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COMPARATIVE DESIGN OF REINFORCED CONCRETE SHEAR WALLS

By

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À mes parents,

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ABSTRACT

Code provisions for the determination of earthquake loads are intended to give reasonable estimate of the lateral forces that occur on a building as a result of an earthquake. Two major steps can be described in the procedure: the calculation of the base shear based on both the characteristics of the earthquake and the building, and the distribution of the base shear over the stories and the resisting earthquake elements of the building. Reinforced concrete ductile shear walls are the earthquake resisting elements considered in this study. Code provisions for the design of reinforced concrete ductile shear walls are intended to provide adequate reinforcement details and concrete strength to permit inelastic response under major earthquakes without critical damage or collapse.

The objective of this study is to provide, using an assumed building in Victoria, British Columbia, detailed description of the design procedures used by different design codes, and to compare the results obtained on the earthquake loads determination and on the reinforcement details provided to the shear walls. The NBCC-1995 and the IBC-2000 design code procedures were used to determine the earthquake design loads in Chapter 2, and the ACI318-99, the CSA-1995 and the NZS-1995 were used to design a reinforced concrete shear wall in Chapter 3. Comparative conclusions are presented Chapter 4.

Generally, design of reinforced concrete shear walls using Canadian, American, and New Zealand provisions should be done based on the earthquake loads obtained from code provisions of Canada, the United States, and New Zealand, namely the comprehensive provisions of NBCC-1995/CSA23.3-94, IBC-200/ACI318-99, and NZS:3101:1995 respectively. However, it was necessary in this study to use the same loads in the different reinforced concrete shear wall design procedures in order to make comparative conclusions more effective. Therefore, the earthquake loads obtained from NBCC-1995 provisions were exclusively used to do the three different design procedures of reinforced concrete design using the code provisions of ACI318-99, CSA23.3-94, and NZS:3101:part 1:1995 respectively. The choice was based on the fact that the location of the building is in Canada.

The fundamental assumptions that were made in this study include that: the building, described in Section 2.2, is be braced by reinforced concrete ductile shear wall systems, which means that the shear walls resisting system will resist 100% of the lateral forces resulting from an earthquake. The shear wall considered in the design has adequate foundation able to transmit 100% of all structural actions to the ground.

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CHAPTER 1

INTRODUCTION

Reinforced concrete shear walls are structural elements that are supposed to resist the assigned large fraction of, if not the entire, lateral forces on the building resulting from the horizontal base shear of the earthquake and the resulting horizontal shear force. For this reason, they have been called shear walls despite the fact that shear is only one of the phenomena that control their behavior and performance during an earthquake. Many recent references have been using the term structural walls to avoid the connotation that could be misleading. However, the term shear walls will be used in this study since it's the term that has been popularly common. Attention in the following discussion will be focused on slender shear walls. They have a height-to-width ratios greater than 2 but less than 4, and they generally behave like cantilever beams. Other types of shear walls include:

- short or squat walls that resist horizontal forces in their plane by a predominantly truss-type mechanism with the concrete providing the diagonal compressive struts and the steel reinforcement the equilibrating vertical and horizontal ties.
- Coupling beams resisting shear walls that are usually used when the height-to-width ratio is greater than 4. It's a system that combines the considerable stiffness of the wall with properly proportioned coupling beams that can provide an efficient energy dissipation mechanism.

To determine the earthquake loads, two provisions in this study were used. The NBCC-1995 and the IBC-2000 Provisions. Earthquake loads provisions of NBCC-1995 are given in Chapter 4. Clause 4.1.9 specifies the loading due to earthquake motion and gives the procedures to determine the seismic base shear and the lateral design forces.

Appendix C of NBCC contains the climatic and seismic information for building design in Canada. Earthquake design provisions of the IBC are provided in Chapter 16, sections 1614 through 1623. Seismic base shear and lateral force distribution procedures are investigated in details in sections 1617 and 1618. The base shear and the force distribution of lateral forces were calculated by both NBCC and IBC and compared. However, only the earthquake loads obtained from NBCC are used to do the design of reinforced concrete ductile shear wall in this study.

The aim of both provisions is to give a reasonable estimate of the lateral forces for which the building should be designed to resist. Such forces could be extremely large and acting on severe reverse action during a major earthquake, for this reason, the code provisions take into consideration the effective inelastic resistance of the structure. However, all the possible actions of an earthquake are carefully considered by the codes, and that includes: the peak ground spectral acceleration related to the seismic zone and the geological data available, the type of soils, the type of foundation, and the period of the structure. Earthquake loads include the determination of the seismic base shear which is the equivalent lateral force acting on the base of the building and directly related to its weight, and the distribution of the lateral forces on the stories. The determination of the story shears gives the lateral shear forces, torsions, and overturning moments on every earthquake resisting elements of the structure.

To do in here seismic reinforced concrete detailing, three provisions are used: ACI318-99, CSA23.3-94, and the NZS:3101:part 1:1995. In general, according to the three codes, the forces considered in the design are the:

- the shear force over the height of the structure; maximum at the base
- the flexure that is due to the overturning moment also maximum at the base
- Gravity loads due to the assigned dead and live loads.

Provisions of ACI for seismic reinforced concrete design are found in Chapter 21 of ACI318-99. Shear walls, or structural walls are designed according to Section 21.6, some requirements of Chapters 7, 10, and 14 also apply. The basic criteria that ACI aim to satisfy are stiffness, strength, and ductility. According to ACI, shear walls are assumed to behave like vertical cantilever beams that must have the strength to resist flexure and shear as well as axial loads caused by the earthquake moments and gravity loads. Shear walls also stiffen the building against excessive lateral displacement, and must have enough ductility to resist inelastic stresses and deformations without total failure that is crushing in the region of the formation of plastic hinges or fracture of the yielded steel bars.

The first thing to do according to ACI procedure is to check whether boundary elements are required. If the maximum extreme compressive fiber stress exceeds $0.2f_c'$, boundary elements are required. Boundary elements are regions at the extremities of the shear wall to protect it against damage caused by load reversals and crushing in the plastic hinge region. Boundary elements are reinforced as columns with confined vertical reinforcement. The use of boundary elements at the edges of shear walls is believed to be very effective to resist axial loads and the combination of axial loads and bending moments at the base of the shear wall.

The CSA Provisions for seismic reinforced concrete detailing are found in Chapter 21 of CSA Standard A23.3-94. Shear walls, or ductile flexural walls are designed according to Section 21.5, some requirements of Chapters 7, and 14 also apply. The NZS provisions for seismic design of shear walls are found in NZS 3101:part 1:1995, Section 12. Some requirements of Sections 7, and 8 also apply. One of the major consideration of the CSA and NZS is to design the shear walls to behave in a ductile manner, and to accommodate yielding of the flexural reinforcement in the plastic hinge region. This is generally achieved by providing moment and shear capacity in excess of that corresponding to the elastic strength capacity called the probable moment and the probable shear in the CSA code, and the overstrength moment and shear in the NZS code. Both codes provide procedures to evaluate the ductility requirements that are directly related to the overstrength factor and the depth of the neutral axis of the shear wall section. The overstrength factor is the ratio of the probable moment to the resisting moment.

An important goal of both code procedures, is to avoid failure in the wall web due to shear before the formation of plastic hinges at the base of the boundary elements. For this reason, the design generally provides shear capacity of the wall to resist the overstrength shear rather than the shear determined from earthquake loads. To ensure the formation of the plastic hinge region in the boundary elements first, the shear wall is designed in flexure like a cantilever beam by providing a resisting moment capacity to resist the earthquake design moment, and the effects of axial loads.

CHAPTER 2

EARTHQUAKE LOADS

2.1 Overview

In order to design a reinforced concrete shear wall to withstand an earthquake, the forces on the structure must be determined. The seismic loads of an earthquake depend on many factors that includes the size and weight and characteristics of the building, the type of soil and other characteristics of the earthquake. The characteristics of the earthquake are directly related to the magnitude of the forces that will occur on the structure, they include factors like the site geological location, the ground acceleration, the earthquake intensity and the distance from the epicenter. All those factors are included in the specification of the earthquake design loads by various design provisions.

In the discussion which follows, the two following seismic design code provisions are included:

1. *The National Building Code of Canada,* issued by the Canadian Commission on Building and Fire Codes of the National Research Council of Canada, 1995, referred to as NBCC-1995
2. *The International Building Code,* issued by the International Code Council, 1998, referred to as IBC-2000

2.2 Description of building and design data

An eight story cast in place reinforced concrete building with 7 – 7 m bays in the E-W direction and 3 – 7 m bays in the N-S direction and total height 30.4m is located in Victoria, B.C.

The building has 500×500 mm constant cross section square columns as shown in Figure 1-1, and a two-way spanning waffle floors with a module of 1750 mm and a slab thickness of 150 mm as shown in Figure 1-2.

Material Properties:

- Concrete

  Normal density concrete with

  $f'_c = 30$ MPa

  $w_c = 24$ kN/m^3

- Reinforcement

  $f_y = 400$ MPa

Service loads:

- Floor live load

  2.4 kN/m^2

  (from CPCA Table 1.13, Live load for office building.)

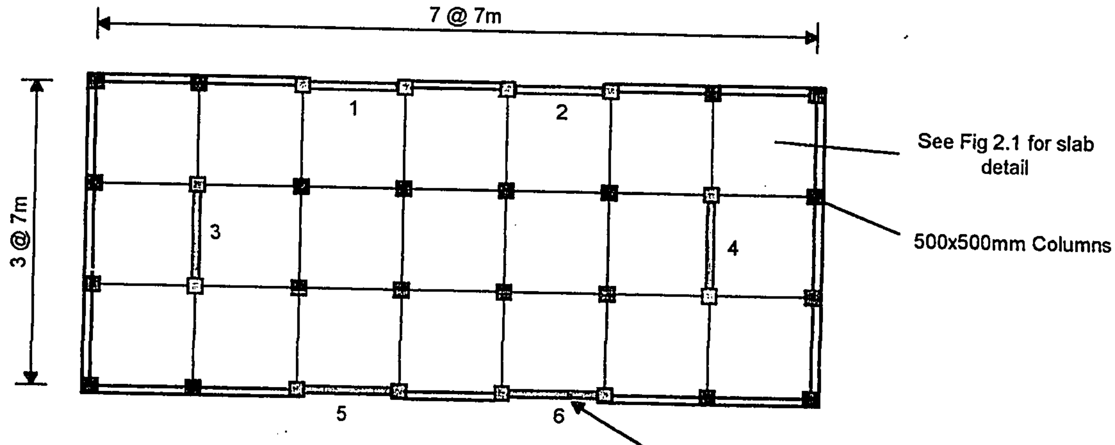
- Dead loads

  Self-weight of reinforced concrete members calculated at 24 kN/m^3

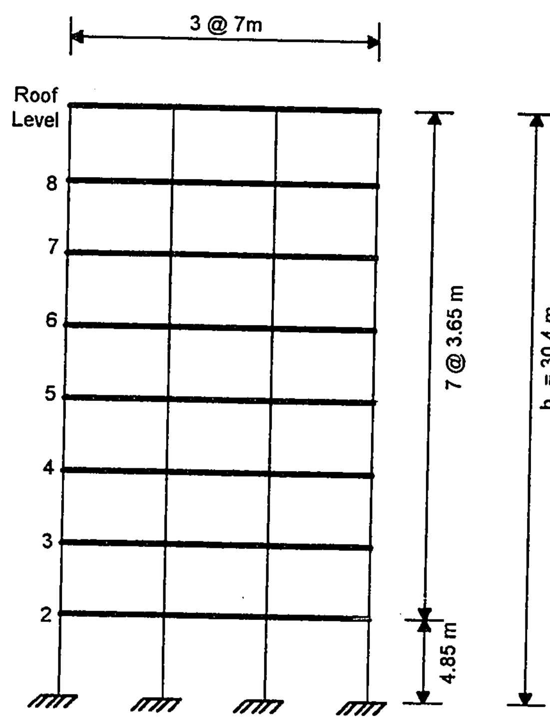
  1.0 kN/m^2 partitions walls on all floors

  0.5 kN/m^2 mechanical services

  0.5 kN/m^2 roof insulation



(a)



(b)

Figure 2-1 Structure considered in design example. (a) Typical floor plan. (b) Longitudinal section

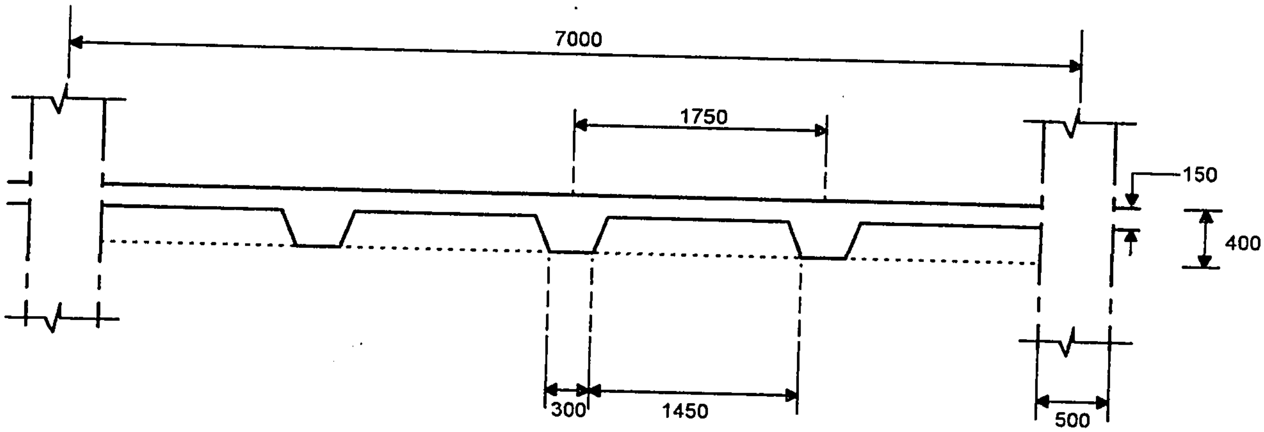


Figure 2-2 Slab span section between two columns in both directions (Dimensions in mm)

2.3 Determination of design lateral forces using NBCC-1995

2.3.1 Overview and determination of the seismic base shear

The National Building Code of Canada, NBCC-1995, contains the necessary data to determine earthquake loads and gives the procedures to determine the seismic base shear and the lateral design forces. Clause 4.1.9 of NBCC specifies the loading due to earthquake motion, and Appendix C of NBCC contains the climatic and seismic information for building design in Canada. (See Appendix A.)

The structure of Figure 2-1 is located in Victoria, the acceleration-related seismic zone, Z_a, is 6, the velocity-related seismic zone, Z_v , is 4, and the zonal velocity ratio v, is 0.3 as giving by Appendix C.

According to Clause 4.1.9.4, the minimum lateral seismic force, V, also known as lateral base shear, shall be calculated in accordance with the following formula

$$V = (V_e / R)U$$

where

U = 0.6, and R is the modification factor that shall conform to Table 4.1.9.1.B.

V_e is defined by Clause 4.1.9.5, as the equivalent lateral seismic force, representing elastic response, and it equals to v.S.I.F.W

where

v is the zonal velocity ratio,

S is the seismic response factor that shall conform to Table 4.1.9.1.A,

I is the seismic importance factor given by Clause 4.1.9.10 equal to: 1.5 for post-disaster buildings, 1.3 for schools and 1.0 for all other buildings,

F is the foundation factor that shall conform to Table 4.1.9.1.C, W is the weight that includes the dead load plus 25% of the design snow load plus 60% of the storage load for areas used for storage and the full content of any tanks as defined by NBCC.

In this example, according to NBCC 4.1.9.1.10, the importance factor I is 1.0 since this is an office building, and the foundation factor F is taken to be 1.0 assuming that the building is founded on rock conforming to Table 4.1.9.1.C.

The building is a reinforced concrete structure and it will be assumed that the shear walls will resist 100% of the lateral force. Therefore, the structure will be considered as a ductile wall system and R will be taken as equals to 3.5 according to NBCC Table 4.1.9.1.B.

The seismic response factor, S, is dependent on, T, the approximate fundamental period of the structure as given in Table 4.1.9.1.A of NBCC. Clause 4.1.9.7.a gives the expression of T for moment resisting frame structures and Clause 4.1.9.7.b gives the expression of T for other structures which the one used in this example since we have a reinforced concrete shear wall structure.

Therefore T equals $0.09h_n/(D_s)^{0.5}$, where h_n is the total height of the building, and D_s is the length of the wall which constitutes the main lateral-force-resisting system in the direction parallel to the applied force.

In this example, h_n equals to 30.4 meters and the length of shear walls including adjacent columns D_s is 15 meters in the E-W direction and 7.5 meters in the N-S direction.

Therefore, in the E-W direction:

$$T = \left(\frac{0.09 \times 30.4}{\sqrt{15}}\right) = 0.71 \sec$$

Since T>0.5, the seismic response factor is determined according to Table 4.1.9.1.A. as:

$S = 1.5/T^{0.5}$

$= 1.5/0.71^{0.5} = 1.785$

Similarly, in the N-S direction we:

$$T = \left(\frac{0.09 \times 30.4}{\sqrt{7.5}}\right) = 1.0\,\text{sec}$$

and

S = 1.501 sec

Let V to be V_x in the E-W direction and V_y in the N-S direction.

The seismic base shear is therefore calculated as follows:

$$V = \frac{vSIFW}{R}U$$

$$V_x = \frac{0.3 \times 1.785 \times 1 \times 1 \times W}{3.5} \times 0.6 = 0.0918W$$

$$V_y = \frac{0.3 \times 1.501 \times 1 \times 1 \times W}{3.5} \times 0.6 = 0.0772W$$

2.3.2 Weight of the building

As shown in Figure 1-1 (b), the story height is 4.85 m for the first story and 3.65 m for all other stories.

- For the first story,

Weight of the columns

$$0.5\times 0.5\times\left(\frac{3.25+4.45}{2}\right)\times 24\times 32=740kN$$

that includes half weight of the columns from the previous and following stories.

