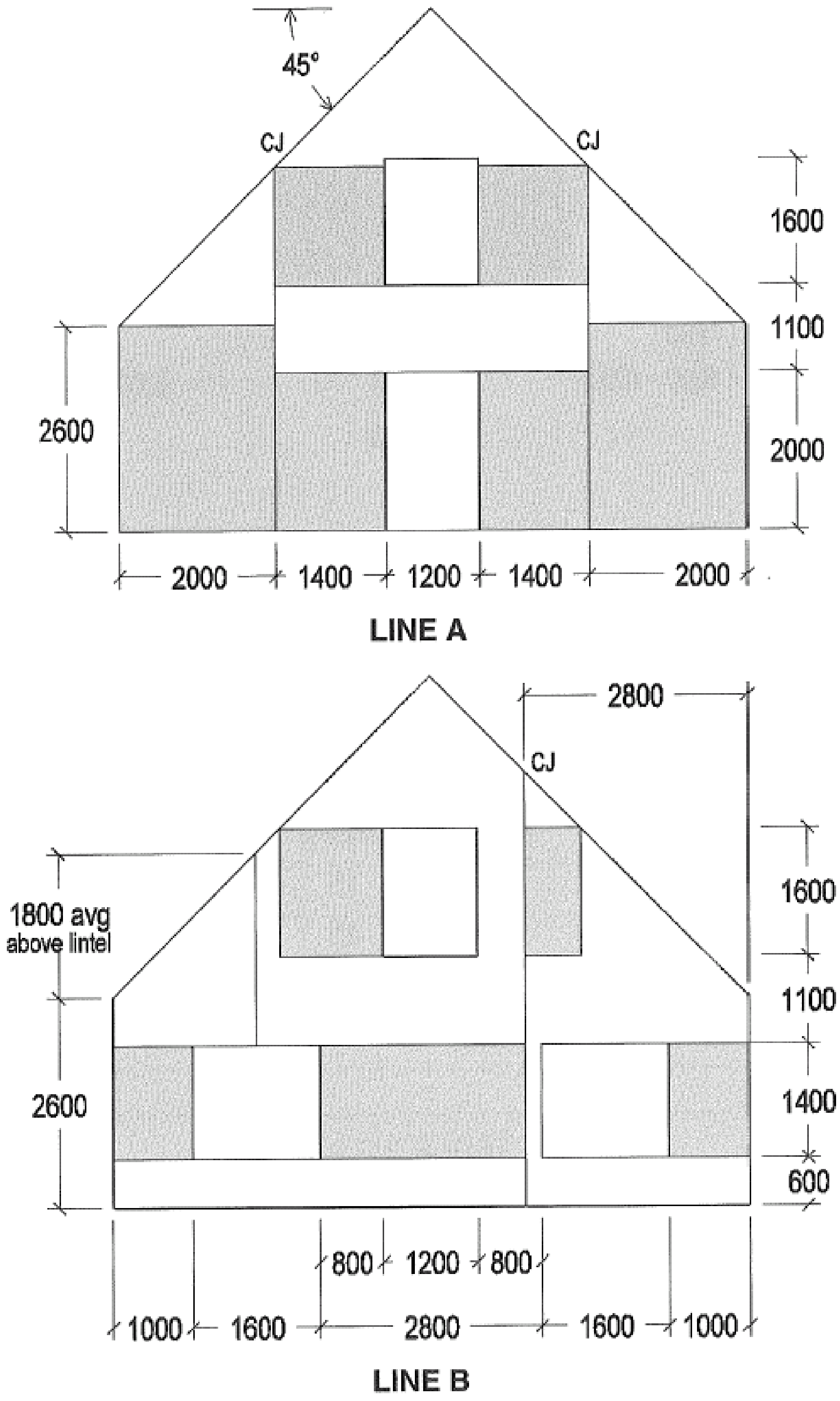


Figure 3.2.3: Bracing Panels for Design Example 2

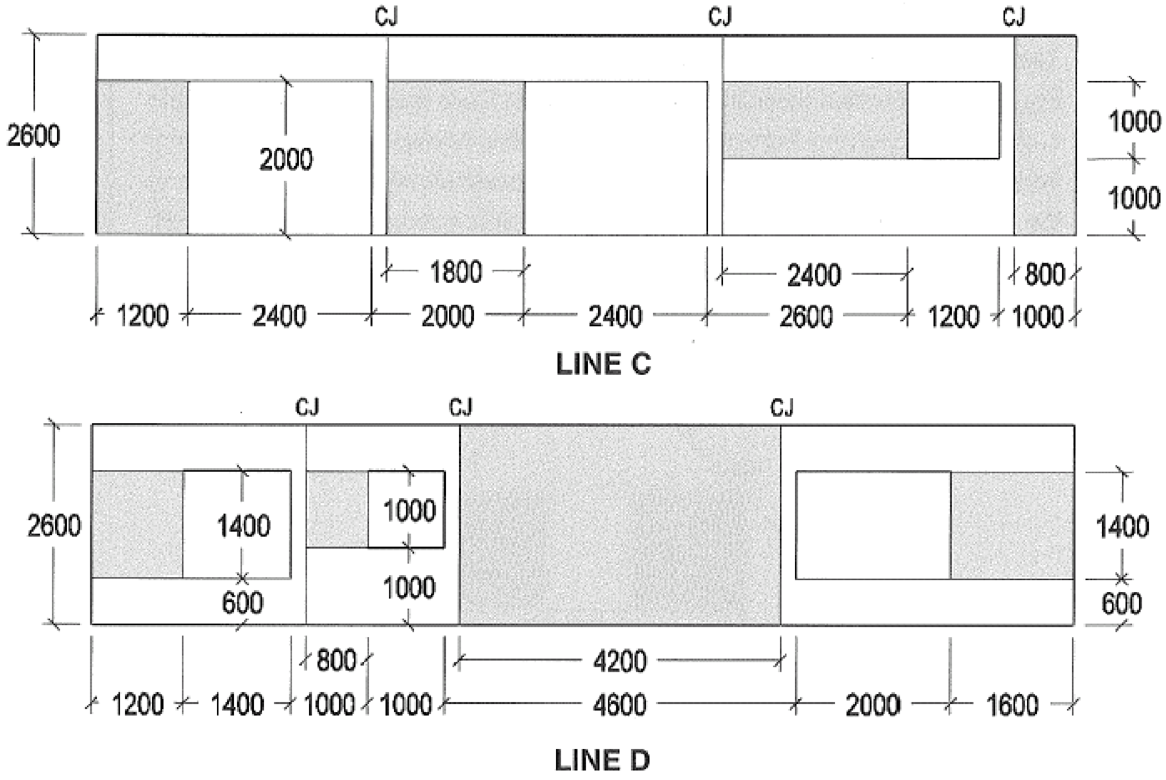


Figure 3.2.3: (Continued) Bracing Line D for Design Example 2

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

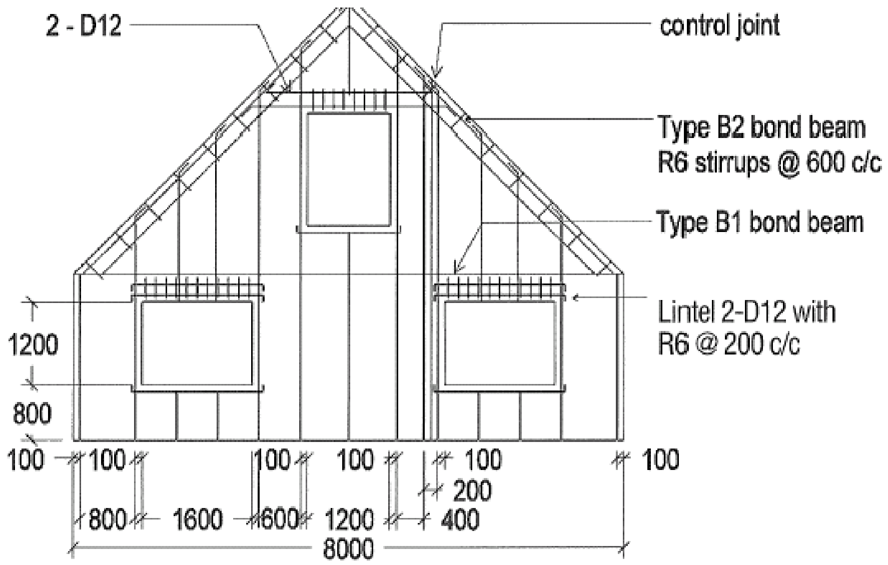


Figure 3.2.4: Bond Beam and Lintel Reinforcement for West Wall (Bracing Line B)

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.