Weight of slab

From Figure 1.2 for the slab details, the slab concrete weight is calculated to be 5.76 kN/m^2.

As a result, the total dead load resulting from the slab is found to be:

$(5.76 + 1 + 0.5 + 0.5)\times 49.5\times 21.5 = 8259$ kN.

Weight of shear walls

The weight of walls parallel to the applied forces does not contribute to the diaphragm shears, only the shear walls in the direction perpendicular to the force are included (Naeim, 1989). For each story, half of the weight of the shear walls in each of the bottom and top stories is considered similar to the way the weight of columns is calculated.

The clear height of the shear walls in the first story is 4.45 meters, that is 4.85 minus the depth of the slab beams. Similarly, the clear height of the shear walls in the other stories is 3.25 meters. The shear walls are 300 mm thick and 6.5 meters long without including the boundary elements that were included in the calculation of the weight of the columns.

Therefore, the weight of one shear wall without the boundary elements at the ground level is

$4.45\times 6.5\times 0.3\times 24 = 208$ kN,

and the weight of one shear wall at the other story levels is

$3.25\times 6.5\times 0.3\times 24 = 152$ kN,

Hence, the weight of shear walls in the E-W direction is for the first story is

$(208.26\times 4)/2 + (152.1\times 4)/2 = 720$ kN

and in the N-S direction:

$(208.26\times 2)/2 + (152.1\times 2)/2 = 360$ kN

Therefore

$W_{1x} = 740 + 8259 + 720 = 9719$ kN

$W_{1y} = 740 + 8259 + 360 = 9359$ kN

- For the other stories,

weight of columns

$0.5 \times 0.5 \times 3.25 \times 24 \times 32 = 624$ kN

weight of shear walls in the E-W direction is $152.1 \times 4 = 608$ kN

weight of shear walls in the N-S direction is $152.1 \times 2 = 304$ kN

$W_{2x} = 624 + 8259 + 608 = 9491$ kN

$W_{2y} = 624 + 8259 + 304 = 9187$ kN

- For the roof level story,

weight of columns

only include half the weight of the columns is included in this case

$624 / 2 = 312$ kN,

weight of slab

the dead load from the partition walls is not included in this case

$(5.78 + 0.5 + 0.5) \times 49.5 \times 21.5 = 7216$ kN,

weight of shear walls in the E-W direction is $608.4/2 = 304$ kN

weight of shear walls in the N-S direction is $304.2/2 = 152$ kN.

Snow load

NBCC 4.1.7.1 specifies the loading, S, due to snow accumulation on a roof to be determined from the formula

$S = S_s(C_bC_wC_sC_a) + S_r$

where

S_s is the ground snow load in kPa, given in Appendix C of NBCC

S_r is the associated rain load in kPa, given in Appendix C but not greater than $S_s(C_bC_wC_sC_a)$ as mentioned by section 4.1.7.1 of NBCC

C_b is the basic roof snow factor of 0.8

C_w is the wind exposure factor that shall be taken as 1.0 except for certain conditions described in 4.1.7.3 such as when the building to wind loading equally and with no obstruction, C_w is reduced to 0.75, or when the snow loading does not involve accumulation of snow due to drifting from adjacent surfaces.

C_s is the slope factor that shall be taken as 1.0 if the slope of the roof is less than 30° according to section 4.1.7.4

C_a is the accumulation factor that shall be taken as 1.0 except for certain conditions described in section 4.1.7.7 related to the shape of non uniform roofs

For this building located in Victoria, Appendix C of NBCC 95 gives the values for the ground snow S_s to be 1.0 kN/m^2 and the associated rain S_r to be 0.2 kN/m^2.

Moreover,

$C_b = 0.8,\ C_w = 1.0,\ C_s = 1.0,$ and $C_a = 1.0$

Hence $S = 1.0\times(0.8\times1\times1\times1) + 0.2 = 1$ kN/m^2

As mentioned earlier, NBCC 4.1.9.1. states that 25% of the design snow load shall be included with the dead load in the calculation of W for seismic analysis.

So finally the snow load considered is calculated to be:

$1\times49.5\times21.5\times0.25 = 266$ kN

Finally:

$W_{rx} = 312 + 7216 + 266 + 304 = 8098$ kN

$W_{ry} = 312 + 7216 + 266 + 152 = 7946$ kN

The total weight of the building considered in the seismic design in the E-W direction is

$W_x = W_{1x} + 6\times W_{2x} + W_{rx} = 9719 + 6\times 9491 + 8098 = 74762$ kN

and the weight in the N-S direction is

$W_y = W_{1y} + 6\times W_{2y} + W_{ry} = 9359 + 6\times 9187 + 7946 = 72424$ kN

Finally the seismic base shear are found to be:

$V_x = 0.0918W_x = 0.0918 \times 74762 = 6863$ kN

$V_y = 0.0772W_y = 0.0772 \times 72424 = 5591$ kN

2.3.3 Seismic lateral forces

The total base shear is distributed to the floors using Tables 2-1 and 2-2.

Where:

$\phi_i = (h_i W_i)/(\Sigma h_i W_i)$,

F_i is the lateral earthquake load equals the seismic base shear times ϕ_i,

V_i is the sum of all the seismic lateral forces introduced above the assumed level.

V_i is the design shear at each level, and obviously, V_i at the base level is equal to the seismic base shear.

The seismic loads analysis according to all codes requires consideration of any torsional loads that could occur at each story during an earthquake. Table 2-3 represents the calculation of the centre of mass for the base level story in both direction. x represents the E-W direction and y represents the N-S direction. x and y are the distance from the centre of the shear walls to the outside face parallel face of the building.

Table 2-1 Story lateral forces and shears in the E-W Direction where $V_x = 6863$ kN (NBCC provisions)

Floor No.	h_i, m	W_i, kN	h_iW_i	ϕ_i	F_i	V_{ix}
Roof	30.4	8098	246175	0.1898	1303	1303
8	26.75	9491	253884	0.1957	1343	2646
7	23.1	9491	219242	0.1690	1160	3806
6	19.45	9491	184600	0.1423	977	4783
5	15.8	9491	149957	0.1156	794	5577
4	12.15	9491	115315	0.0889	610	6187
3	8.5	9491	80673	0.0622	427	6614
2	4.85	9719	47135	0.0363	249	6863
Σ		74762	1296981		6863	

Table 2-2 Story lateral forces and shears in the N-S Direction where $V_y = 5591$ kN (NBCC provisions)

Floor No.	h_i, m	W_i, kN	h_iW_i	ϕ_i	F_i	V_{iy}
Roof	30.4	7946	241551	0.1919	1073	1073
8	26.75	9187	245746	0.1953	1092	2165
7	23.1	9187	212215	0.1686	943	3108
6	19.45	9187	178683	0.1420	794	3902
5	15.8	9187	145151	0.1153	645	4547
4	12.15	9187	111619	0.0887	496	5043
3	8.5	9187	78088	0.0621	347	5390
2	4.85	9359	45387	0.0361	202	5591
Σ		72424	1258440		5591	

Table 2-3 Center-of-Mass Calculations

Wall No.	Wall Weight	Dir.	x_i, m	x_iW_i	y_i, m	y_iW_i
6	208.26	x	17.75	3696.62	21.25	4425.53
5	208.26	x	31.75	6612.26	21.25	4425.53
4	208.26	y	7.25	1509.89	10.75	2238.80
3	208.26	y	42.25	8798.99	10.75	2238.80
2	208.26	x	17.75	3696.62	0.25	52.07
1	208.26	x	31.75	6612.26	0.25	52.07
Σ	1249.56			30926.61		13432.77

The centre of mass of the walls is located at

$$x_1 = \frac{\sum xW}{\sum W} = \frac{30926.61}{1249.56} = 24.75m$$
$$y_1 = \frac{\sum yW}{\sum W} = \frac{13432.77}{1249.56} = 10.75m$$

Obviously, the centre of mass of the walls and the floor coincide with the floor geometric centroid due to the symmetry and the uniform slab thickness and wall weights. Therefore, the combined centre of mass of the base level floor is located at:
$x_m = 24.75$ m, and $y_m = 10.75$ m.

In order to calculate the centre of rigidity, the relative rigidities, R, of each wall are calculated in Table 2-4. R is equal to $1/\Delta$ where Δ is the lateral drift of the wall for a lateral force P as shown is Figure 2-3. The relative wall rigidities, R, may be computed assuming a constant value of P, say P = 100 kN.

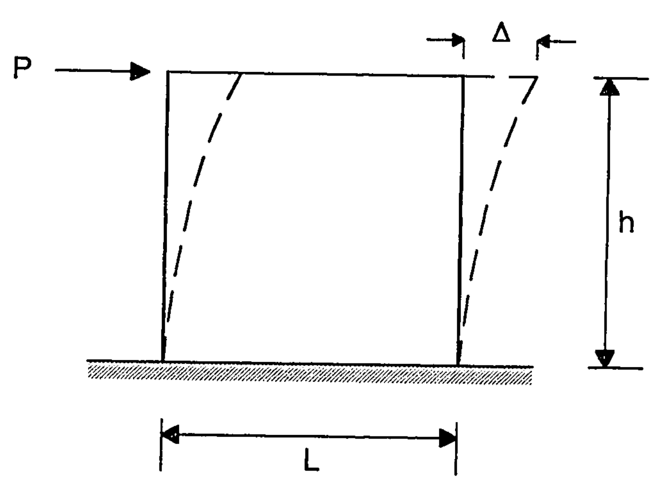


Figure 2-3 Deformation of a free-standing wall panel.

The drift is calculated as:

$$\Delta = \frac{Ph^3}{3EI} + \frac{1.2Ph}{AG}$$

where E is the modulus of elasticity of concrete equals to $4500\sqrt{f_c'}$ according to Clause 8.6.2.3 of CSA Standard A23.3-94.

Therefore, $E = 4500\sqrt{30} = 24.6$ GPa.

Taking t as the thickness as the wall, and assuming G = 0.4E, the above expression of Δ may be rewritten as:

$$\Delta = \frac{4P(h/L)^3}{3Et} + \frac{3P(h/L)}{Et}$$

Table 2-4 Relative Rigidity of the Walls

Wall No.	Height	Length	H/L	E, MPa	t	P, N	Δ, m	R = 1/Δ
1	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156
2	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156
3	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156
4	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156
5	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156
6	4.45	6.5	0.68462	24647515	0.3	100000	0.04513	22.156

Table 2-5 Centre-of-Rigidity Calculations

Wall No.	Dir.	x, m	y, m	R_x	R_y	xR_y	yR_x
1	x	-	21.25	22.156	-	-	470.815
2	x	-	21.25	22.156	-	-	470.815
3	y	7.25	-	-	22.156	160.63	-
4	y	42.25	-	-	22.156	936.09	-
5	x	-	0.25	22.156	-	-	5.539
6	x	-	0.25	22.156	-	-	5.539
Total				88.62	44.312	1096.72	952.708

Using the parameters generated in Tables 2-4 and 2-5, the location of the centre of rigidity is established as:

$$x_r = \frac{\sum xR_y}{\sum R_y} = \frac{1096.72}{44.312} = 24.75m$$

$$y_r = \frac{\sum yR_x}{\sum R_x} = \frac{952.708}{88.62} = 10.75m$$

As expected, the centre of rigidity coincides with the centre of mass and the geometric centroid because of the perfect symmetry of the floor.

The torsional eccentricity is defined as:

$e_x = x_r - x_m$

$e_y = y_r - y_m$

Obviously in this case, the torsional eccentricity is zero. However, Commentary J in part 4 of NBC Structural Commentaries, requires a minimum accidental eccentricity equal to 0.1 times the plan dimension perpendicular to the direction of loading.

Therefore

$e_x = 0.1 \times 49.5 = 4.95$ m

$e_y = 0.1 \times 21.5 = 2.15$ m

As a result, the torsional moments are computed as follows:

$T_y = V_y e_x = 5591 \times 4.95 = 27676$ kN.m

$T_x = V_x e_y = 6863 \times 2.15 = 14756$ kN.m

The in-plane forces in the walls due to direct shear F_v and the in-plane forces due to torsion F_t are computed in Tables 2-6 and 2-7 using the following expressions:

$$F_{vx} = V_x \frac{R_x}{\sum R_x}$$

$$F_{vy} = V_y \frac{R_y}{\sum R_y}$$

and,

$$F_{tx} = T_x \frac{Rd}{\sum Rd^2}$$

$$F_{ty} = T_y \frac{Rd}{\sum Rd^2}$$

Table 2-6 Wall Shears for Seismic force in the E-W Direction: $V_x = 6863$ kN, $T_x = 14756$ kN.m

Wall No.	Dir	R_x	R_y	d_x, m	d_y, m	Rd	Rd^2	F_v, kN	F_t, kN	F_{total}, kN
1	x	22.156	-	-	10.50	232.638	2442.70	1715.8	147.07	1862.87
2	x	22.156	-	-	10.50	232.638	2442.70	1715.8	147.07	1862.87
3	y	-	22.156	-17.5	-	-387.730	6785.28	-	-245.11	-245.11
4	y	-	22.156	17.5	-	387.730	6785.28	-	245.11	245.11
5	x	22.156	-	-	-10.50	-232.638	2442.70	1715.8	-147.07	1715.80
6	x	22.156	-	-	-10.50	-232.638	2442.70	1715.8	-147.07	1715.80
Σ		88.62	44.312				23341.35			

Table 2-7 Wall Shears for Seismic force in the N-S Direction: $V_y = 5591$ kN, $T_y = 27676$ kN.m

Wall No.	Dir	R_x	R_y	d_x, m	d_y, m	Rd	Rd^2	F_v, kN	F_t, kN	F_{total}, kN
1	x	22.156	-	-	10.50	232.638	2442.70	-	275.84	275.84
2	x	22.156	-	-	10.50	232.638	2442.70	-	275.84	275.84
3	y	-	22.156	-17.5	-	-387.730	6785.28	2795.6	-459.74	2795.60
4	y	-	22.156	17.5	-	387.730	6785.28	2795.6	459.74	3255.34
5	x	22.156	-	-	-10.50	-232.638	2442.70	-	-275.84	-275.84
6	x	22.156	-	-	-10.50	-232.638	2442.70	-	-275.84	-275.84
Σ		88.62	44.312				23341.35			

F_{total} is computed in Tables 2-6 and 2-7, as the sum of F_v and F_t, and as F_v alone when F_t is negative. F_{total} acting in the direction parallel to the shear wall is considered to be the design shear for that wall as shown in Table 2-8.

Table 2-8 Design Shear forces V_u

Wall No.	V_u, kN	Direction
1	1862.87	E-W
2	1862.87	E-W
3	2795.60	N-S
4	3255.34	N-S
5	1715.80	E-W
6	1715.80	E-W

The overturning moment on a wall on a particular level is defined to be the sum of moments of the story forces above that level. Hence, it is given by the following expression:

$$M_x = \sum_{i=x}^{n} F_i\left(h_i - h_x\right)$$

Where, n, is the number of stories and, F_i, is the lateral force on the wall calculated as in Table 2-7 for F_v, the effects of torsion are not included in the calculation of the overturning moment. Table 2-9 represents the overturning moment for Wall No. 4.

Table 2-9 Overturning Moment for Wall No. 4

Floor	hi, m	Fi	F on Wall	M
roof	30.4	1073.20	536.6	
8	26.75	1091.84	545.9	1958.59
7	23.1	942.86	471.4	5909.78
6	19.45	793.88	396.9	11581.68
5	15.8	644.90	322.4	18702.42
4	12.15	495.92	248.0	27000.09
3	8.5	346.94	173.5	36202.81
2	4.85	201.65	100.8	46038.70
Σ		5591	2796	59597

It was noticed from the calculations of Tables 2-6 and 2-7 that F_v, the story shear, is equally divided among the shear walls acting in the direction of the story shear, which was expected due to the symmetrical distribution of the shear walls and the symmetry in the floor itself. F_t is calculated on each floor following the same procedures described for the first floor. As a result, it can be concluded through similar calculation procedures, the design shears as well as the overturning moments for all walls on all stories as shown in Table 2-10 and 2-11

Table 2-10 Design Shear forces V_u (in kN) according to NBCC-1995 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	1863	1795	1679	1514	1298	1033	718	354
2	1863	1795	1679	1514	1298	1033	718	354
3	2796	2695	2521	2273	1951	1554	1083	537
4	3255	3138	2936	2647	2272	1809	1261	625
5	1716	1653	1547	1394	1196	952	662	326
6	1716	1653	1547	1394	1196	952	662	326

Table 2-11 Design Moments M_u (in kN.m) according to NBCC-1995 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	36532	28210	22175	16530	11441	7077	3603	1189
2	36532	28210	22175	16530	11441	7077	3603	1189
3	59597	46039	36203	27000	18702	11582	5910	1959
4	59597	46039	36203	27000	18702	11582	5910	1959
5	36532	28210	22175	16530	11441	7077	3603	1189
6	36532	28210	22175	16530	11441	7077	3603	1189

2.4 Determination of design lateral forces using IBC-2000

2.4.1 Seismic base shear

In 1997, the International Conference of Building Officials (ICBO) joined the effort of the Building Officials and Code Standards (BOCA) and the Southern Building Code Congress International (SBCCI) to prepare a comprehensive set of regulations and standards for building systems requirements: the International Building Code IBC 2000 which will replace the Uniform Building Code UBC.
Traditionally, the UBC code was the most used code in the United States and in many parts of the world. The UBC 1997, volume 2 Structural Engineering Design Provisions published by the International Conference of Building Officials (ICBO), specified the provisions of earthquake design in division IV of Chapter 16. However, the seismic detailing provisions for reinforced concrete structures are specified in Chapter 19 Section 1910.

Unlike the UBC, the IBC does not specify the detailed requirements of the concrete shear walls design, the ACI requirements are adopted instead. Section 1910.2.2.3 of IBC specifies: "Reinforced concrete shear walls are walls conforming to the requirements of ACI 318 exclusives of Chapters 21 and 22". Earthquake design provisions of the IBC are provided in Chapter 16 "Structural Design Requirements" namely in Sections 1614 through 1623. Earthquakes loads are investigated in detail in Sections 1617 and 1618. Section 1618 describes the seismic design of buildings using the Dynamic Analysis Procedure which is based on a dynamic analysis that requires the determination of the natural modes of vibration for the building including the period of each mode that leads to obtain the modal base shear. This study is limited to the methods discussed in Section 1617 leading to the minimum design lateral forces and related effects. Two methods can be used in accordance with section 1617: the Equivalent Lateral Force Procedure for Seismic Design of Building (Section 1617.4), and the Simplified Analysis Procedure for Seismic Design of Buildings (Section 1617.5).

Section 1617.4 describes the seismic design according to the Equivalent Lateral Force Procedure. In this method, the Seismic Base Shear, V, is given by section 1617.4.1 as

$V = C_s W$

where

W is the effective seismic weight of the structure that includes the total dead load and some additional loads resulting from storage areas and permanent equipment plus 20% of snow load instead of the 25% required by the NBCC. However, according to IBC and unlike the Canadian requirements of NBCC, the snow load shall be included only in the case where it exceeds 1.43 kN/m^2 (30 pcf), where in this example, the snow load was determined to be 1 kN/m^2. Therefore, the snow load will not be included with the weight of the building of this study.

C_s is the seismic response coefficient determined in section 1617.4.1.1 as follows:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

R is the response modification factor given in Table 1617.6 (See Appendix B)

I is the Occupancy Importance Factor related to the Seismic Use Group and given by Table 1604.5 in section 1616.2 of IBC

S_{DS} is the design spectral acceleration at short period determined by section 1615.1.3 as equal to $(2/3)S_{MS}$.

S_{MS} is defined to be the maximum Considered Earthquake spectral response acceleration for short period and is given by section 1615.2 as:

$S_{MS} = F_a S_s$,

where

S_s is the mapped spectral response acceleration for short periods given by the maps of Figures 1615(1) to (5)

F_a is the site coefficient given as a function of the Site Class and S_s in Table 1615.1.2

Section 1617.5 describes the seismic design according to the Simplified Analysis Procedure. In this method, the seismic base shear, V, is given by section 1617.5.1 as follows:

$$V = \frac{1.2 S_{DS}}{R} W$$

Obviously, this expression of the base shear gives a more conservative value than the one determined with the Equivalent Lateral Force Procedure since it doesn't take into consideration the Importance Factor and it has a factor of 1.2 .

Section 1616 describes the limitations of the use of each the two methods. The simplified analysis is permitted for structures in Seismic Use Group I, not exceeding two or three stories in height. In general, structures assigned to Seismic Design Category B or C shall be designed using the Equivalent Lateral Force Procedure.

The Seismic Use Group is related to the nature of occupancy and the associated importance factor given in Table 1604.5 of IBC.

In the example of this study, the Seismic Use Group is Group I, and the Seismic Importance Factor I, is equal to 1.0

The Seismic Design Categories A, B, C, D, E, and F are the Site Class related to the type of the soil, and are given in Table 1616.1. It was already assumed that the structure located in Victoria, is built on rock, therefore, the Site Class is B. As a result, the Equivalent Lateral Force Procedure will be used in this seismic design.

The mapped maximum considered earthquake spectral response accelerations at short periods, S_s, are given in the IBC for the United States only. Figure J-1 in Structural

Commentaries (Part 4) of NBCC gives the contour lines in Canada of peak ground accelerations, in units of g, having a probability of exceedance of 10% in 50 years. From this map, S_s, is determined as 0.32g for Victoria. (See Appendix B)

From Table 1615.1.2-1, the Site Coefficient F_a is found to be 1.0

From Table 1617.6, the response modification factor, R, is determined to be 5.5, for Special Reinforced concrete shear walls. This category of shear walls system was selected based on the assumption of the use of ductile shear walls in the design.

$S_{MS} = F_a S_s = 1.0 \times 0.32 \times 9.81 = 3.14 \text{ m/s}^2$

$S_{DS} = (2/3) S_{MS} = (2/3) \times 0.32 \times 9.81 = 2.09 \text{ m/s}^2$

Therefore,

$$C_s = \frac{2.09}{\left(\frac{5.5}{1.0}\right)} = 0.38$$

Section 1617.4.1.1 requires that the value of the seismic response coefficient C_s shall not be taken less than $0.0044 S_D$ and need not to exceed the following:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I}\right) T}$$

where

S_{D1} is the 1 second period design spectral response acceleration as defined in section 1615.1.3.

T is the fundamental period of the building determined by section 1617.4.2 as equal to $C_T (h_n)^{3/4}$

C_T is the Building Period Coefficient taken to be 0.049 (0.020 for English Units), for buildings other than moment resisting frame and braced frames buildings as described in section 1617.4.2.1

h_n is the height of the buildings.

There are no accurate data available for Victoria to determine S_1, the mapped spectral accelerations for 1 second period, therefore, the maximum possible value for S_1 from Table 1616.3-2 which is 0.133g for a building of category B and Seismic Use Group I will be used. (See Appendix B)

Therefore,

$T = 0.049 \times 30.4^{3/4} = 0.63$ sec

$S_{D1} = (2/3)S_{M1} = (2/3)F_aS_{S1} = (2/3) \times 1.0 \times 0.133 \times 9.81 = 0.869$ m/s^2

$$\frac{S_{D1}}{\left(\frac{R}{I}\right)T} = \frac{0.869}{\left(\frac{5.5}{1.0}\right) \times 0.63} = 0.25$$

As a result, C_s shall be taken as 0.25

Hence, the seismic base shear, V, is:

$V = 0.25W$

2.4.2 *Weight of the building*

W is calculated the same way as in the NBCC requirements, except that the snow load is not included as mentioned earlier in this Section. Therefore,

$W_{1x} = 9718$ kN

$W_{1y} = 9358$ kN

$W_{2x} = 9491$ kN

$W_{2y} = 9187$ kN

$W_{rx} = 312 + 7215.61 + 304.2 = 7832$ kN

$W_{ry} = 312 + 7215.61 + 152.1 = 7680$ kN

The total weight of the building considered in the seismic design in the E-W direction is

$W_x = W_{1x} + 6 \times W_{2x} + W_{rx} = 9718 + 6 \times 9491 + 7832 = 74496$ kN

and the weight in the N-S direction is

$W_y = W_{1y} + 6 \times W_{2y} + W_{ry} = 9358 + 6 \times 9187 + 7680 = 72160$ kN

Finally the seismic base shears are found to be

$V_x = 0.25 W_x = 0.25 \times 74496 = 18624$ kN

$V_y = 0.25 W_y = 0.25 \times 72160 = 18040$ kN

2.4.3 Lateral force distribution

The vertical distribution of seismic forces is described in section 1617.4.3 as follows:

$F_x = C_{vx} V$

F_x is the lateral force induced at any level of the structure

C_{vx} is the vertical distribution factor calculated as:

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Where k is a distribution exponent related to the building period as follows:

- For buildings having a period of 0.5 seconds or less, k = 1
- For buildings having a period of 2.5 seconds or more, k = 2
- For buildings having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2

In this example, since T = 0.63 seconds, k is assumed to be 2 for the sake of simplicity.

The accidental torsion required by the IBC code is half that is required by NBCC, Section 1617.4.4.4 of IBC determines the minimum accidental eccentricity to be 5% of the plan dimension perpendicular to the direction of loading. Whereas the NBCC requires a minimum accidental eccentricity equals 0.1 times the plan dimension perpendicular to the direction of loading as mentioned earlier in this paper.

Therefore,

$e_x = 0.05 \times 49.5 = 2.475$ m

$e_y = 0.05 \times 21.5 = 1.075$ m

As a result, the torsional moments are computed as follows

$T_y = V_y e_x = 2.475 \times 0.25 W_x$ kN.m

$T_x = V_x e_y = 1.075 \times 0.25 W_y$ kN.m

Through similar calculation procedures, the lateral forces and shears distribution were reproduced for the new value of the base shear in Tables 2-12 and 2-12. And the design shear forces and overturning moments on all walls could also be determined as shown in Tables 2-14 and 2-15

Table 2-12 Story lateral forces and shears in the E-W Direction where V_x = 18624 kN (IBC provisions)

Floor No.	h_i, m	W_i, kN	$h_i W_i$	C_{vi}	F_i	V_{ix}
roof	30.4	7831.81	7237845.5	0.2645	4925.2	4925.2
8	26.75	9490.98	6791389.4	0.2481	4621.4	9546.6
7	23.1	9490.98	5064481.8	0.1850	3446.3	12992.9
6	19.45	9490.98	3590461.5	0.1312	2443.2	15436.2
5	15.8	9490.98	2369328.2	0.0866	1612.3	17048.5
4	12.15	9490.98	1401082.2	0.0512	953.4	18001.9
3	8.5	9490.98	685723.3	0.0251	466.6	18468.5
2	4.85	9718.5	228603.4	0.0084	155.6	18624.0
Σ		74496	27368915.4		18624	

It can be clearly seen that the base shears and the earthquake loads calculated according to IBC exceed those found using the provisions of NBCC by comparing Tables 2-1 and 2-2 to Tables 2-12 and 2-13, and Tables 2-10 and 2-11 to Tables 2-14 to 2-15.

The value for V_x equal 18624 kN, is 2.71 times this value calculated according to NBCC which was 6863 kN, and V_y equal 18040 kN is 3.87 times the value found using NBCC which was 5591 kN. On the average, the base shear of IBC is 3.3 times the one found with NBCC.

The design forces are also higher than those calculated previously. In the case of shear wall No. 4, the design shear at the base found in this section of 9761 kN, is about 3 times the design shear found previously for the same wall, and the overturning moment of 214404 kN.m at the base, is 3.6 times the value previously calculated for the same wall.

Table 2-13 Story lateral forces and shears in the N-S Direction where V_y = 18040 kN (IBC provisions)

Floor No.	h_i, m	W_i, kN	h_iW_i	C_{vi}	F_i	V_{iy}
roof	30.4	7945.77	7097280.8	0.2670	4816.5	4816.5
8	26.75	9186.78	6573715.3	0.2473	4461.2	9277.7
7	23.1	9186.78	4902157.7	0.1844	3326.8	12604.5
6	19.45	9186.78	3475381.8	0.1307	2358.5	14963.1
5	15.8	9186.78	2293387.8	0.0863	1556.4	16519.4
4	12.15	9186.78	1356175.4	0.0510	920.4	17439.8
3	8.5	9186.78	663744.9	0.0250	450.4	17890.2
2	4.85	9358.14	220126.8	0.0083	149.4	18039.6
Σ		72160	26581970.5		18040	

Table 2-14 Design Shear forces V_u (in kN) according to IBC-2000 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	4856	4815	4693	4445	4024	3387	2489	1284
2	4856	4815	4693	4445	4024	3387	2489	1284
3	9020	8945	8720	8260	7482	6302	4639	2408
4	9761	9681	9437	8939	8097	6820	5020	2606
5	4656	4617	4500	4262	3859	3248	2387	1231
6	5587	5541	5401	5115	4631	3898	2864	1478

Table 2-15 Design Moments M_u (in kN.m) according to IBC-2000 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	110565	87983	71131	54704	39147	25062	13206	4494
2	110565	87983	71131	54704	39147	25062	13206	4494
3	214404	170658	138008	106181	76033	48725	25722	8790
4	214404	170658	138008	106181	76033	48725	25722	8790
5	110565	87983	71131	54704	39147	25062	13206	4494
6	110565	87983	71131	54704	39147	25062	13206	4494

2.4.4 Seismic base shear determination using the provisions of UBC-1997

In this section, the UBC-97 provisions are briefly discussed. The results obtained are useful to the interpretation of the significant differences obtained between the results of the IBC and the NBCC provisions.

The UBC-97 expression to determine the base shear was given as

$$V = \frac{C_v I}{RT} W$$

where

C_v is defined as Seismic coefficient that is given in Table 16-R of UBC 1997 and that depends on the Seismic zone factor, Z, and the soil profile type.

Assuming the site location is in Seismic zone 3 and with a soil profile type of S_B, the following parameter can be determined. (See Appendix C)

Z = 0.3 (Table 16-I of UBC-1997)

C_v = 0.3 (Table 16-R)

R = 4.5 (Table 16-N)

I = 1.0 (Table 16-K)

T has the same expression as the new Code, it was found to be 0.63 sec.

Hence,

$$V = \frac{0.3 \times 1}{4.5 \times 0.63} W = 0.105W$$

Therefore

$V_x = 7822$ kN

$V_y = 7577$ kN

V_x here is only 1.14 times V_x found using the Canadian provisions of NBCC, and V_y is 1.36 times the V_y found using NBCC.

Using similar lateral force distribution procedures, and taking into account the contribution of accidental torsion, the design shears and overturning moment are reproduced in Tables 2-16 and 2-17.

Compared to the Canadian code NBCC-1995, the results obtained using, UBC-1997, gave closer results than the new code IBC-2000, but in both cases the forces obtained were higher than those obtained with NBCC.

Table 2-16 Design Shear forces V_u (in kN) according to UBC –1997 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	2039	2022	1971	1867	1690	1423	1045	539
2	2039	2022	1971	1867	1690	1423	1045	539
3	3788	3757	3662	3469	3142	2647	1948	1011
4	4100	4066	3963	3754	3401	2865	2109	1095
5	1956	1939	1890	1790	1621	1364	1002	517
6	1956	1939	1890	1790	1621	1364	1002	517

Table 2-16 shows that the design shear at the base of wall No. 4 is 4100 kN, and the design moment is 90050, that are respectively 1.3 and 1.5 times the values obtained using NBCC.

Table 2-17 Design Moments M_u (in kN.m) according to UBC-1997 provisions

Wall	Floor							
No.	2	3	4	5	6	7	8	Roof
1	46437	36953	29875	22976	16442	10526	5546	1888
2	46437	36953	29875	22976	16442	10526	5546	1888
3	90050	71676	57964	44596	31934	20465	10803	3692
4	90050	71676	57964	44596	31934	20465	10803	3692
5	46437	36953	29875	22976	16442	10526	5546	1888
6	46437	36953	29875	22976	16442	10526	5546	1888

CHAPTER 3

DESIGN OF SHEAR WALL

3.1 Design of Reinforced Concrete Shear Wall According to CSA

3.1.1 Overview

The design in this section is done according to CSA23.3-94. Shear wall No. 4 is considered in the design because it was seen from Tables 2-10and 2-11 that it had the largest earthquake loads.

The design forces on the wall are reproduced below:

Wall	Floor							
No.4	2	3	4	5	6	7	8	Roof
Shear (kN)	3255	3154	2981	2733	2411	2014	1542	996
Moment (kN.m)	59600	46040	36200	27000	18700	11580	5910	1960

At the floor level, the maximum shear, V_u, is evaluated to be 3255 kN, including shear resulting from the accidental torsion, and the maximum moment at the base due to the seismic overturning moment is calculated to be 59600 kN.m

The preliminary calculations predicted that increasing the total length and using larger boundary elements was necessary. Figure 3-1 shows the assumed shear wall section.

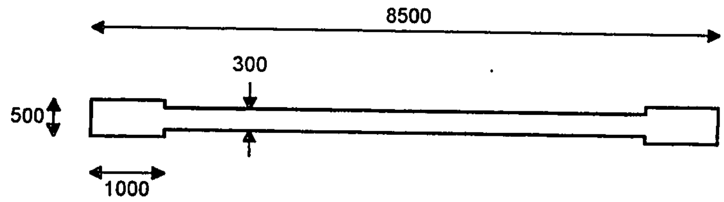


Figure 3-1 Shear wall section (Dimensions in mm)

CSA does not specify special conditions on the requirements of boundary elements, but it provides general requirements of what it calls the "area of concentrated reinforcement region" that are applicable to all kinds of shear walls. This area has to be at the ends of the wall and can have a thickness greater than the h_w if necessary, but the CSA does not require any other special conditions for the use of such boundary elements. But on the other hand, as it will be shown in this example, the CSA provides strict limits on the dimensions of the wall and other requirement on the transverse reinforcements to ensure that the ductility and stability of the wall will prevent crushing at the formation of plastic hinges.

3.1.2 Choice of the vertical reinforcement

Clause 21.5.6.4 specifies the minimum area of concentrated reinforcement in region of plastic hinging shall be $0.002b_wl_w$ at each end of the wall. A plastic hinge as defined by CSA is the region of a member where inelastic flexural curvatures occur. Its length extends at least d from a critical section, where d is the depth from the extreme compression to the centroid of tension reinforcement. The plastic hinge region extends over a height equals to l_w but not less than $h_w/6$ according to Figure N21.5.2.1 of CSA.

In the case of shear walls, according to Clause 21.7.3.2, d is taken to be the effective shear depth $d_v = 0.8(l_w) = 0.8\times8500 = 6800$ mm

$h_w/6 = 30400/6 = 5067\text{ mm} < l_w$

Therefore, the plastic hinge region extends over a height of 8500 mm, that is the first two stories of the building.

$A_s \geq 0.002 b_w l_w = 0.002 \times 300 \times 8500 = 5100\text{ mm}^2$

Use 8 – No. 30 bars as minimum requirements, providing 5600 mm^2

Outside the area of plastic hinging, only one half of this area is required.

Clause 21.5.6.3 specifies the maximum ratio of concentrated reinforcement to the area of concentrated reinforcement shall not be more than 0.06

$A_s \leq 0.06 \times 1000 \times 500 = 30000\text{ mm}^2$

The area of concentrated reinforcement is equivalent to be the boundary elements in the design according to ACI. The length 100 cm of this area is less than d_v, therefore, the plastic hinging occurs in this area.

Clause 21.5.4.4 specifies that the diameter of the bars used in the wall shall not exceed one-tenth of the thickness of the wall at the bar location, therefore, in the web the

maximum bar diameter is $0.1 \times 300 = 30$ mm. In the area of concentrated reinforcement the maximum bar diameter is $0.1 \times 500 = 50$ mm

To make the first estimate of the vertical reinforcement, the most practical way is to do a flexural design assuming that the wall is a rectangular beam, therefore,

$K_r = M_r \times 10^6 / bd^2 = 59600 \times 10^6 / 300 \times 6800^2 = 4.3$

Using Table 2.1 of CSA for $K_r = 4.3$ and $f_c' = 30$ MPa, $\rho = 0.0154$

$A_s = \rho b d_v = 0.0154 \times 300 \times 6800 = 3141600\text{ mm}^2$

Use 32 – No. 35 bars. A_s (Provided) = 320 cm^2, as first assumption.