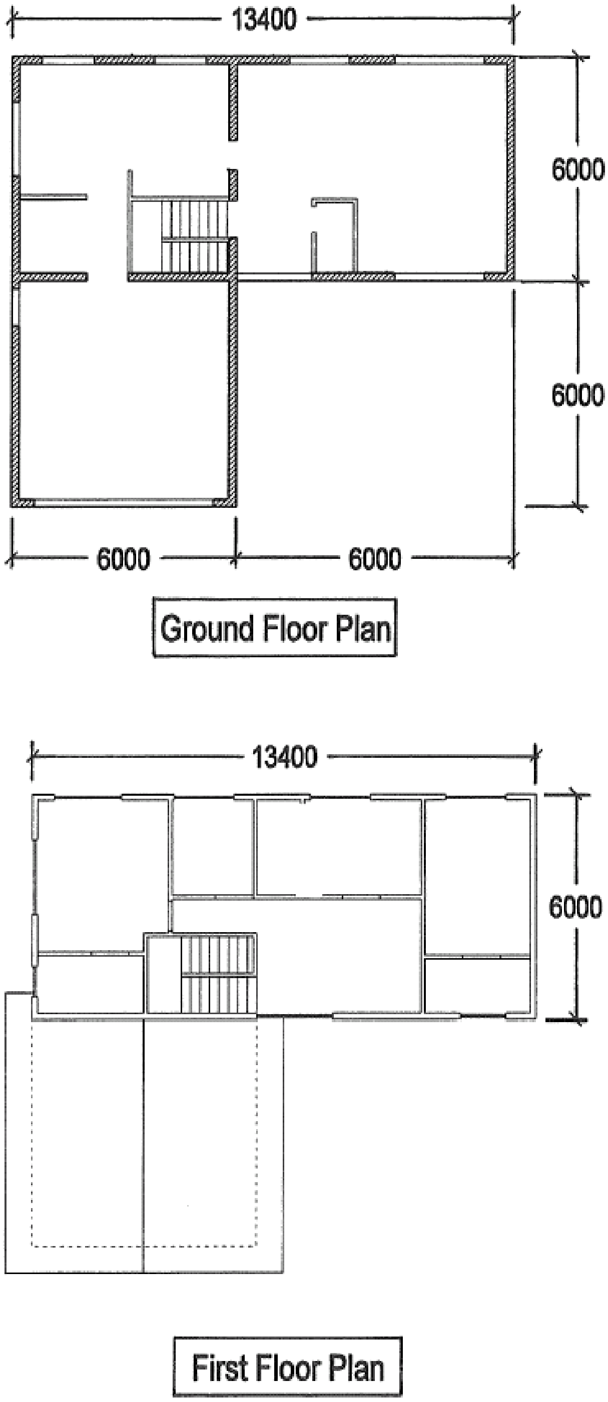


Figure 3.3.1: Design Example 3

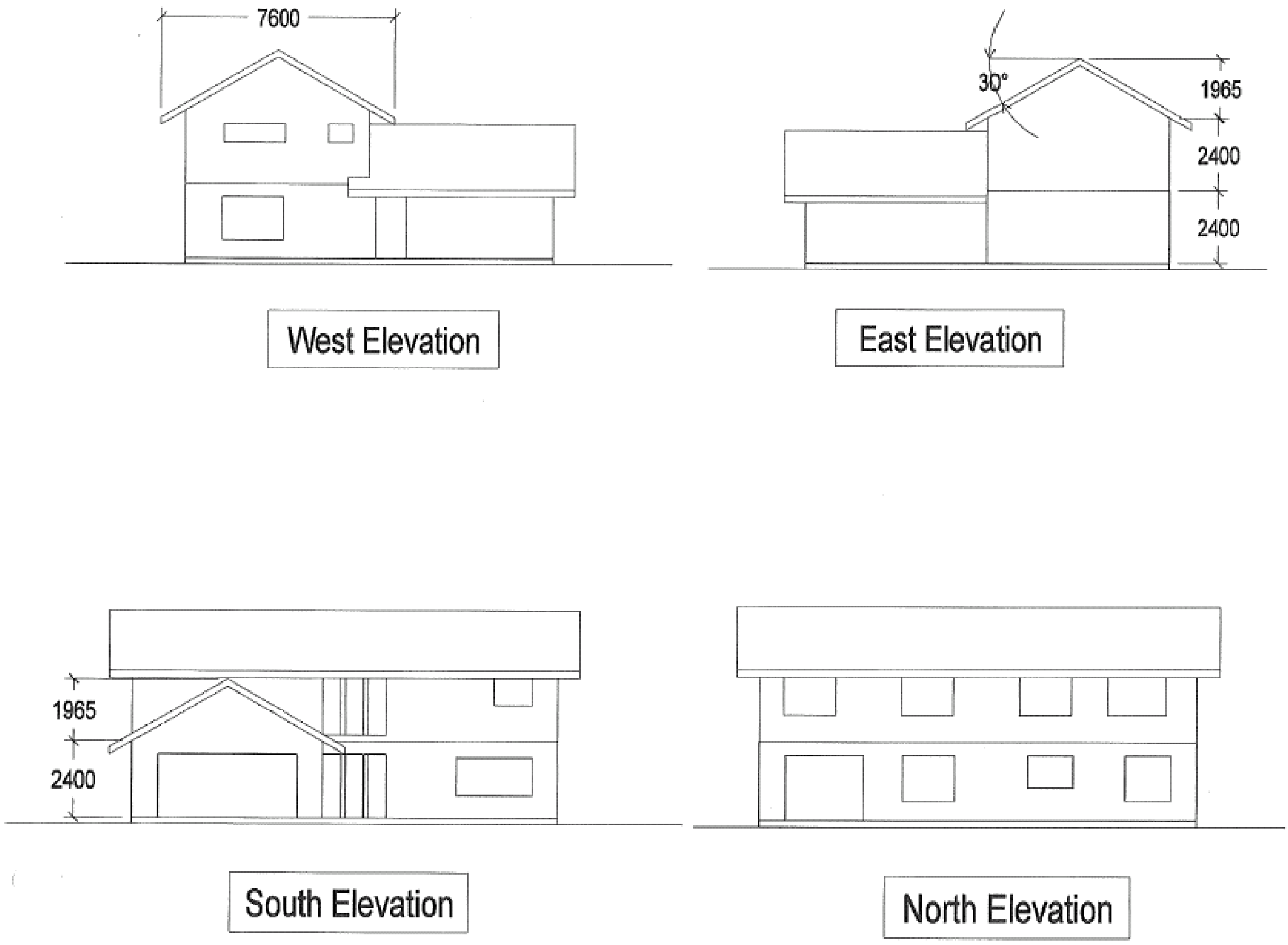


Figure 3.3.2: (Continued) Design Example 3

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 wind demand N-S 2037 BU's
	→ Total demand = 152 x 13.4 = 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 wind demand E-W 1248 BU's
	→ Total demand = 144 x 6 + 64 x 6 = 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

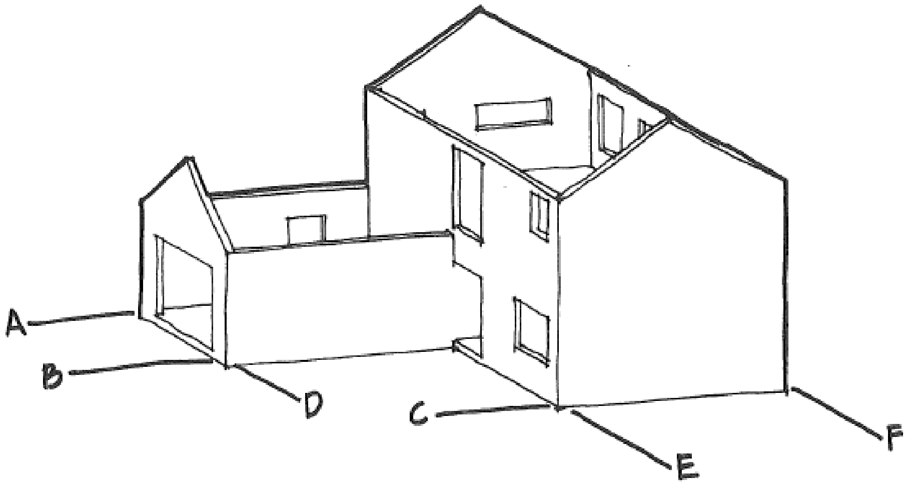


Figure 3.3.2(a): Bracing Lines for Design Example 3

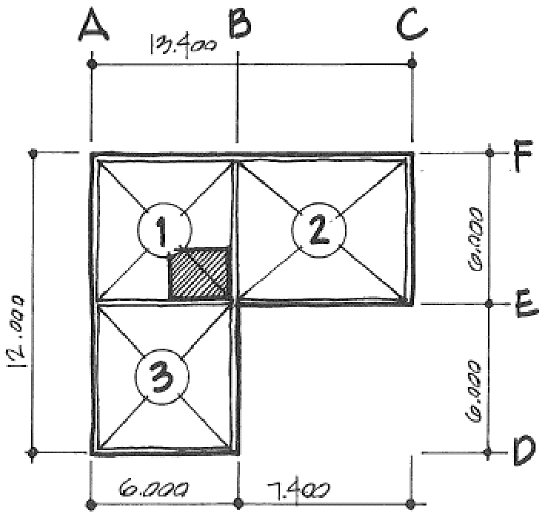


Figure 3.3.2(b): Bracing Lines and Diaphragm Positions

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).