Since the maximum permissible area of reinforcement in the boundary elements is 30000 mm^2, it is practical to provide 28 – No. 35 bars in the boundary element providing 28000 mm^2, and 4 other bars in the wall web providing the remaining 4000 mm^2.

3.1.3 Choice of distributed reinforcement

Clause 21.5.5.1 determines that the reinforcement ratio for both longitudinal and transverse distributed reinforcement shall be taken as not less than 0.0025 in each direction, and that the spacing of the reinforcement shall not exceed 450 mm.

In one meter of wall, the minimum area of distributed reinforcement required is:

$0.0025 \times 1000 \times 300 = 750\ mm^2/m$

Use No. 15 bars with a spacing of 450 mm as minimum requirements.

Assuming two curtains of reinforcement, they will provide:

$A_{sv\,(min)} = 2 \times 200 = 400\ mm^2/(0.45m\ \text{of wall}) = 889\ mm^2/m$

Clause 21.5.5.3 determines that at least two curtains of reinforcement shall be used if, in region of plastic hinging, the in-plane factored shear force assigned to the wall exceeds $0.2\lambda A_{cv}\sqrt{f_c'}$ where:

A_{cv} is the gross area of the concrete section bounded by the web thickness and length of section in the direction of lateral forces considered as defined by CSA.

λ is defined by Clause 8.6.5 as the modification factor for concrete density and it's given to be 1.0 for normal density concrete.

$A_{cv} = 8500 \times 300 = 2550000\ mm^2$

$$0.2\lambda A_{cv}\sqrt{f_c'} = \frac{0.2 \times 1.0 \times 2550000 \times \sqrt{30}}{1000} = 1676kN < V_u = 3255kN$$

Therefore, two curtains of reinforcements are required.

Preliminary calculations showed that the minimum area of distributed reinforcement is not enough in this case. Let's assume at this stage, two curtains of reinforcements of No. 15 bars spaced at 200 mm in both horizontal and vertical directions.

Therefore, $A_{sv} = 2\times200 = 400\ mm^2/(0.2m\ of\ wall) = 2000\ mm^2/m$

3.1.4 Check for flexure and ductility

3.1.4.1 Calculation of the depth of the neutral axis

Clause 21.5.1.2 clearly determines that ductile flexural walls shall not be deigned using the provisions of Clause 14.5 that gives the empirical design method for solid rectangular reinforced concrete walls. As a result, the assumption that the wall is a rectangular beam made to chose the vertical reinforcement is not sufficient, further investigation is required.

Moreover, to ensure the ductility of the wall especially in the plastic hinge region, Clause 21.5.7 requires Clause 21.5.7 specifies that, the distance to the neutral axis c_c, found by calculating the factored flexural resistance of the wall under the action of the axial loads, P_s, P_n, and P_{ns}, shall not exceed $0.55l_w$. when c_c exceeds $0.14\gamma_w l_w$, special confinement reinforcement must be provided over a distance of not less than $c_c(0.25 + c_c/l_w)$ from the compression face of the wall even if it has to be outside the area of concentrated reinforcement.

P_s, P_n, and P_{ns} are respectively defined as: the axial force at section resulting from specified dead load plus specified live load, the earthquake transfer force resulting from interaction between elements of a coupled wall, and the nominal net force due to yielding of reinforcement during plastic hinge formation.

γ_w is the overstrength factor equal to the ratio of the ratio of the load corresponding to the nominal moment resistance of the wall system to the factored load on the wall system as defined by CSA.

As mentioned by CSA in Clause 21.5.7, c_c can be found by calculating the flexural resistance of the wall or by using the equation:

$$c_c = \frac{P_s + P_n + P_{ns} - \alpha_1 \phi_c f_c' A_f}{\alpha_1 \beta_1 \phi_c f_c' b_w}$$

In this example, c_c is evaluated by calculating the flexural resistance of the wall. In this case, this method is more accurate especially because there is no P_n. Moreover, the flexural resistance is needed to ensure that the wall is able to resist the design moment with the assumed vertical reinforcement.

Figure 2-3 below represents the assumption for the stress block.

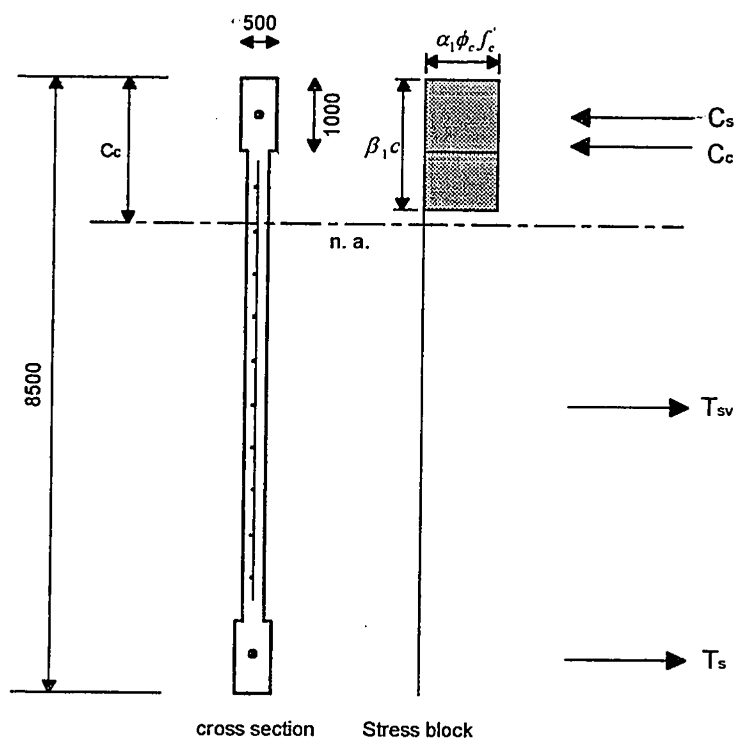


Figure 3-2 Shear wall assumed stress block
(Dimensions in mm)

The maximum concrete strain is 0.0035. Assuming that the distributed reinforcement in the wall web will yield before the vertical reinforcement, the vertical steel in the boundary element in compression will balance the steel in tension and C_s is in this case equal T_s. The resultant force of concrete in compression, C_c, has to be balanced by the distributed steel in tension that is not balancing the other distributed steel in compression, and that is found over a length 8500 minus $2c_c$. In this case, the difference between the compression force and the tension force must equal the axial load applied on the shear wall.

The axial load is the assigned dead load plus live load as required by CSA

$A_g = 1000 \times 500 \times 2 + 6500 \times 300 = 2950000 \text{ mm}^2$

$DL = 2.95 \times 24 \times 30.4 = 2152 \text{ kN}$

Assuming that the shear wall is carrying an area of 7×14m from each floor, applying 2.4 kN/m^2 on this area, the total live load is therefore calculated to be:

$7 \times 14 \times 2.4 \times 7 = 1646$ kN, where 7 is the number of stories neglecting the roof.

$LL = 1646 \text{ kN}$

$W = DL + LL = 2152.32 + 1646.4 = 3799 \text{ kN}$

$C - T = W$

$C = ((\beta_1 c_c - 1000) \times 300 + 1000 \times 500) \times \alpha_1 \times \phi_c \times f_c'$

$T = (8.5 - 2 \times c_c(\text{in meters})) \times (A_{sv}/(\text{meter})) \times \phi_s f_y$

With $\phi_c = 0.6$, and $\phi_s = 0.85$ (Clause 8.4.2 & 8.4.3),

$\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67$, $\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67$ (Clause 10.1.7)

$\alpha_1 = 0.85 - 0.0015 \times 30 = 0.805$, $\beta_1 = 0.97 - 0.0025 \times 30 = 0.895$

That leads to the equations,

$C = ((0.895c_c - 1000)\times300 + 1000\times500)\times0.805\times0.6\times30$

$T = (8.5 \ - 2\times c_c)\times2000\times0.85\times400$

$C - T = 3798.72$

Solving leads to:

$c_c = 1272.4$ mm,

$C = 7848.35$ kN

$T = 4049.53$ kN

$7848.35 - 4049.53 = 3798.8 \text{ kN} = W$

3.1.4.2 Check for flexure

The resisting moment of the shear wall could be determined by calculating the moment about the line of wall and neglecting the distributed reinforcement:

$$M_r = A_s\phi_s f_y(4250 - 500)\times2 + 500\times200\times\alpha_1\phi_c f_c'\times(4250 - 500) + \beta_1 c_c\times300\times\alpha_1\phi_c f_c'\times(4250 - 0.5\beta_1 c_c)$$

$$M_r = 32000\times0.85\times400\times(4250 - 500)\times2 + 500\times200\times0.805\times0.6\times30\times(4250 - 500) + 0.895\times1272.4\times300\times0.805\times0.6\times30\times(4250 - 0.5\times0.895\times1272.4)$$

$$M_r = 105254 \text{ kN.m} > 59600 \text{ kN.m}$$

Since the resisting moment is exceeding by far the design moment, it is possible to decrease the amount of steel that was assumed by the direct flexural analysis. Predicting that the ratio of M_r to M is normally about 1.2, let's assume at this time, 24 – No. 30 vertical bars providing 16800 mm^2. The change in this assumption will not affect the previous calculations for the depth of the neutral axis c_c. Since 16800 mm^2 is less than the maximum permitted of 30000 mm^2, it will be possible to place all the vertical reinforcement within the boundary elements.

The new value for M_r is:

$M_r = 16800\times0.85\times400\times(4250-500)\times2 +$

$500\times200\times0.805\times0.6\times30\times(4250-500) +$

$0.895\times1272.4\times300\times0.805\times0.6\times30\times(4250-0.5\times0.895\times1272.4)$

$M_r = 66494$ kN.m > 59600 kN.m O.K.

Use 24 – No. 30 bars , A_s provided = 16800 mm^2

3.1.4.3 Check for ductility

In order to calculate the overstrength ratio γ_w, the nominal moment resistance of the wall will be carried out with $\phi_s = \phi_c = 1.0$ and using equivalent yield strength of steel of $1.25f_y$. The nominal moment resistance will be called the probable moment resistance of the wall M_{pw}, and its ratio to the design moment will be γ_w.

Therefore,

$M_{pw} = 16800\times1.0\times1.25\times400\times(4250-500)\times2 +$

$500\times200\times0.805\times1.0\times30\times(4250-500) +$

$0.895\times1272.4\times300\times0.805\times1.0\times30\times(4250-0.5\times0.895\times1272.4)$

$M_{pw} = 102423$ kN.m

$\gamma_w = M_{pw} / (J\times M_{overturning}) = 102423 / (0.8\times59600) = 2.15$

Now we can check the ductility of the wall, according to the requirements of Clause 21.5.7:

$0.55l_w = 0.55\times8500 = 4675 \text{ mm} > c_c$ O.K.

$0.14\gamma_w l_w = 0.14\times2.15\times8500 = 2558.5 \text{ mm} > c_c$

The ductility requirements of Clause 21.5.7 are met because c_c is less than the upper limit of $0.55l_w$, and less than $0.14\gamma_w l_w$.

3.1.5 Check wall stability

Clause 21.5.3.1 provides dimensional limitations to ensure the stability of shear wall. Special limitations are required to the effective flange widths to be used in the design of L-, C-, or T-shaped sections.

According to Clause 21.5.3.2, the wall thickness within the plastic hinge region shall not be less than l_u / 10, within those parts of a wall which, under factored vertical and lateral loads, are more than half-way from the neutral axis to the compression face of the wall section.

In this example, the wall thickness within the plastic hinge region is 500 mm.
$l_u / 10 = 3040/10 = 3040$ mm > 500 mm O.K.
This condition is satisfied, therefore, the shear wall meets the stability requirements.

3.1.6 Confinement of concentrated reinforcement

Clause 21.4.4.3 gives similar design procedures as in ACI as it will be seen in Section 3.2 in this study. The maximum vertical spacing of the ties s is determined in Clause 21.4.4.3 as not exceeding the smallest of:
- one-quarter of the minimum member dimension, $0.25 \times 500 = 125$ mm
- 100 mm
- 6 times the diameter of the smallest longitudinal bar, $6 \times 35 = 210$ mm
- the requirements of confinement reinforcement as a column in accordance with Clause 7.6

Clause 7.6 requires the spacing to be the smallest of: 16 times the bar diameter, 48 times the tie diameters, the least dimension of the compression member, and 300 mm that all exceeds the above dimensions in this clause.

Clause 21.4.4 specifies the total area of rectangular hoop reinforcement A_{sh} as not less than the larger of the amounts $0.3(sh_c f_c'/f_{yh})[(A_g/A_{ch}) - 1]$, and $0.09 sh_c f_c'/f_{yh}$

However, Clause 21.5.7 clearly specifies: " when c_c **exceeds** $\mathbf{0.14\gamma_w l_w}$, the compression region of the wall shall be confined as a column, as specified in Clause 21.4.4, over a distance of not less than $c_c(0.25 + c_c/l_w)$ from the compression face of the wall" but in this example, it c_c does not exceed $0.14\gamma_w l_w$.

Clause 21.5.6.5 requires that the concentrated reinforcement shall be at least tied as a column in accordance with Clause 7.6, and the ties shall be detailed as hoops. **In region of plastic hinging**, the spacing between ties shall not exceed the least of:

- 6 longitudinal bar diameter, 6×30 = 180 mm
- 24 tie diameter, 24×15 = 360 mm
- One-half of the wall thickness, 0.5×500 = 250 mm
- That required by Clause 21.5.7, **if applicable**

That required by Clause 21.5.7 **is not applicable** because c_c **does not** exceed $0.14\gamma_w l_w$. Hence, in the region of plastic hinging, that is the boundary elements over a height of up to two stories of the building, provide No. 15 lateral ties detailed as hoops at a spacing of 180 mm

3.1.7 Check for shear

The design shear on the wall determined from the lateral force distribution analysis was 3255 kN. It is assumed that earthquake loading causes plastic hinging at the base of the wall when the moment increases to the probable resistance. Assuming that the ratio of the shear to the moment remains constant as the moment increases to its probable value, the shear at the base as the wall develops a plastic hinge will be

$V_f = (M_{pw}/M) \times V_u$

$V_f = (102423/59600) \times 3255 = 1.72 \times 3255 = 5594$ kN

The requirements of Clause 11 are used to calculate the shear capacity as follows:

With $V_{rg} = V_{cg} + V_{sg}$

V_{cg} is given by Clause11.4.3.1 as:

$V_{cg} = 1.3\lambda\phi_c\beta\sqrt{f_c'}b_w d_v$,

and V_{sg} is given by Clause 11.4.3.2 as:

$V_{sg} = (\phi_s A_v f_y d_v \cot\theta)/s$

The average vertical strain ε_v is given by Equation 21-8 of Clause 21.7.3.2 as:

$$\varepsilon_v = \frac{0.001}{\gamma_w}\left(0.5 - \frac{c_v}{l_w}\right)\left(18 + \frac{h_w}{l_w}\right)$$

$$\varepsilon_v = \frac{0.001}{2.15}\left(0.5 - \frac{1272.4}{8500}\right)\left(18 + \frac{30400}{8500}\right) = 0.003$$

v_f is given by Clause 11.4.5 as:

$v_f = V_f/b_w d_v$

$$v_v = \frac{5594 \times 1000}{300 \times 6800} = 2.74MPa$$

$$\frac{v_f}{\lambda\phi_c f_c'} = \frac{2.74}{1.0 \times 0.6 \times 30} = 0.15$$

With $\lambda = 1.0$ for normal weight concrete according to Clause 8.6.5:

From Table 21-1 of CSA, β is taken to be 0.122 because ($v_f / \lambda\phi_c f_c'$) equals 0.15 and ε_v equals 0.003

θ is taken to be 45° as required by Clause 21.7.3.2, therefore $\cot\theta$ equals 1.0

Hence,

$$V_{rg} = \left(1.3\phi_c \beta \sqrt{f_c'} b_w d_v + \frac{\phi_s A_v f_y d}{s}\right)$$

It was previously assumed to provide two curtains of distributed reinforcements of No. 15 bars with a spacing of 200 mm in both directions, factored shear resistance is:

$$V_{rg} = \left(1.3 \times 0.6 \times 0.122 \times \sqrt{30} \times 300 \times 6800 + \frac{0.85 \times 2 \times 200 \times 400 \times 6800}{200}\right) \times 10^{-3}$$

$V_{rg} = 5687$ kN > $V_f = 5594$ kN

Hence, the shear strength is adequate with the assumed reinforcement
Use No.15 bars with a spacing of 200 mm centre-to-centre in both directions

3.1.8 Determine required development and splice lengths

Clause 21.5.5.2 requires that horizontal reinforcement shall extend to the ends of the wall and shall be contained at each end of the wall within a region of concentrated reinforcement. Unlike ACI, the CSA does not provide any further special requirements for developing reinforcement. Development of reinforcement in shear walls could be done using the basic requirements of Clauses 12.2, 12.14, and 12.15.