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

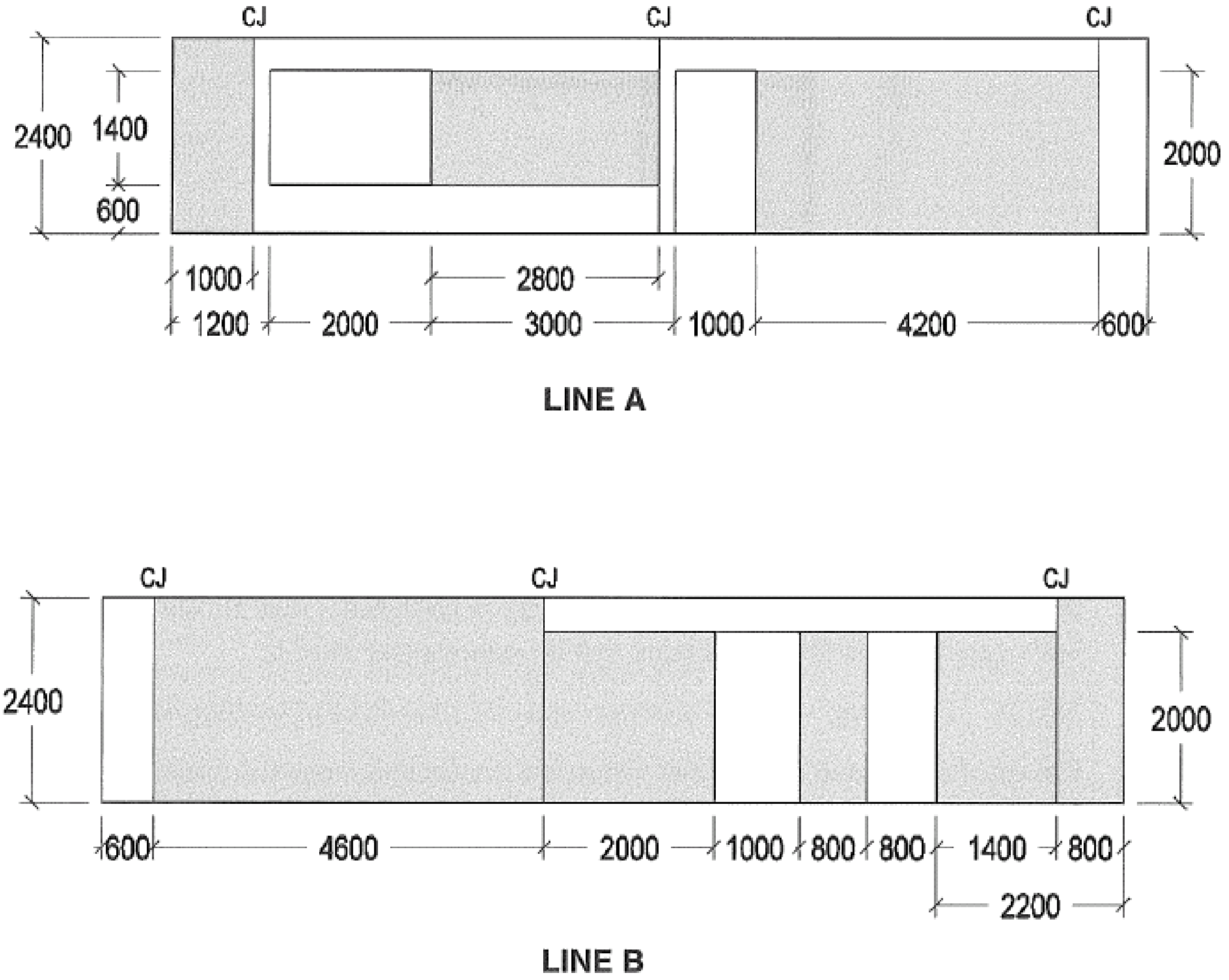


Figure 3.3.3: Bracing Panels for Design Example 3

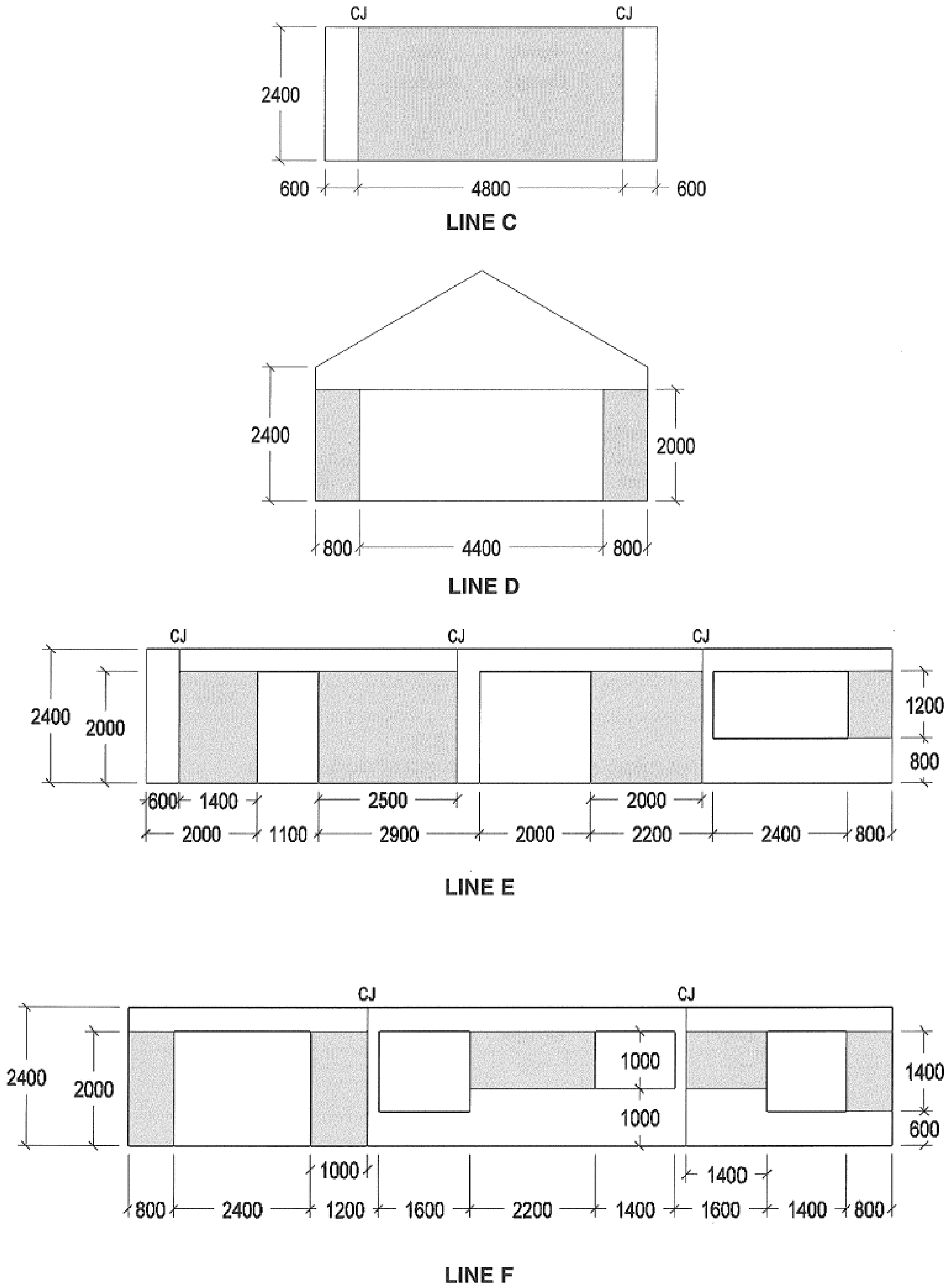


Figure 3.3.3: (Continued) Bracing Panels for Design Example 3

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 = $0.6 \times (28\,\mathrm{BU/m^2} \times 13.4\,\mathrm{m} \times 6\,\mathrm{m}) + (17\,\mathrm{BU/m^2} \times 6\,\mathrm{m} \times 6\,\mathrm{m})$ = 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.	