The Shear wall section is represented in Figure 3-3

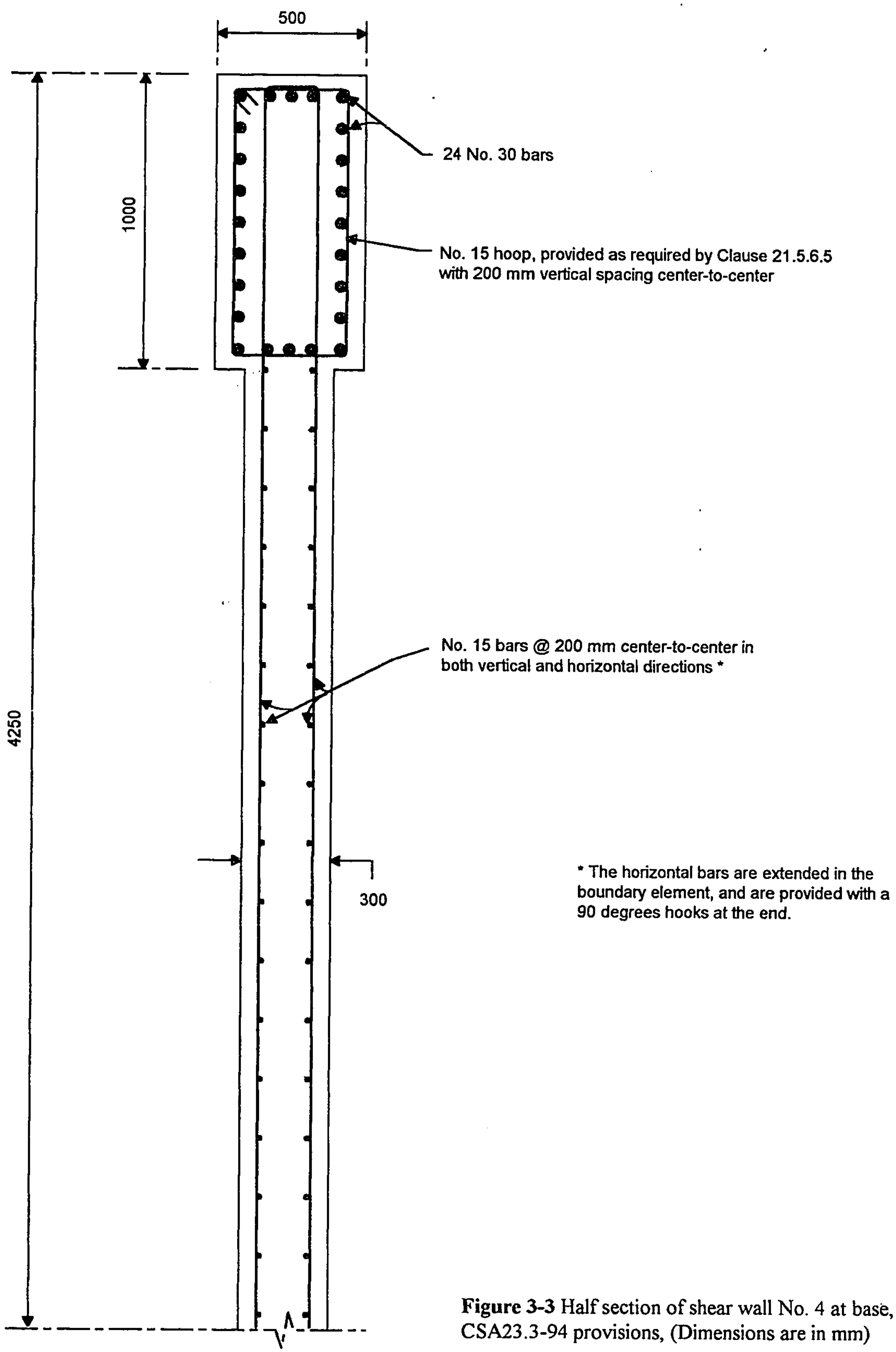


Figure 3-3 Half section of shear wall No. 4 at base, CSA23.3-94 provisions, (Dimensions are in mm)

3.2 Design of Reinforced Concrete Shear Wall According to the ACI

3.2.1 General principles

This section describes the detailing of shear wall No. 4 according to the provisions of ACI 318-99, using the seismic loads obtained from the NBCC-1995 provisions, that is, at the floor level, the maximum shear V_u is evaluated to be 3255 kN, and the maximum moment at the base is calculated to be 59600 kN.m.

The principle provisions of Chapter 21 of ACI, and particularly Section 21.6, relating to shear walls are as follows:

(a) Shear walls must be provided with shear reinforcement in both directions in the plane of the wall. The minimum reinforcement ratio for both longitudinal and transverse directions is:

$\rho_n = A_{sv} / A_{cv} \geq 0.0025$

where

A_{cv} is the gross concrete area,

A_{sv} is distributed reinforcement perpendicular to the plane of A_{cv}

The maximum spacing shall not exceed 450 mm. (Section 21.6.2.1 of ACI)

(b) Shear strength is determined using the expression:

$V_n = A_{cv}\,((1/12)\alpha_c\sqrt{f_c'} + \rho_n f_y)$ (Section 21.6.4.1)

Where α_c is a coefficient that depends on the height-to-width ratio of the wall h_w/l_w

(c) All continuous reinforcement must be anchored or spliced in accordance with the provisions of Section 21.5.4

(d) Boundary elements: Shear walls considered effectively continuous from base to top of wall are designed to have a single critical section for flexure and axial loads. Otherwise, shear walls must be provided with boundary elements when the maximum extreme-fibre stress in wall due to factored forces including earthquake effects exceeds $0.2f_c'$. In that case, a single boundary element is required to carry the entire compressive axial loads resulting from gravity and overturning moment, and they must be provided with special confinement reinforcement.

3.2.2 *Check for boundary elements*

Boundary elements at the edges of shear walls are very effective to resist axial loads and the combination of axial loads and bending moments at the base of shear wall. The first step in shear wall design is to check whether boundary elements are required: section 21.6.6.3 of ACI requires boundary elements at boundaries and edges around openings of shear walls if the maximum extreme fibre compressive stress, corresponding to factored forces including earthquake effect, exceeds $0.2f_c'$. Section 21.6.6.2 requires specific dimension of compression zones of the boundary elements if the walls are effectively continuous from the base of the structure to the top of wall. In this example, the shear wall is not considered as effectively continuous over its entire height, therefore, the requirements of Section 21.6.6.3 apply.

Extreme fibre stress is given by:

$$f_c = \frac{P_u}{A_g} + \frac{M_u(h_w/2)}{I_{n.a.}}$$

Where P_u is the axial load from gravity and M_u is the moment from earthquake lateral loads.

IBC-2000 provides in Section 1605.2 basic load combinations that include earthquake, (see Appendix E)

The load combination used in this example is the expression given by Section 9.2 of ACI.

The required strength to resist earthquakes loads is given by section 9.2 as:

$U = 0.75(1.4D + 1.7L + 1.7W)$

Where W is substituted by 1.1E according to Section 9.2.3

Hence,

$U = 1.05D + 1.28L + 1.4E$

Weight of shear wall is 208.26 × 8 = 1664 kN (8 is the number of stories 208.26 kN is the weight if the shear wall between ground level and first story.

D = 1664 kN

Following the same approach used in the previous section to determine the live load:

L = 1646.4 kN

W = 0.75(1.4×1664 + 1.7×1646.4) = 3846.36 kN

Using the assumed wall section of Figure 3-1:

$I_{n.a.} = 0.3\times(8.5)^3 / 12 = 15.35\ m^4 = 1.535\times10^{13}$ mm

$A_g = 8500\times300 = 2550000\ mm^2$

$$f_c = \frac{3846.36\times1000}{2550000} + \frac{59600\times10^6(8500/2)}{1.535\times10^{13}} = 16.5MPa$$

$0.2f_c' = 0.2\times30 = 6$ MPa

Since the extreme-fibre compressive stress is equal to 16.5 MPa > $0.2f_c'$ = 6, boundary elements are required, subject to the confinement and special loading requirements specified in Sections 21.6.6.3 and R21.6.6.3 of ACI 318

3.2.3 Check adequacy of the boundary element to resist axial load

The shear wall is 8500 mm long from end to end with a thickness of 300 mm and with 1000×500 meter rectangular boundary elements as shown in Figure 3.1

$A_g = 6500\times300 + 1000\times500\times2 = 2950000\ mm^2$

D = 2.95×24×30.4 = 2152.32 kN

L = 1646.4 kN

W = 0.75(1.4×2152 + 1.7×1646.4) = 4359 kN

Figure 3.2 below illustrates the condition requiring that boundary elements of wall be designed for all the gravity loads, W, as well as the vertical forces associated with the overturning moment on the wall due to earthquake forces, H. This requirements assumes that the boundary element alone may have to carry all the vertical (compressive) forces at the critical wall section when the maximum horizontal earthquake force acts on the wall. With the accordance of Section 21.6.6.3 and R21.6.6.3

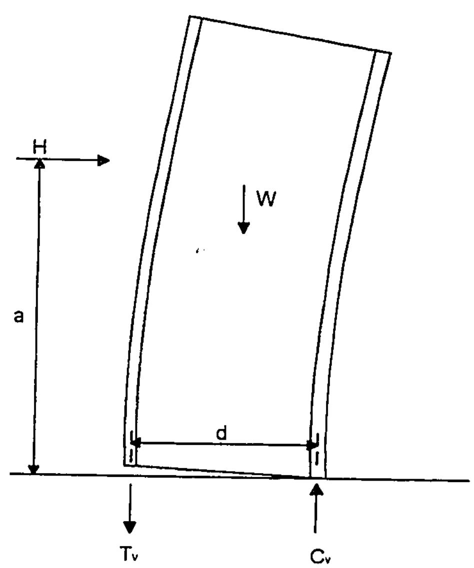


Figure 3-4 Loading condition assumed for design of boundary elements of shear walls

E is multiplied by a factor of 1.4 according to the load combination of Section 9.2.3 of ACI (see page 49), with an overturning moment $M_u = 59600$ kN.m,

$Ha = 1.4M_u = 1.4 \times 59600 = 83440$ kN.m

$C_v = W/2 + Ha/d = 4359/2 + 83440/8.5 = 11995.97$ kN

$C_v = 11996$ kN

With the boundary elements having the rectangular dimensions of 1000×500 mm . Let's assume that the section is reinforced with 26 – No. 30 bars, $A_{st} = 26000\ mm^2$

Axial load capacity of boundary element acting as a short column is given by section 10.3.5.2 of ACI as:

$$\phi P_n(max) = 0.8\phi[0.85f_c'(A_g - A_{st}) + f_yA_{st}]$$

Strength reduction factored considered is $\phi = 0.7$ according to section 9.3.2.2 of ACI

$$\phi P_n(max) = 0.8\times0.7\times[0.85\times30\times(500000 - 26000) + 400\times26000]$$

$$= 12592720\ N$$

$$= 12592\ kN > P_u = 11996\ kN\ \ O.K.$$

Maximum force on boundary element P_u = 11995.97 kN, and the reinforcement ratio for the boundary elements ρ_{st} is 26000/500000 = 0.052, section 10.9.1 requires the limits of the reinforcement ratios for compression members as not less than 0.01 and not more than 0.08, therefore, the section considered is adequate.

3.2.4 Check whether two curtains of reinforcement are required

Section 21.6.2.2 of the ACI code specifies that at least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds:

$(1/6)A_{cv}\sqrt{f_c'}$

where A_{cv} is the gross area of the concrete section bounded by the web thickness and length of section in the direction of shear force considered.

$$A_{cv} = 8500\times300 = 2550000\ mm^2$$

$$(1/6)A_{cv}\sqrt{f_c'} = \frac{(1/6)\times2550000\times\sqrt{30}}{1000} = 2327.82kN < V_u = 3255kN$$

Therefore, two curtains of reinforcement are required.

3.2.5 Required transverse reinforcement to resist shear

The minimum required web reinforcement ratios, ρ_v, and ρ_n, are specified in section 21.6.2.1 as not less than 0.0025 except when the design shear force is less than $(1/12)A_{cv}\sqrt{f_c'}$ which is not the case in this example. The maximum spacing shall not exceed 450 mm. ρ_v is the ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}, and ρ_n is the ratio of the distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement.

Let's evaluate the required the longitudinal and transverse reinforcement per meter of wall, we have $A_{cv} = 1000\times300 = 300000$ mm^2/(meter of wall)

The required area of reinforcement will be $0.0025\times300000 = 750$ mm^2/m in both directions

Use No. 15 bars with a spacing of 450 mm as minimum requirements.

Assuming two curtains of reinforcement, they will provide:

$A_{sv\,(min)} = 2\times200 = 400$ mm^2/(0.45m of wall) = 889 mm^2/m

The shear strength is given by section 21.6.4.1 as

$V_n = A_{cv}\,((1/12)\alpha_c\sqrt{f_c'} + \rho_n f_y)$

Where the coefficient α_c is:

1/4 for $h_w/l_w \leq 1.5$,

1/6 for $h_w/l_w \geq 2$,

varies linearly between 1/4 and 1/6 for h_w/l_w between 1.5 and 2.0.

h_w is the height of the entire wall and l_w is the length of the entire wall.

$h_w/l_w = 30.4/8.5 = 3.57 > 2$, so α_c is 1/6

Section 9.3.4 gives the strength reduction factor ϕ to be 0.6 for shear strength in any structural member designed to resist earthquake effects.

With the minimum reinforcement:

$\phi V_n = 0.6\times2550000((1/12)\times(1/6)\times\sqrt{30} + 0.0025\times400)/1000 = 1646$ kN $< V_u = 3255$ kN

Therefore, the minimum amount of reinforcement is not enough.

Let's assume No. 15 bars with a spacing of 250 mm centre to centre in two curtains in both directions, $A_{sv} = 2\times200 = 400$ mm^2/(0.25m of wall) = 1600 mm^2/m

$\rho_v = \rho_n = 1600/300000 = 0.00533$

$\phi V_n = 0.6\times2550000((1/12)\times(1/6)\times\sqrt{30} + 0.00533\times400)/1000 = 3380$ kN $> V_u = 3255$ kN O.K.

Therefore, use two curtains of reinforcement of No. 15 spaced at 250 mm in both horizontal and vertical directions

3.2.6 Check adequacy of the section to resist flexure

The flexural design of the shear wall conservatively assumes that the shear wall will have to resist the moment at the base which is the overturning moment at that level calculated from the lateral force distribution. Hence, the flexural design capacity is evaluated the same way as any concrete member resisting flexural bending moment. Since boundary elements are used, the shear wall is assumed to behave like a rectangular beam with a the effective depth d. The same area of steel should be provided in both boundary elements because the lateral force can generally act in either direction.

Strength reduction factored for flexure is $\phi = 0.9$ according to section 9.3.2.2 of ACI

According the reinforced concrete flexural design procedure:

$$M_u = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

where

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

With d is taken to be 0.8 times l_w and $M_u = 59600$ kN.m

$M_u = 59600$ kN.m kN.m

d = 0.8×8500 mm = 6800 mm

b = 300 mm

Solving the above two equations:

a = 1273.25 mm

$A_s = 24400\ mm^2$

In the boundary elements, 26 – No. 30 bars are already providing 26000 mm^2 > 24400 mm^2 O.K.

The reinforcement limits are according to ACI calculated as follows:

Maximum permissible reinforcement ratio by ρ_{max} is $0.75\rho_b$ Sections 8.4 and 10.2.7

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\left(\frac{600}{600+f_y}\right)$$

Where $\beta_1 = 0.85$, for $f_c' \leq 30$ MPa

$= 0.85 - 0.05\times(f_c' - 30)/7 \geq 0.65$, for $f_c' > 30$ MPa

For $f_{c'} = 30$ MPa, $\beta_1 = 0.85$,

$$\rho_{max} = 0.75 \times \frac{0.85 \times 0.85 \times 30}{400}\left(\frac{600}{600+400}\right) = 0.0243$$

The reinforcement ratio required is 24400/(6800×300) = 0.01196 and the reinforcement ratio provided $\rho = A_{st}/bd$ = 26000/(6800×300) = 0.01274. With ρ_{min} = 0.0025 and ρ_{max} = 0.0227, the reinforcement ratios are within the limits, therefore, the section is adequate with respect to bending moment resulting from the maximum earthquake overturning moment at the base of wall.

3.2.7 Determine the lateral confinement reinforcement required for boundary elements

Section 21.4.4.2 gives the maximum spacing of lateral ties as not exceeding:

- One-quarter of the minimum member dimension, 0.25×500 = 125 mm
- Six times the diameter of the longitudinal reinforcement, 6×35 = 210 mm
- $s_x = 100 + \frac{1}{3}(350 - h_x)$, not exceeding 150 mm and not less than 100 mm, where h_x is the maximum horizontal spacing of hoop or cross legs on all faces of the boundary element.

Assuming four cross ties (two legs and the outside hoop) of No. 5 in the short direction with a concrete clear cover of 40 mm,

$h_x = (1000 - 40 - 40 - 15)/4 = 226.25$ mm

Therefore,

$s_x = 100 + \frac{1}{3}(350 - 226.75) = 141.25$ mm

Figure R21.4.4 of ACI shows that the horizontal spacing, h_x, must not exceed 350 mm. In this case the assumed h_x is 226.25 < 350 mm O.K.

For the spacing of lateral ties, the value of s_x = 141.25 mm governs; however, preliminary calculations showed that a spacing of 120 mm would be more appropriate to avoid using more than 4 cross ties in the short direction as already assumed. Therefore a vertical spacing of lateral ties s =120 mm centre-to-centre in the boundary elements was used.

Required cross-sectional area of confinement reinforcement A_{sh} is given by section 21.4.4.1.b as not less than:

$0.3(sh_c f_c'/f_{yh})[(A_g/A_{ch}) - 1]$,

or $0.09 s h_c f_c'/f_{yh}$

whichever is greater

where

h_c is the cross-sectional dimension of the boundary element core measured centre-to-centre of confinement reinforcement,

A_g is the gross area of section,

A_{ch} is the cross-sectional area measured out-to-out of transverse reinforcement.

Assuming No. 5 ties,

$h_c = 1000 - 40 - 40 - 15 = 905$ mm, in the short direction

$h_c = 500 - 40 - 40 - 15 = 405$ mm, in the long direction

$A_{ch} = (1000 - 40 - 40) \times (500 - 40 - 40) = 386400\ mm^2$

$A_g = 1000 \times 500 = 500000\ mm^2$

In the short direction:

$$A_{sh} \geq \begin{cases} 0.3 \times 120 \times 905 \times \left(\dfrac{30}{400}\right) \times \left[\dfrac{500000}{386400} - 1\right] = 718.37 mm^2 \\ 0.09 \times 120 \times 905 \times \left(\dfrac{30}{400}\right) = 733.05 mm^2 \end{cases}$$

Therefore $A_{sh} = 733.05\ mm^2$

Provide four cross ties of No. 5, (two legs and the outside hoops), Spacing between ties is 226.25 mm < 350 mm O.K.