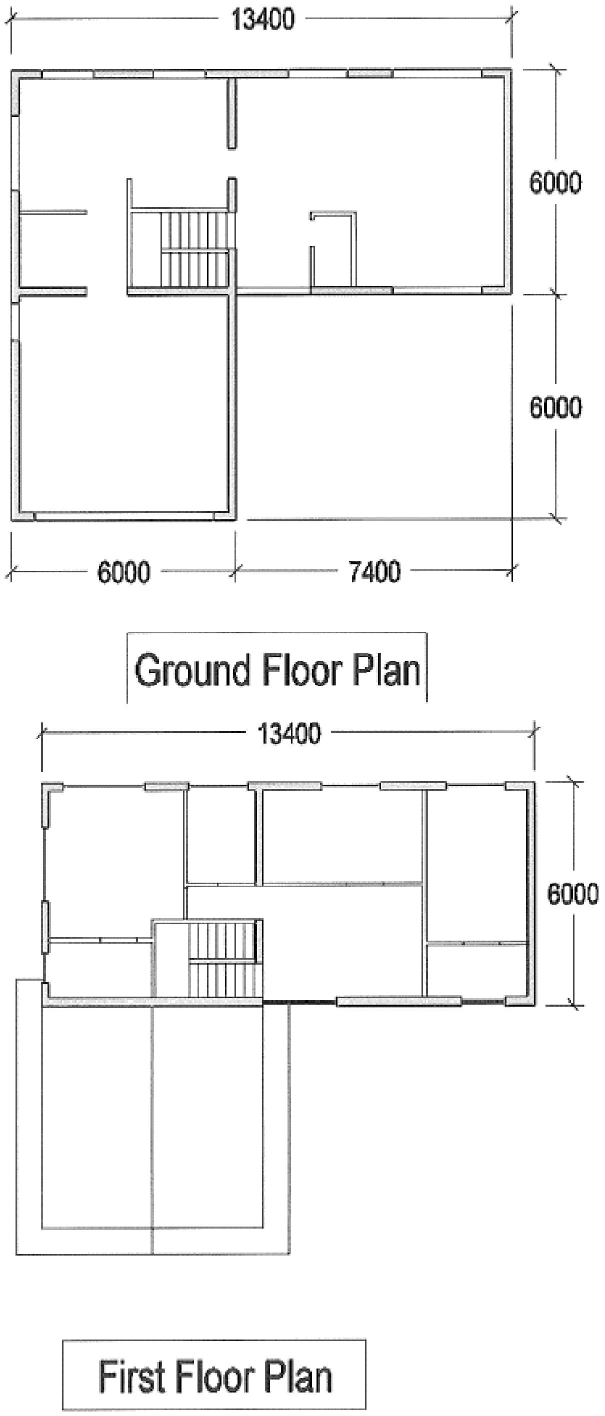


Figure 3.4.1: Design Example 4

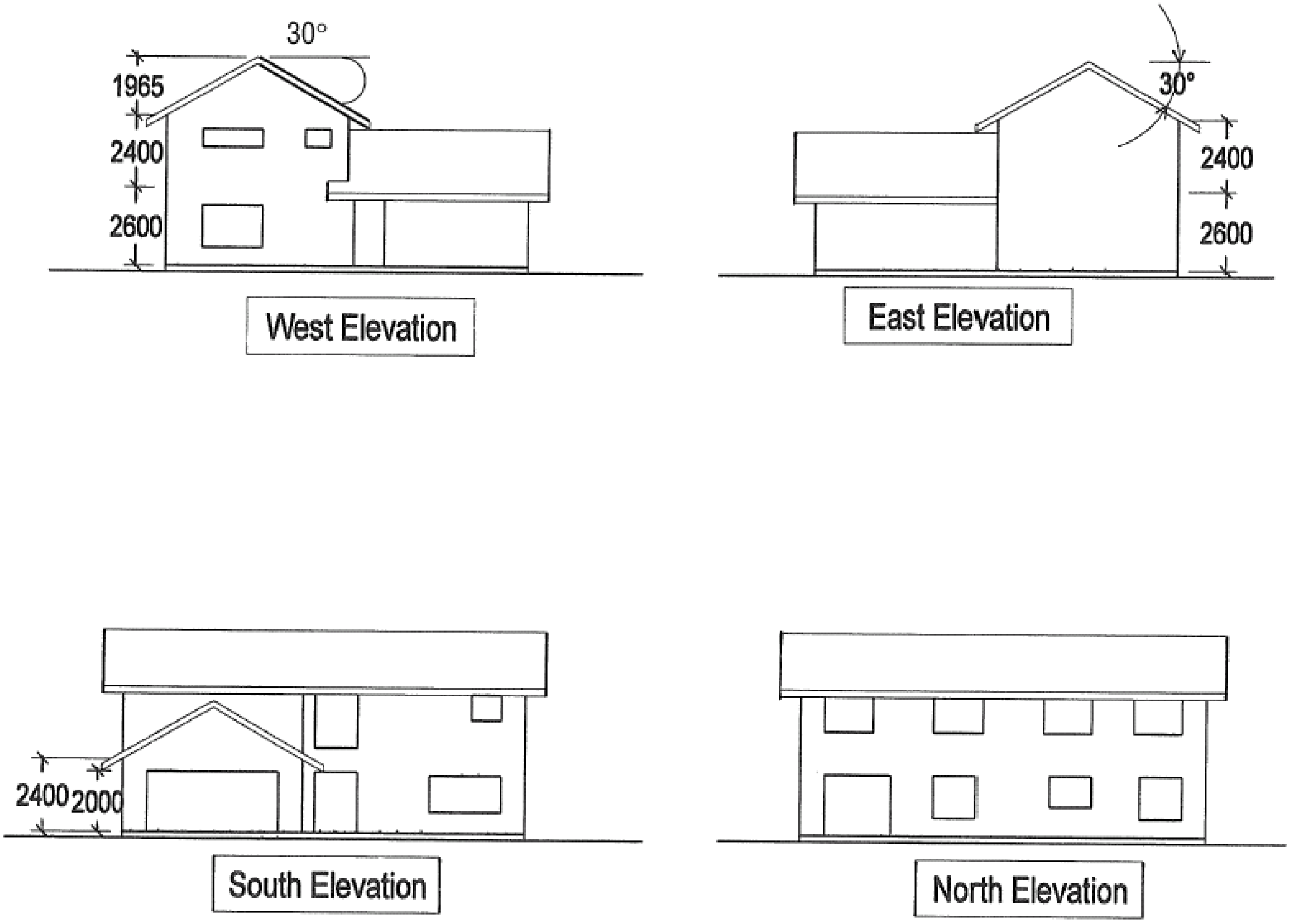


Figure 3.4.1: (Continued) Design Example 4

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.

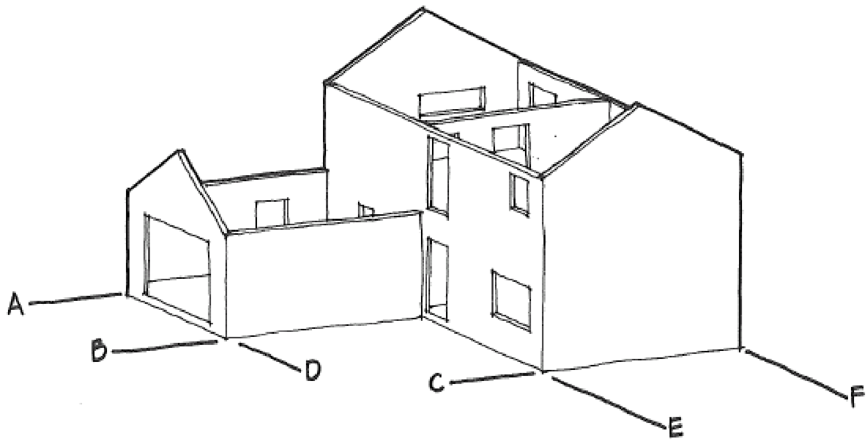


Figure 3.4.2(a): Bracing Lines for Design Example 4

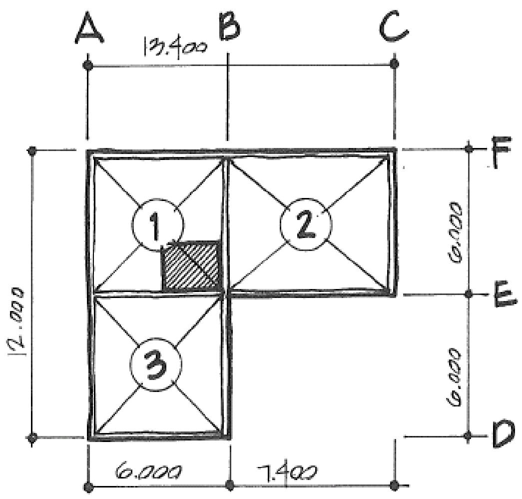


Figure 3.4.2(b): Bracing Lines Diaphragm Positions