A_{sh} (provided) = 4×200 = 800 mm^2 O.K.

In the long direction:

$$A_{sh} \geq \begin{cases} 0.3 \times 120 \times 405 \times \left(\dfrac{30}{400}\right) \times \left[\dfrac{500000}{386400} - 1\right] = 321.48 mm^2 \\ 0.09 \times 120 \times 405 \times \left(\dfrac{30}{400}\right) = 328.05 mm^2 \end{cases}$$

Therefore A_{sh} = 328.05 mm^2

The use of two cross ties providing 2×2 = 4 cm^2 will make the spacing between the ties equal to (500 – 40 – 40 – 15) = 405 cm, which exceeds the maximum of 350 mm allowed by the ACI as shown in Figure R21.4.4. As a result, three cross ties (one leg and the outside hoop) have to be used in the long direction giving a spacing of (500 – 40 – 40 – 15)/2 = 202.5 mm < 350 mm O.K.

A_{sh} (provided) = 3×200 = 600 mm^2 O.K.

3.2.8 Determine required development and splice lengths

ACI requires that all continuous reinforcement in structural walls be anchored or spliced in accordance with the provisions for reinforcement in tension as given

Lap splices for the No. 35 vertical bars in boundary elements:

Section 12.15.2 of ACI determines that lap splices of deformed bars and deformed wire in tension shall be Class B splices except that Class A are allowed when: (a) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and (b) one-half or less of the total reinforcement is spliced within the required lap length.

Assuming that 50% or less of the vertical bars at any one location, and since the area of provided reinforcement is not twice as that required by the design, a class B splice may be used.

Required length of splice is $1.3l_d$ according to section 12.15.1 of ACI
l_d is in this case given by section 21.5.4.2 as follows:

$$l_d \geq 2.5\times\begin{cases} f_y d_b / 5.4\sqrt{f_c'} = \dfrac{400\times 35}{5.4\times\sqrt{30}} = 473.34mm \\ 8d_b = 8\times 35 = 280mm \\ 150mm \end{cases}$$

Thus the required splice length for the longitudinal bars is:
1.3×2.5×473.34 = 1538.35 mm. Use 160 cm.

Note that in the case where it's difficult to be provide this length for these large bars for practical reasons, section 21.2.6 of ACI allows the use of mechanical splices of both Type I and Type II. The mechanical splice, according to section 12.14.3.2, shall develop at least 125% of the specified yield strength f_y of the longitudinal bar.

For lap splice for No.15 vertical bars in the wall web, and assuming again no more than 50% of bars spliced at any one level, so that Class B splice may be used, and using the same expression for l_d above:

$$l_d \geq 2.5\times\begin{cases} f_y d_b / 5.4\sqrt{f_c'} = \dfrac{400\times 15}{5.4\times\sqrt{30}} = 202.86mm \\ 8d_b = 8\times 15 = 120mm \\ 150mm \end{cases}$$

Therefore, the required splice length for the vertical reinforcement is:
1.3×2.5×202.86 = 659.3 mm Use 66 cm.

For the development length for No.15 horizontal bars in wall, assuming no hooks are used in the boundary elements, and assuming that the depth of concrete cast in one lift beneath the bar exceeds 300 mm, the length of should be multiplied by 3.5 instead of 2.5 as determined by section 21.5.4.2, thus, the required length for the horizontal reinforcement is:

$1.3 \times 3.5 \times 202.86 = 923$ mm

Cross-sectional dimension h_c of the confined core of the boundary element is determined to be 905 mm. This distance cannot accommodate the length of the required splices of the horizontal reinforcement, therefore, the horizontal bars will be provided with 90° hooks engaging a vertical bar as shown in Figure 3-5. Figure 3-5 represent the shear wall detailing at base according to the provisions discussed in this section.

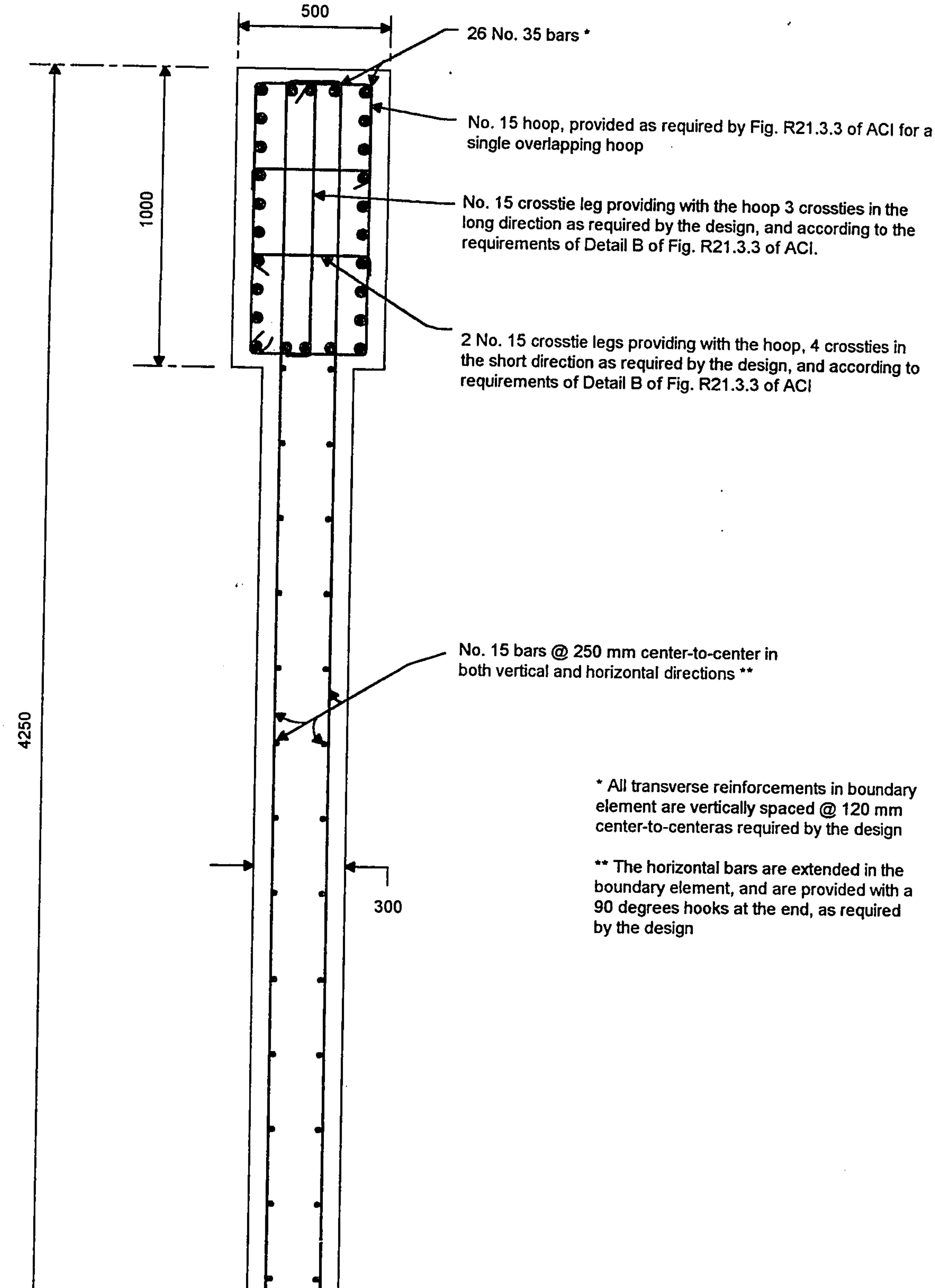


Figure 3-5 Half section of shear wall No. 4 at base, ACI 318-94 provisions, (Dimensions are in mm)

3.3 Design of Reinforced Concrete Shear Wall According to NZS

3.3.1 Overview

The General seismic design requirements are listed in Section 12.4 of NZS:3101:part 1:1995 (See Appendix D.) The requirements are similar to those of the CSA to a large extent as it will be seen in the study that follows. The major differences include the dimensional limitations and the confinement of vertical bars against buckling.

3.3.2 Limitations on Wall Reinforcement

Section 12.4.3.1 of NZS 3101 requires that the ratio ρ_l of longitudinal reinforcement over any part of a wall shall not be less than $0.7/f_y$ nor more than $16/f_y$. Where ρ_l is defined by NZS as the ratio of the steel area to the gross concrete area of the section

Hence,

$\rho_{l\,min} = 0.7/400 = 0.00175$

$\rho_{l\,max} = 16/400 = 0.04$

Section 12.4.3.2 determines that: "in walls thicker than 200 mm at least 2 layers of reinforcement shall be used, one near each face of the wall". Therefore, since b_w in this case is 300 mm, it can be directly determined that two curtains of transverse reinforcement must be provided without the need of any further investigations.

Section 12.4.3.3 determines that the diameter of the bars used in any part of a ductile wall shall not exceed one tenth of the thickness of the wall, which is the same requirement as the CSA code.

In the wall web, $(1/10)\times300 = 30$ mm, bars No. 30 or less shall be used

In the boundary elements, $(1/10)\times500 = 50$ mm, bars up to No. 45 are allowed.

3.3.3 Check for flexure and ductility

3.3.3.1 Calculation of the depth of the neutral axis

Following the same approach of the design according to the CSA, and using the same assumption of Figure 3-4, the compression force of concrete in compression balanced by distributed reinforcement over a length of l_w minus 2c. The difference between the compression force and the tension force must equal the axial load applied on the shear wall.

The NZS requires the axial load on the shear wall, calculated for seismic design, to be:

$P_u = D + 1.3L_r$

Where D is the dead load, L_r is the live load, L, times a live-load reduction factor, r, calculated as follows:

$$r = 0.3 + \frac{3}{\sqrt{A}} \leq 1.0$$

A is defined as the total tributary area where the live load is applied, not to be taken less than 20 m^2

It was assumed that the shear wall is carrying an area of 7×14m from each floor. If we take into account seven floors and neglect the roof, the tributary area for the live load on the shear wall becomes

$A = 7 \times 14 \times 7 = 686\ m^2$

Hence,

$$r = 0.3 + \frac{3}{\sqrt{686}} \leq 1.0$$

$$r = 0.414$$

Using a live load of 2.4 kN/m^2,

$L = 686 \times 2.4 = 1646.4$ kN

$L_r = 0.414 \times 1646.4 = 682$ kN

On the other hand,

$A_g = 1000 \times 500 \times 2 + 6500 \times 300 = 2950000\ mm^2$

$D = 2.95 \times 24 \times 30.4 = 2152.32$ kN

Therefore,

$P_u = 2152.32 + 1.3 \times 682 = 3038.5$ kN

To calculate the depth of the neutral axis, the following must be satisfied,

$C - T = P_u$

Using the same approach of Section 3.1.4.1 of this study, and with $\beta_1 = 0.85$ for $f_c' = 30$ MPa, the following equations are considered:

$C = ((\beta_1 c - 1000) \times 300 + 1000 \times 500) \times 0.85 f_c'$

$T = (8.5 - 2 \times c(\text{in meters})) \times (A_{sv}/(\text{meter})) \times f_y$

But preliminary calculations showed that c_c is less than 1000 mm and therefore, lies in the boundary element, as a result, the following expressions for C and T are used:

$C = \beta_1 c \times 500 \times 0.85 f_c'$

$T = 6.5 \times (A_{sv}/(\text{meter})) \times f_y$

It was assumed in that case, the all the distributed reinforcement in the wall web are in tension balancing the area of concrete in compression of the boundary element. It is assumed at this stage, a distributed reinforcement area , $A_{sv} = 2000\ mm^2/m$, (No. 15 bars spaced at 200 mm)

That leads to the equations,

$C = 0.85c \times 500 \times 0.85 \times 30$

$T = 6.5 \times 2000 \times 400$

$C - T = 3038.5$

Solving leads to:

$c = 760.2$ mm,

$C = 8238.66$ kN

$T = 5200$ kN

3.3.3.2 Check for flexure

The resisting moment of the shear wall is determined similar to the method used in the detailing according to CSA in Section 3.2 of this study:

$M_r = A_s f_y(4250 - 500) \times 2 + \beta_1 c \times 500 \times \phi_c f_c' \times (4250 - 0.5\beta_1 c)$

The resisting moment of the wall must more than M_u.

M_u is according to NZS the design moment divided by ϕ, where ϕ equals 0.9 in flexure.

Hence,

$M_u = 59600 / 0.9 = 66222.22$ kN.m

Let's assume 22 – No. 30 reinforcing longitudinal bars. $A_s = 15400\ mm^2$

Thus,

$M_r = 15400 \times 400 \times (4250 - 500) \times 2 + 0.85 \times 760.2 \times 500 \times 0.85 \times 30 \times (4250 - 0.5 \times 0.85 \times 760.2)$

$M_r = 78552.547$ kN.m > 66222.22 kN.m O.K.

Therefore, use 26 – No. 30 bars.

ρ_l, defined as the ratio of steel to the gross area of the section is found to be:

$$\rho_l = \frac{15400}{500 \times 1000} = 0.0308$$

ρ_l is well within the code reinforcement limitations determined in Section 3.3.2

3.3.3.3 Check for Ductility

NZS defines a flexural overstrength factor $\phi_{o,w}$ similar in purpose and in definition to the γ_w ratio, defined by CSA. $\phi_{o,w}$ is defined to be the ratio of moment of resistance at overstrength, $M_{o,w}$, to moment from specified earthquake forces, M_E, where both moments refer to the base section of wall. CSA defined the moment of resistance at overstrength as the probable moment carried out with $\phi_s = \phi_c = 1.0$ and using equivalent yield strength of steel of $1.25f_y$. NZS determines $M_{o,w}$ as equal to M_r multiplied by a factor λ_o related to the yield strength of steel since the overstrength primarily results from reinforcement actual yield strength above the specified nominal value.

λ_o is defined as,

$\lambda_o = \lambda_1 + \lambda_2$

where, λ_2 is taken to be 1.25 for $f_y = 400$ MPa,

λ_1 usually depends on the manufacturing supplier of reinforcing steel. A value of 0.15 is an appropriate assumption for this example.

Hence,

$\lambda_o = 0.15 + 1.25 = 1.4$

The flexural overstrength factor for this wall becomes

$$\phi_{o,w} = \frac{M_{o,w}}{M_E} = \frac{1.4 \times 78552.547}{59600} = \frac{109973.57}{59600} = 1.845$$

In order to evaluate the ductility of the shear wall, NZS defines a factor, μ_Δ, as the displacement ductility capacity calculated as:

$\mu_\Delta = \Delta / \Delta_y$

where Δ_y is the yield deflection, and Δ is the sum of the yield deflection Δ_y and the fully plastic deflection Δ_p.

The values used for μ_Δ in the design are selected according to the desired ductility demand of the shear wall, that will depend on the displacement and on the strength of the shear wall. Shear walls designed as ductile reinforced concrete shear walls, are assigned to have a displacement ductility capacity varying between 3.5 and 8.0. The most popular μ_Δ used for these types of walls is 4.0

Hence, it will be assumed,

$\mu_\Delta = 4.0$

The ductility capacity is considered adequate according the NZS if the depth of the neutral axis, c, is less than the critical value c_c. In the case where c is greater than c_c, confinement of concrete in the boundary elements is required. Section 12.4.4.5 of NZS gives a minimum conservative value of c_c equal to $(0.3\phi_c / \mu)l_w$, However, the code permits making allowance in proportions of excess or deficiency of flexural strength and ductility demands, therefore, it is more convenient to relate c_c to the flexural overstrength and the expression mentioned can be modified to:

$$c_c = \frac{M_{o,w}}{2.2\lambda_o \mu_\Delta M_E} l_w$$

$$c_c = \frac{109973.57}{2.2 \times 1.4 \times 4 \times 59600} \times 8500 = 1273mm > c = 760.2mm$$

Therefore, confinement of concrete to ensure the adequacy of ductility is not required. However, the code specifies confinement to prevent buckling of the principle vertical bars. Confinement requirements are discussed in section 3.3.6 in this chapter.

3.3.4 Dimensional Limitations

NZS provides strict and detailed limitations on the dimensions of the shear wall that cannot be found in CSA or ACI codes. Section 12.4.2.1 determines the purpose of the dimensional limitations as: "To safeguard against premature out-of-plane buckling in the potential plastic hinge region of walls." Those limitation apply to walls with thin sections, and more than 2 stories high, according to the same section of NZS.

Section 12.4.2.2 requires the following:

Where one layer of reinforcement near each face of a wall is used, the thickness in the boundary region of the wall section, b_m , exceeding at least over the full height of the first story, shall not be less than:

$$b_m = \frac{K_m(\mu_\Delta + 2)(A_r + 2)l_w}{1700\sqrt{\xi}}$$

A_r is the aspect ratio of wall equals to h_w/l_w (h_w is the total height of wall)

$A_r = 30.4/8.5 = 3.576$

$K_m = 1.0$, unless it can be shown that for long walls

$$K_m = \frac{l_n}{(0.25 + 0.055A_r)l_w}$$

Where l_n is the clear vertical distance between floors

Long walls are walls with an aspect ratio of less than 2.

Therefore, $K_m = 1.0$

$$\xi = 0.3 - \frac{\rho_l f_y}{2.5 f_c'} > 0.1$$

Since those limitations apply only to the boundary regions, ρ_l was determined to be 0.0308 for that section of the wall. Therefore,

$$\xi = 0.3 - \frac{0.0308 \times 400}{2.5 \times 30} = 0.164 > 0.1$$

Hence,

$$b_m = \frac{1.0\times(4+2)\times(3.576+2)\times 8500}{1700\times\sqrt{0.164}} = 412.73mm$$

Section 12.4.2.4 also requires that when boundary elements are provided to fulfil the above requirements, the gross area of the boundary element, A_{wb} , shall satisfy the following limitations

$$A_{wb} \geq b_m^2$$
$$A_{wb} \geq \frac{b_m l_w}{10}$$

$(b_m)^2 = 412.73^2 = 170346\ mm^2$,

$(b_m l_w/10) = (412.73\times8500) / 10 = 350820.5\ mm^2$, governs.

$A_{wb} = 500\times1000 = 500000\ mm^2 > 350820.5\ mm^2$

Section 12.4.2.5 of NZS requires that: "The ratio of the wall thickness in the boundary regions to the clear height of a story for walls designed for ductility shall not be less than $0.04(1 + \mu_\Delta/10)$".

$0.04(1 + \mu_\Delta/10) = 0.04\times(1 + 4/10) = 0.056$

Comparing to the smallest thickness to height ratio, the largest story height which is 4.85 meters is considered,

$500 / 4850 = 0.103 > 0.056$

Therefore, the shear wall satisfies all the dimensional limitations of NZS

3.3.5 Shear strength

The shear strength required for reinforcement design, V_u, is the design earthquake shear on the wall, V_E, times $\phi_{o,w}$, in an approach similar to CSA that assumes that the ratio of the shear to the moment remains constant as the moment increases to its over strength value (Paulay, 1992). Where V_E is the design shear divided by ϕ. However, according to NZS, V_u shall not be taken greater than $\mu_\Delta V_E$. The reason is that in the case where flexural strength of the wall is well in excess of that is required, no flexural ductility demand will arise and it will be unnecessary to design such a wall for shear which would be in excess of the elastic response demand that is assumed to be equal to $\mu_\Delta V_E$

$$V_E = \frac{V}{\phi} = \frac{3255}{0.85} = 3829kN$$

$$V_u = \phi_{o,w} \times V_E = 1.845 \times 3829 = 7064kN$$

The shear stress is given by NZS as,

$$v_i = \frac{V_u}{b_w d_v}$$

Where d_v is the effective depth defined by NZS equals $0.8l_w$

$$d_v = 0.8 \times 8500 = 6800mm$$

$$v_i = \frac{7064 \times 10^3}{300 \times 6800} = 3.46MPa$$

NZS requires that the shear stress should be limited as follows,

$$v_{i,\max} \le \begin{cases} \left(\dfrac{0.22\phi_{o,w}}{\mu_\Delta} + 0.03\right) f_c' \\ 0.16 f_c' \\ 6MPa \end{cases}$$

The reason for that limit is to avoid having significant shear acting on the boundary element in the case of the failure of wall web, because web crushing will eventually spread over the entire length of the wall.

Therefore,

$$v_{i,\max} \leq \begin{cases} \left(\dfrac{0.22 \times 1.845}{4} + 0.03\right) \times 30 = 3.944 \\ 0.16 \times 30 = 4.8MPa \\ \\ 6MPa \end{cases}$$

Hence, $v_{i,\max} = 3.944$ MPa > 3.46 MPa O.K.

The area of distributed steel A_v with a spacing, s, is given as,

$$\frac{A_v}{s} \geq \frac{(v_i - v_c)b_w s}{f_y}$$

Where v_c is the contribution of the concrete to shear strength, expressed in terms of nominal shear stress, taken as:

$$v_c = 0.6\sqrt{\frac{P_u}{A_g}}$$

Hence,

$$v_c = 0.6\sqrt{\frac{3038.5 \times 10^3}{2950000}} = 0.61MPa$$

Taking the spacing, s, to be in meters,

$$\left(\frac{A_v}{s}\right)_{Required} = \frac{(3.46-0.61)\times 300}{400}\times 10^3 = 2137.5 mm^2/m$$

Therefore, the assumed 2000 mm^2/m provided by No. 15 distributed bars at 200 mm should be slightly increased. A spacing of 180 mm is adequate.

$A_{v\,provided} = 2\times 200 / 0.18 = 2222.22$ mm^2

$$\rho_l = \frac{2000}{300\times 1000} = 0.00667$$

ρ_l for the distributed reinforcement is within the code limitations.

3.3.6 Confinement of Concentrated reinforcement

In the case where the depth of the neutral axis, c, exceeds the critical value, c_c, confinement of concrete in longitudinal reinforcement region is required.

Section 12.4.4.5 of NZS 3101 requires:

Where the neutral axis depth in the potential yield region of a wall, computed for the appropriate design forces for the ultimate limit state, exceeds c_c, the following requirement shall be satisfied in that part of the wall section which is subjected to compression due to the design forces:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used as in confined columns such as:

$$A_{sh} = \left(\frac{\mu_\Delta}{40}+0.1\right)s_h h'' \frac{A_g^*}{A_c^*}\frac{f_c'}{f_y}\left(\frac{c}{l_w}-0.07\right)$$

where,

s_h is the center-to-center spacing of horizontal hoops h'' is the dimension of the concrete core perpendicular to the direction of the cross ties

A_g^* is the gross area of concrete section extending over outer length, c' defined in (b).

A_c^* is the area of the concrete core extending over c'

(b) The length of the confined region of the compressed wall section c' shall be such that:

$c' \geq c - 0.7c_c$ but not less than 0.5c

(c) Longitudinal bars shall be restrained against possible buckling in accordance with the code requirements

(d) The centre-to-centre spacing of hoops along longitudinal bars shall not exceed 6 times the diameter of the longitudinal bar, or ½ of the wall thickness in the confined region

(e) Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie (...)

It was seen in Section 3.3.3.3 that the calculated depth of the neutral axis, c = 760.2 mm, was less than the critical value, c_c = 1273 mm. Therefore, the confinement provision of Section 12.4.4.5 of NZS described above, are not required. However, Section 12.4.4.3 requires the use of lateral ties only to retrain the longitudinal bars against possible buckling. It determines that:

"In region of potential compression yielding of the longitudinal reinforcement within a wall with 2 layers of reinforcement, where the vertical reinforcement ratio ρ_l computed from $\rho_l = \Sigma A_b/bs_v$ exceed $2/f_y$, transverse tie reinforcement satisfying the following requirements shall be provided

(a) Ties suitably shaped shall be so arranged that each longitudinal bar or bundle of bars, placed close to the wall surface, is restrained against buckling by a 90° bend or at least a 135° standard hook of a tie. Where 2 or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement.

(b) The area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, shall be computed from the following equation:

$$A_{te} = \frac{\sum A_b f_y}{16 f_{yt}} \frac{s}{100}$$

Where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 12.4.4.3(a). Longitudinal bars centred more than 75 mm from the inner face of stirrup ties need not be considered in determining the value of ΣA_b

(c) The spacing of the ties along the longitudinal bars shall not exceed 6 times the diameter of the longitudinal bar to be restrained."

In this case, an assumed wall section of the boundary element is shown in Figure 3.6 below,

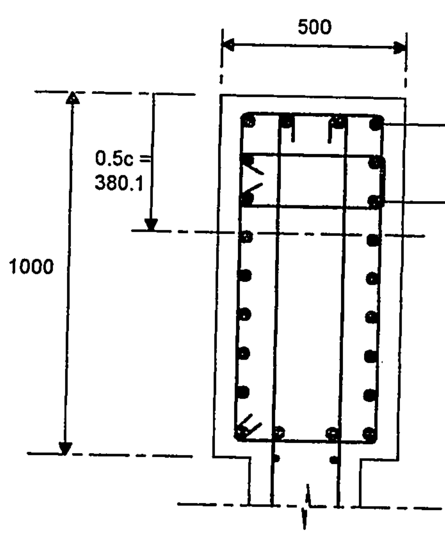


Figure 3-6 Assumed section of the boundary element showing the distribution of vertical steel. (Dimensions in mm)

having 22 – No. 30 bars, with 9 bars in the two long direction and 4 in the short direction all equally spaced. With a concrete clear cover of 40 mm and No. 15 assumed lateral hoops, the spacing between two bars centre-to-centre will be 107.5 mm < 200 mm. and the distance between three bars shown in Figure 3-6 is more than 200 mm, therefore, if $\rho_l = \Sigma A_b/bs_v$ exceed $2/f_y$, all the vertical bars need to be tied over the distance of 0.5c as required by section 12.4.4.3 of NZS.

Since it will be required to have two bars tied with one cross leg,

$\Sigma A_b = 2\times700 = 1400\ mm^2$

$$\rho_l = \frac{1400}{500 \times 215} = 0.013$$

and,

$2/f_y = 2/400 = 0.005 < \rho_l$

$$A_{te} = \frac{1400}{16}\frac{107.5}{100} = 94mm^2$$

The vertical spacing required by section 12.4.4.3 should not exceed six times the bar diameter that is,

$6\times30 = 180$ mm

Hence, use No. 15 cross legs with a vertical spacing of 180 mm. See Figure 3.7 for complete section detailing.

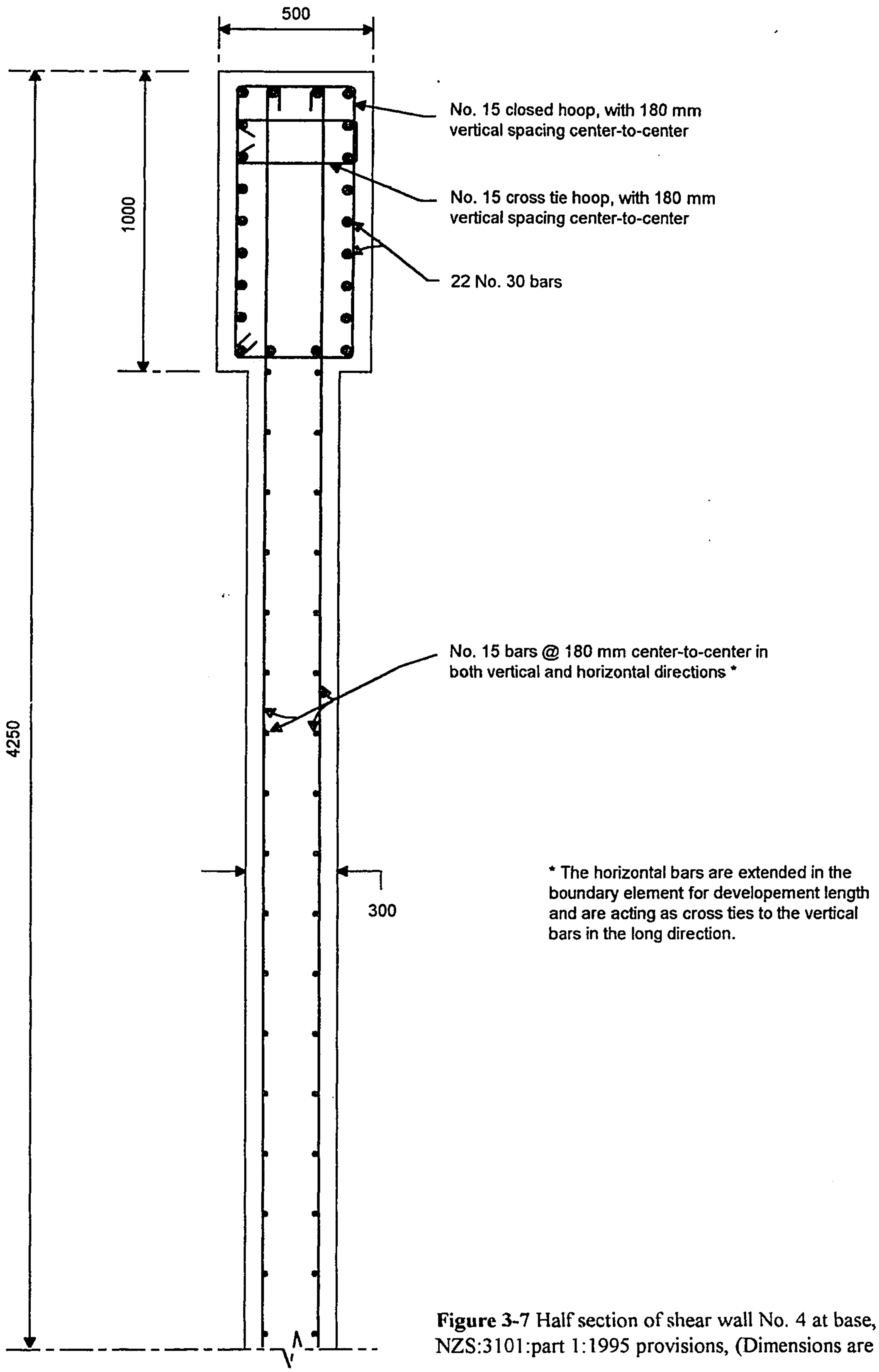


Figure 3-7 Half section of shear wall No. 4 at base, NZS:3101:part 1:1995 provisions, (Dimensions are in mm)

CHAPTER 4

CONCLUSIONS

4.1 Overview

Although the distributed reinforcement in the shear wall were similar in the three designs done according to the three codes, the amount of longitudinal bars with the confinement requirements, provided by the design done according to the ACI, made the reinforcement of the shear wall according to the ACI design requirements the most conservative. Many conclusions could be made on the effectiveness and comparisons of the different design methods used in this study, and also on the differences obtained in the determination of the earthquake loads using the two provisions of NBCC and IBC.

4.2 Flexural capacity

The 26 – No. 35 longitudinal bars provided to resist flexure according to the design of ACI, made the reinforcement significantly larger than that required by the design of the other codes. This was mainly due to the method used in the selecting the reinforcement for the boundary elements that assumed them as short columns acting under factored vertical forces due to gravity and lateral loads (see Figure 2-2). As a result, the boundary elements were designed to have a axial load capacity, $\phi P_n(max)$, equal to some 12600 kN, that required an area of steel of $A_{st} = 260\ cm^2$, that is, 26 – No. 30 bars.

A flexural capacity check was done that assumed the shear wall acting as a cantilever beam subjected to pure bending, and it was shown that the steel, of 26 – No. 35 bars provided in the boundary element, was able to resist flexural bending alone without any axial loads! This doesn't mean that ACI requires not to take into account the axial loads effect when checking for flexure, on the contrary, section R21.6.5.1 of the ACI Commentary clearly determines: "Flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces." Moreover, most of the references, that follow the ACI procedures, recommend a design check for flexure and axial loads using a $\phi P_n - \phi M_n$ interaction diagram for the shear wall section (generally obtained using a computer program for generating P – M diagrams). But ACI doesn't really give details and it doesn't clearly defines the "applied axial loads" that needs to be taken into consideration. It is evident that the philosophy of Section R9.2.3 of ACI for earthquake loads (see Appendix E), is moving from "service-level seismic force", with a load factor of 1.4 or 1.43 to "strength-level seismic force" with a load factor of unity. This change in philosophy will require changes to the required strength.

A different approach is required in the design done according to CSA and NZS codes. In their cases, it was required by the code to check the ability of the shear wall to resist flexure by calculating the resisting moment of the wall. It was clearly determined in the codes that the resisting moment is the capacity of the shear wall to resist both the bending moment resulting from the earthquake and the axial load defined as the sum of the assigned un-factored dead load plus live load in the CSA, and as the sum of the assigned dead load plus a reduced live load in the NZS.

The resisting moment could be determined using interaction diagrams, but there are simplified procedures available to calculate the depth of the neutral axis of the shear wall section, like the one used in this study. Having the depth of the neutral axis calculated, it is possible to determine the resisting moment using the stress block factors, strain compatibility and a maximum compressive strain of 0.0035, as that was done in Sections 3.2.4.2 and 3.3.3.2 of this study. The difference in the required amount of steel between the two codes is due to the difference in the stress block factors and the strength reduction factors used for concrete and steel used by CSA.

4.3 Shear Capacity

The CSA and NZS had higher requirements for the distributed reinforcement than ACI. This mainly due to the fact that the shear design force was required by the CSA and the NZS to be the probable shear or the over-strength shear rather than just the assigned earthquake lateral force on the wall, V_E. This shear is calculated following by multiplying the value of V_E by an overstrength factor, which is the ratio of the probable moment, or the overstrength moment, to the resisting moment.

It is noted here, that for any overdesign longitudinal flexural reinforcement provided, the probable or overstrength moment will increase, and this will result in a higher shear capacity requirements to the wall since the shear design is determined by multiplying V_E by the overstrength factor. However, according to the ACI design procedures, there is no direct relation between the nominal shear strength of the wall and the amount of flexural reinforcement.

4.4 Ductility Requirements

It was obvious throughout this study, that the CSA and NZS codes assume that the shear walls could have a significant ductility. In fact, NZS adopts the idea that: "the principles for frames are generally also applicable to structural walls and that it is relatively easy to dissipate seismic energy in a stable manner". (Paulay, 1992, p. 362) On the other hand, the design done according to ACI generally assumes that the aim of shear walls is to make the buildings stiffer than framed structures in order to reduce the excessive lateral displacements.

4.5 Confinement of Vertical bars

Empirical formulas and expressions related to confinement of vertical bars are very similar in the three codes and they are the same in the ACI and CSA codes. But it was clearly noticed in this study that ACI requires confinement of vertical bars in the boundary elements in all cases unlike the CSA and NZS that require special confinement only when in the case where the calculated depth of the neutral axis exceeds the critical value c_c. Both CSA and NZS give expressions to calculate c_c which is not mentioned in ACI.

It was seen in this study that confinement of longitudinal reinforcement was not required in the design done according to CSA and NZS while it had to be done according to ACI. However, NZS was unique in defining another purpose of reinforcement confinement and gave special confinement requirements to prevent buckling of the vertical bars, which was done in Section 3.3.6 of this study.

4.6 Development length

A special importance was given by ACI to the development length of bars. Section R21.6.2 of ACI commentary described the importance of developing: "Because the actual forces in the longitudinal reinforcing bars of stiff members may exceed the calculated forces, it is required (21.6.2.3) that all continuous reinforcement be developed fully." (It was noticed here the description of shear walls as "stiff members.") ACI gives an expression for the development length l_d for seismic design as shown in Section 3.2.8 of this study, the normal expression of l_d is satisfactory in the design requirements of CSA and NZS. Section 12.4.7 of NZS provides however, some general principles related to special splice and anchorage requirements.