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

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)

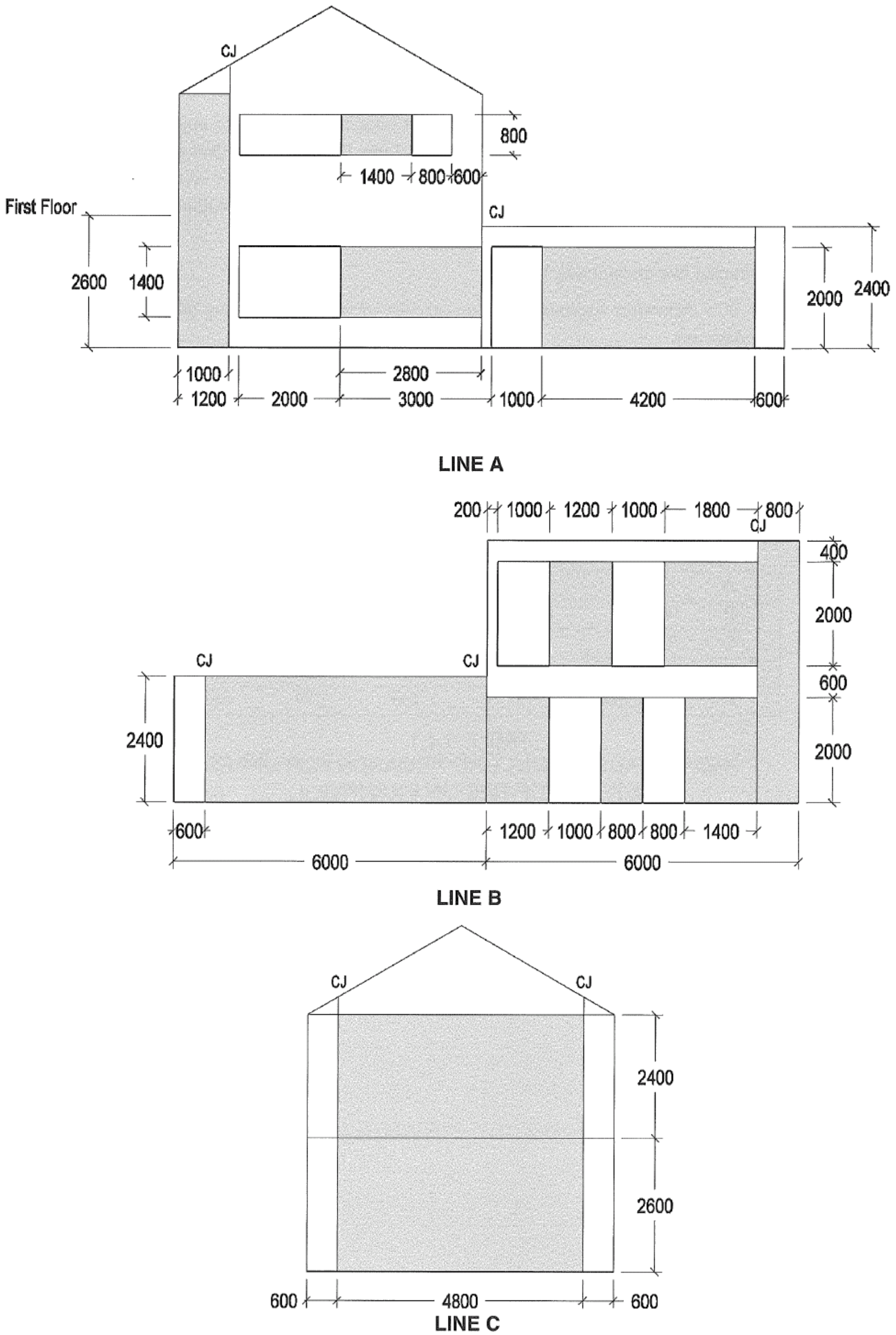
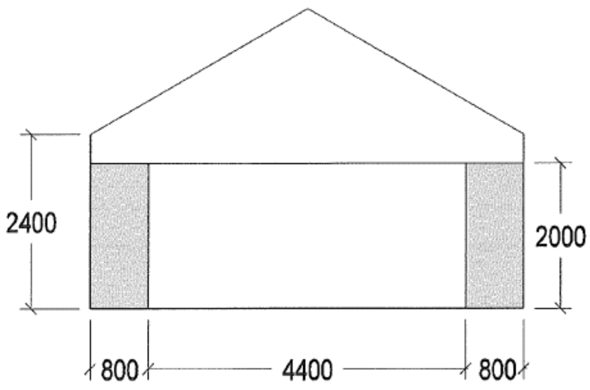
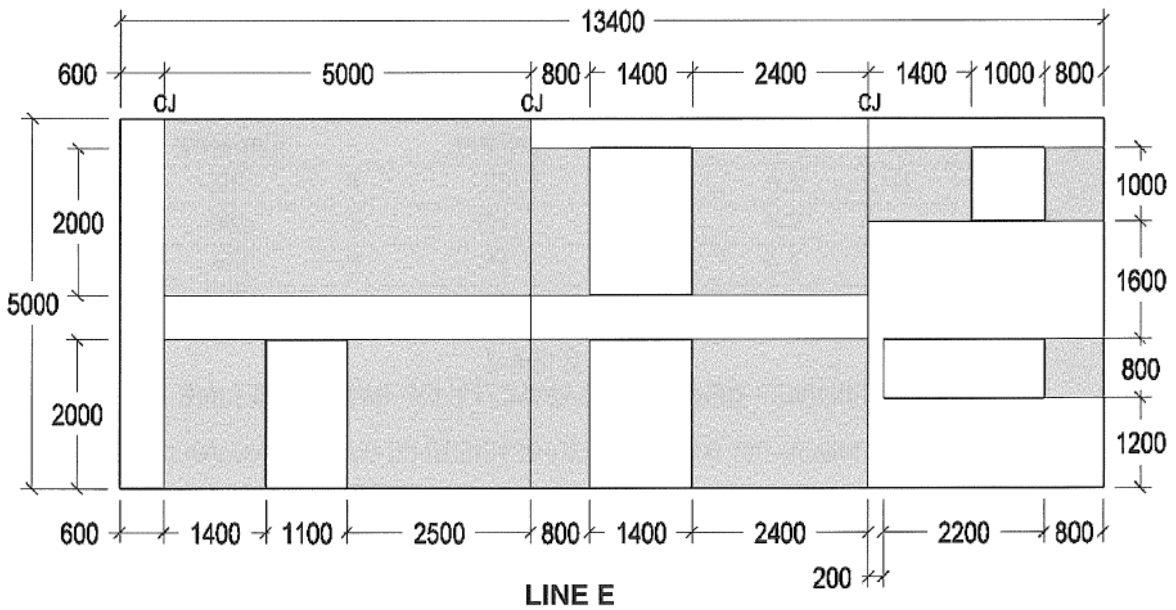


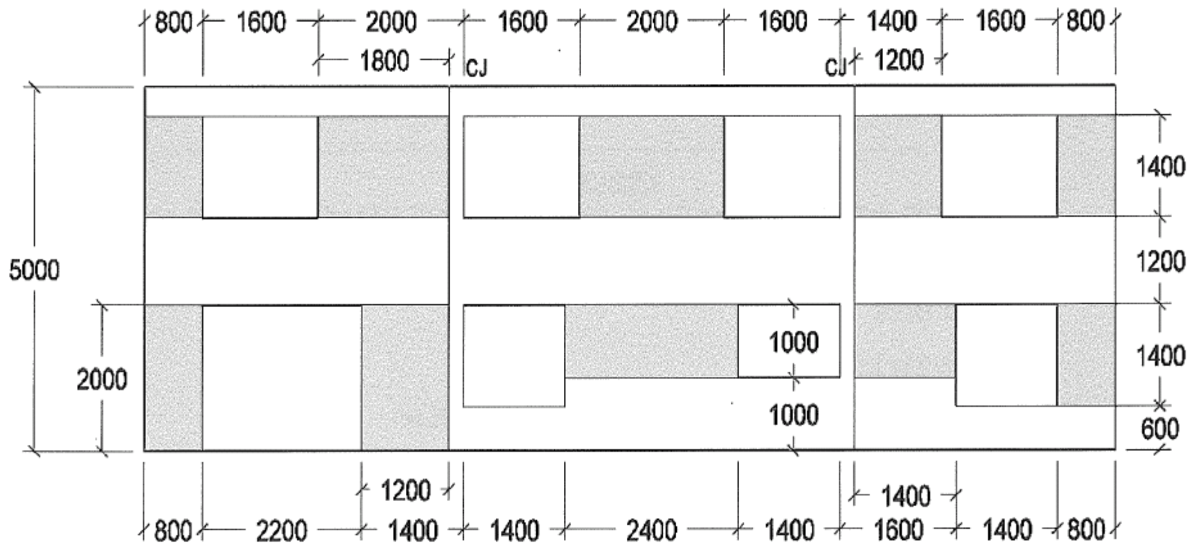
Figure 3.4.3: Bracing Panels for Design Example 4



LINE D



LINE E



LINE F

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	B2 bond beam at top of gable shaped walls B3 bond beam at top of wall (bracing lines A and C)
10.2.1	Note that bracing lines A and C are gable shaped walls (see Note 8 page 12).	
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
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
Figure 8.4
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

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

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

Footing:
width 300 mm
depth 200 mm
steel 2-D12, with R6 stirrups @ 600

Figure 6.5

See Figure 6.5 of the Standard for the footing cross-section detail.