4.7 Stability and the use of boundary elements

NZS had the most strict dimensional limitations to ensure stability and to "safeguard against premature out-of-plane buckling" (Section 12.4.2.1 of NZS) as it was shown in Section 3.3.4 of this study. NZS also defined and required the use of boundary elements in this particular section to ensure that the web thickness is adequate. CSA also have detailed stability requirements but it doesn't have any special requirements for the use of the boundary elements. CSA doesn't give any difference to walls with and without boundary elements and requires the same design procedures for all types of shear walls, while ACI have special consideration and requirements to the use of boundary elements as it was shown in Section 3.1.2. Moreover, Section R21.6.6.3 of ACI commentary determines some special consideration to shear walls with boundary elements: "the wall is considered to be acted on by gravity loads W and the maximum shear and moment induced by earthquake in a given direction. Under this loading, the compressed boundary at the critical section resists the tributary gravity load plus the compressive resultant associated with the bending moment." Those considerations were implemented in Section 3.1.3 of this study and the effects were described earlier in this chapter, and it was clearly shown that ACI requires the boundary elements alone to act as short columns resisting the maximum axial loads as well as flexural loads.

4.8 Earthquake loads

The earthquake loads obtained with the provisions of IBC-2000 were significantly higher than those obtained with the provisions of NBCC. It was also seen that the old base shear expression given in the UBC-1997, which is the previous version of IBC, gave results that are close to those of NBCC. The new expression of the base shear in the IBC code is directly related to the peak ground spectral acceleration for a period of 0.2 seconds, S_s, giving the base shear that could result from an actual earthquake that has the probability of exceedance of 10% in 50 years. This approach obviously leads to high value of the base shear, in addition, it is possible that the assumed value of S_s used in this study could be too conservative. To conclude, it was seen that seismic provisions of the American codes, for both earthquake loads and reinforced concrete design of shear walls, are more conservative than the Canadian provisions. The new expression of the base shear given by IBC sacrificed the need of the economy in the design; therefore, a less conservative reinforcement design of the shear walls recognizing the high effectiveness of those members to act in a ductile manner, is now necessary

APPENDIX A

Tables and Figures Relevant to Seismic loads determination according to NBCC-1995

Case	Type of Lateral-Force-Resisting System	R
	Steel Structures Designed and Detailed According to CAN/CSA-S16.1-M	
1	ductile moment-resisting frame	4.0
2	ductile eccentrically braced frame	4.0
3	ductile steel plate shear wall	4.0
4	ductile braced frame	3.0
5	moment-resisting frame with nominal ductility	3.0
6	nominally ductile steel plate shear wall	3.0
7	braced frame with nominal ductility	2.0
8	ordinary steel plate shear wall	2.0
9	other lateral-force-resisting systems not defined in Cases 1 to 8	1.5
	Reinforced Concrete Structures Designed and Detailed According to CSA A23.3	
10	ductile moment-resisting frame	4.0
11	ductile coupled wall	4.0
12	other ductile wall systems	3.5
13	moment-resisting frame with nominal ductility	2.0
14	wall with nominal ductility	2.0
15	other lateral-force-resisting systems not defined in Cases 10 to 14	1.5
	Timber Structures Designed and Detailed According to CSA O86.1	
16	nailed shear panel with plywood, waferboard or OSB	3.0
17	concentrically braced heavy timber frame with ductile connections	2.0
18	moment-resisting wood frame with ductile connections	2.0
19	other systems not included in Cases 16 to 18	1.5
	Masonry Structures Designed and Detailed According to CSA S304.1	
20	reinforced masonry wall with nominal ductility	2.0
21	reinforced masonry	1.5
22	unreinforced masonry	1.0
23	Other Lateral-force-resisting Systems not Defined in Cases 1 to 22	1.0

Table A-1 Force modification factors R (Table 4.1.9.1.B. of NBCC-1995)

Categories	Type and Depth of *Rock* and *Soil* Measured from the *Foundation* or *Pile* Cap Level	F
1	*Rock*, dense and very dense coarse-grained *soils*, very stiff and hard fine-grained *soils*; compact coarse-grained *soils* and firm and stiff fine-grained *soils* from 0 to 15 m deep	1.0
2	Compact coarse-grained *soils*, firm and stiff fine-grained *soils* with a depth greater than 15 m; very loose and loose coarse-grained *soils* and very soft and soft fine-grained *soils* from 0 to 15 m deep	1.3
3	Very loose and loose coarse-grained *soils* with depth greater than 15 m	1.5
4	Very soft and soft fine-grained *soils* with depth greater than 15 m	2.0

Table A-2 Foundation factors (Table 4.1.9.1.C. of NBCC-1995)

T	Z_a/Z_v	S
≤ 0.25	> 1.0	4.2
	1.0	3.0
	< 1.0	2.1
> 0.25 but < 0.50	> 1.0	4.2 – 8.4 (T – 0.25)
	1.0	3.0 – 3.6 (T – 0.25)
	< 1.0	2.1
≥ 0.50	All values	$1.5/T^{1/2}$

Table A-3 Seismic response factors (Table 4.1.9.1.A. of NBCC-1995)

Province and Location	Elev., m	Design Temperature January 2.5% °C	January 1% °C	July 2.5% Dry °C	July 2.5% Wet °C	Degree-Days Below 18°C	15 Min. Rain, mm	One Day Rain, mm	Ann. Tot. Ppn., mm	Ground Snow Load, kPa S_s	S_r	Hourly Wind Pressures 1/10 kPa	1/30 kPa	1/100 kPa	Seismic Data Z_a	Z_v	Zonal Velocity Ratio, v
West Vancouver	45	-8	-10	28	19	3250	9	140	1700	2.2	0.2	0.36	0.44	0.53	4	4	0.20
Vernon	405	-20	-23	33	20	3900	13	40	400	2.0	0.1	0.32	0.39	0.49	1	1	0.05
Victoria Region																	
Victoria (Gonzales Hts)	65	-5	-7	23	17	2900	9	85	625	1.4	0.3	0.49	0.58	0.69	6	5	0.30
Victoria (Mt Tolmie)	125	-6	-8	24	16	3050	9	85	800	1.9	0.3	0.49	0.58	0.69	6	5	0.30
Victoria	10	-5	-7	24	17	2950	8	85	825	1.0	0.2	0.48	0.58	0.70	6	5	0.30
Williams Lake	615	-31	-34	29	17	5100	10	45	425	2.2	0.2	0.30	0.35	0.41	1	2	0.10
Youbou	200	-5	-7	31	19	3200	10	150	2100	3.5	0.6	0.46	0.55	0.66	4	4	0.20
Alberta																	
Athabasca	515	-35	-38	28	19	6000	18	80	480	1.4	0.1	0.30	0.37	0.45	0	1	0.05
Banff	1400	-30	-32	27	17	5500	18	60	500	3.3	0.1	0.39	0.45	0.52	0	1	0.05
Barrhead	645	-34	-37	28	19	6000	20	80	475	1.6	0.1	0.32	0.39	0.49	0	1	0.05
Beaverlodge	730	-35	-38	28	18	5900	25	85	470	2.2	0.1	0.27	0.33	0.40	0	1	0.05
Brooks	760	-32	-34	32	19	5200	18	80	340	1.1	0.1	0.39	0.48	0.57	0	0	0.00
Calgary	1045	-31	-33	29	17	5200	23	95	425	1.0	0.1	0.40	0.46	0.54	0	1	0.05
Campsie	660	-34	-37	28	19	6000	20	80	475	1.6	0.1	0.32	0.39	0.49	0	1	0.05
Camrose	740	-33	-35	29	19	5700	20	85	470	1.8	0.1	0.30	0.37	0.45	0	0	0.00
Cardston	1130	-30	-33	29	18	4750	20	100	550	1.4	0.1	0.74	0.93	1.15	0	0	0.00
Claresholm	1030	-31	-34	29	18	4800	15	95	440	1.2	0.1	0.66	0.80	0.96	0	0	0.00
Cold Lake	540	-36	-38	28	20	6100	15	75	430	1.6	0.1	0.31	0.37	0.44	0	0	0.00
Coleman	1320	-31	-34	28	18	5300	15	70	550	2.5	0.3	0.54	0.69	0.87	1	1	0.05
Coronation	790	-31	-33	30	19	5800	20	85	400	2.0	0.1	0.23	0.32	0.43	0	0	0.00
Cowley	1175	-31	-34	29	18	5100	15	75	525	1.5	0.1	0.73	0.91	1.13	0	1	0.05
Drumheller	685	-31	-33	29	18	5300	20	80	375	1.1	0.1	0.32	0.39	0.49	0	0	0.00
Edmonton	645	-32	-34	28	19	5400	23	90	460	1.6	0.1	0.32	0.40	0.51	0	1	0.05
Edson	920	-34	-37	28	18	5900	18	75	570	1.9	0.1	0.36	0.43	0.50	0	1	0.05
Embarras Portage	220	-41	-44	27	19	7100	10	80	390	1.7	0.1	0.31	0.37	0.45	0	0	0.00
Fairview	670	-38	-40	27	18	6050	15	80	450	2.4	0.1	0.26	0.32	0.39	0	1	0.05
Fort MacLeod	945	-31	-33	31	18	4600	16	90	425	1.1	0.1	0.68	0.83	1.00	0	0	0.00
Fort McMurray	255	-39	-41	28	19	6550	13	85	460	1.3	0.1	0.27	0.32	0.38	0	0	0.00
Fort Saskatchewan	610	-32	-35	28	19	5700	20	80	425	1.5	0.1	0.31	0.39	0.49	0	1	0.05
Fort Vermilion	270	-41	-43	28	18	6900	13	60	380	1.9	0.1	0.22	0.26	0.32	0	1	0.05
Grande Prairie	650	-36	-39	27	18	6000	23	80	450	2.0	0.1	0.37	0.44	0.52	0	1	0.05
Habay	335	-41	-43	28	18	7150	13	65	425	2.2	0.1	0.21	0.26	0.31	0	1	0.05
Hardisty	615	-33	-35	30	19	5900	20	70	425	1.6	0.1	0.24	0.32	0.42	0	0	0.00
High River	1040	-31	-33	28	17	5300	18	95	425	1.2	0.1	0.51	0.60	0.72	0	1	0.05
Hinton	990	-34	-38	27	17	5700	13	75	500	2.7	0.1	0.36	0.43	0.50	0	1	0.05
Jasper	1060	-32	-35	28	18	5500	10	70	400	3.0	0.1	0.37	0.43	0.50	1	1	0.05
Keg River	420	-40	-42	28	18	6800	13	60	450	2.2	0.1	0.21	0.26	0.31	0	1	0.05
Lac la Biche	560	-35	-38	28	19	6150	15	80	475	1.5	0.1	0.31	0.37	0.44	0	0	0.00
Lacombe	855	-33	-35	29	18	5700	23	85	450	1.9	0.1	0.30	0.37	0.45	0	1	0.05
Lethbridge	910	-30	-33	31	18	4650	20	90	390	1.1	0.1	0.64	0.76	0.91	0	0	0.00
Manning	465	-39	-41	27	18	6700	13	75	390	2.1	0.1	0.21	0.26	0.32	0	1	0.05
Medicine Hat	705	-31	-34	33	19	4750	23	85	325	1.0	0.1	0.39	0.49	0.60	0	0	0.00
Peace River	330	-37	-40	27	18	6350	15	60	390	2.0	0.1	0.24	0.29	0.36	0	1	0.05

Table A-4 Climatic and seismic data that include Victoria as given in Appendix C of NBCC-1995

APPENDIX B

Tables and Figures Relevant to Seismic loads determination according to IBC-2000

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 FT (30 M), AS PER SECTION 1615.1.5		
		SOIL SHEAR WAVE VELOCITY, $\bar{v}_s$, (ft/s)	STANDARD PENETRATION RESISTANCE, $\bar{N}$	SOIL UNCONFINED SHEAR STRENGTH, $\bar{s}_u$ (psf)
A	Hard Rock	$\bar{v}_s > 5,000$	not applicable	not applicable
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	not applicable	not applicable
C	Very Dense Soil and Soft Rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$\bar{s}_u \geq 2,000$
D	Stiff Soil Profile	$600 \leq \bar{v}_s \leq 1,200$	$15 \leq \bar{N} \leq 50$	$1,000 \leq \bar{s}_u \leq 2,000$
E	Soft Soil Profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1,000$
E		Any profile with more than 10 ft of soil having the following characteristics: - plasticity index PI > 20; - moisture content $w \geq 40\%$, and - unconfined shear strength $\bar{s}_u < 500$ psf		
F		Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays (H > 10 ft of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H > 25 ft with plasticity index PI >75) 4. Very thick soft/medium stiff clays (H > 120 ft)		

Table B-1 Site class definition (Table 1615.1.1 of IBC-2000)

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a
F	Note a	Note a	Note a	Note a	Note a

NOTE: Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_s.
[a] Site specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

Table B-2 Values of site coefficient F_a as a function of site class and mapped spectral response acceleration at short periods (S_s) (Table 1615.1.2-1 of IBC-2000)

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a
F	Note a	Note a	Note a	Note a	Note a

NOTE: Use straight line interpolation for intermediate values of mapped spectral acceleration at 1 second period, S_1.
[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

Table B-3 Values of site coefficient F_a as a function of site class and mapped spectral response acceleration at 1 second period (S_1) (Table 1615.1.2-2 of IBC-2000)

VALUE OF S_{DS}	SEISMIC USE GROUP		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D[a]	D[a]	D[a]

a Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1 second period ,S_1, equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

Table B-4 Seismic response category based on short period response accelerations (Table 1616.3-1 of IBC-2000)

VALUE OF S_{D1}	SEISMIC USE GROUP		
	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D[a]	D[a]	D[a]

a Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1 second period ,S_1, equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

Table B-5 Seismic response category based on 1 second period response accelerations (Table 1616.3-2 of IBC-2000)

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_0^g	DEFLECTION AMPLIFICATION FACTOR, C_d^b	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FT) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.1				
					A & B	C	D[d]	E[e]	F[e]
Bearing Wall Systems									
Ordinary steel braced frames	Note k (11), 2211	4	2	3½	NL	NL	160	160	160
Special Reinforced concrete shear walls	1910.2.2.4	5½	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	1910.2.2.3	4½	2½	4	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	1910.2.2.2	2½	2½	2	NL	NL	NP	NP	NP
Ordinary plain concrete shear walls	1910.2.2.1	1½	2½	1½	NL	NP	NP	NP	NP
Special reinforced masonry shear walls	2106.1.1.5	4	2½	3½	NL	NL	160	160	100
Intermediate reinforced masonry shear walls	2106.1.1.4	3½	2½	3	NL	NL	NP	NP	NP
Ordinary reinforced masonry shear walls	2106.1.1.2	2	2½	1¾	NL	160	NP	NP	NP
Detailed plain masonry shear walls	2106.1.1.3	2	2½	1¾	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	2106.1.1.1	1½	2½	1¼	NL	NP	NP	NP	NP
Light frame walls with shear panels-Wood Structural Panels	2305.1/ 2211	6½	3	4	NL	NL	160	160	100
Light frame walls with shear panels-Gypsum Board	2305.1	2	2½	2	NL	NL	35	NP	NP
Building Frame Systems									
Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	Note k (15)	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	Note k (13)	6	2	5	NL	NL	160	160	100
Ordinary steel concentrically braced frames	Note k (14)	5	2	4½	NL	NL	160	100	100

Table B-6 Seismic response factor R (Table 1617.6 of IBC-2000)

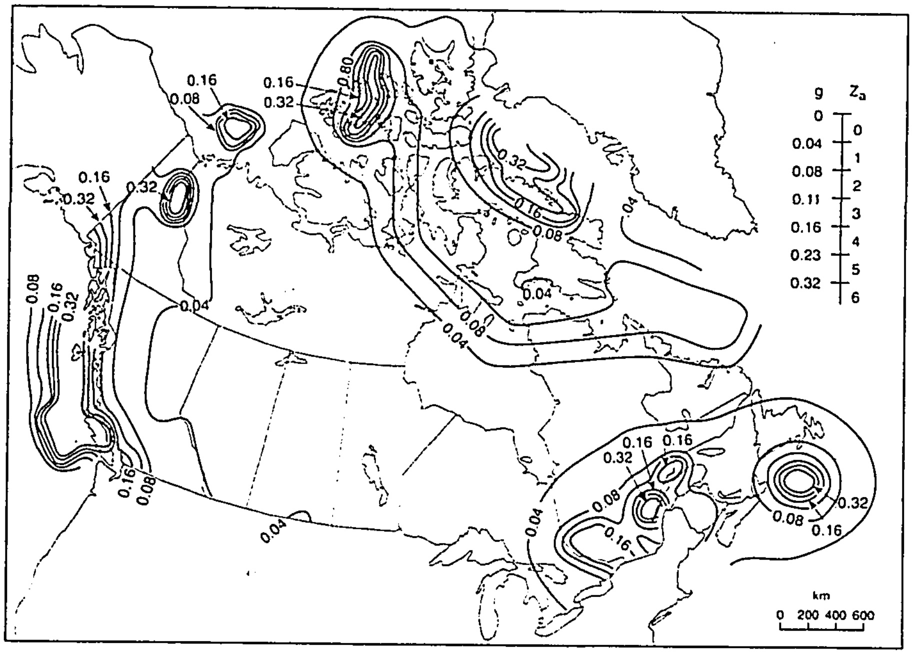


Figure B-1 Contours of peak horizontal ground accelerations of Canada, in units of g, having a probability of exceedance of 10% in 50 years, as given in figure J-1 of NBCC commentary-1995

APPENDIX C

Tables and Figures Relevant to Seismic loads determination according to UBC-1997

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

Table C-1 Seismic Importance factor (Table 16-k of UBC-97)

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[6]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2,8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit
[1]See Section 1630.4 for combination of structural systems.
[2]Basic structural systems are defined in Section 1629.6.
[3]Prohibited in Seismic Zones 3 and 4.
[4]Includes precast concrete conforming to Section 1921.2.7.
[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.
[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use a R value of 8.
[7] Total height of the building including cantilevered columns.
[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

Table C-2 Seismic response factor R (Table 16-N of UBC-97)

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2.

Table C-3 Seismic zone factor (Table 16-I of UBC-97)

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, PI, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

Table C-4 Soil profile types (Table 16-J of UBC-97)

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Table C-5 Seismic coefficient C_v (Table 16-R of UBC-97)

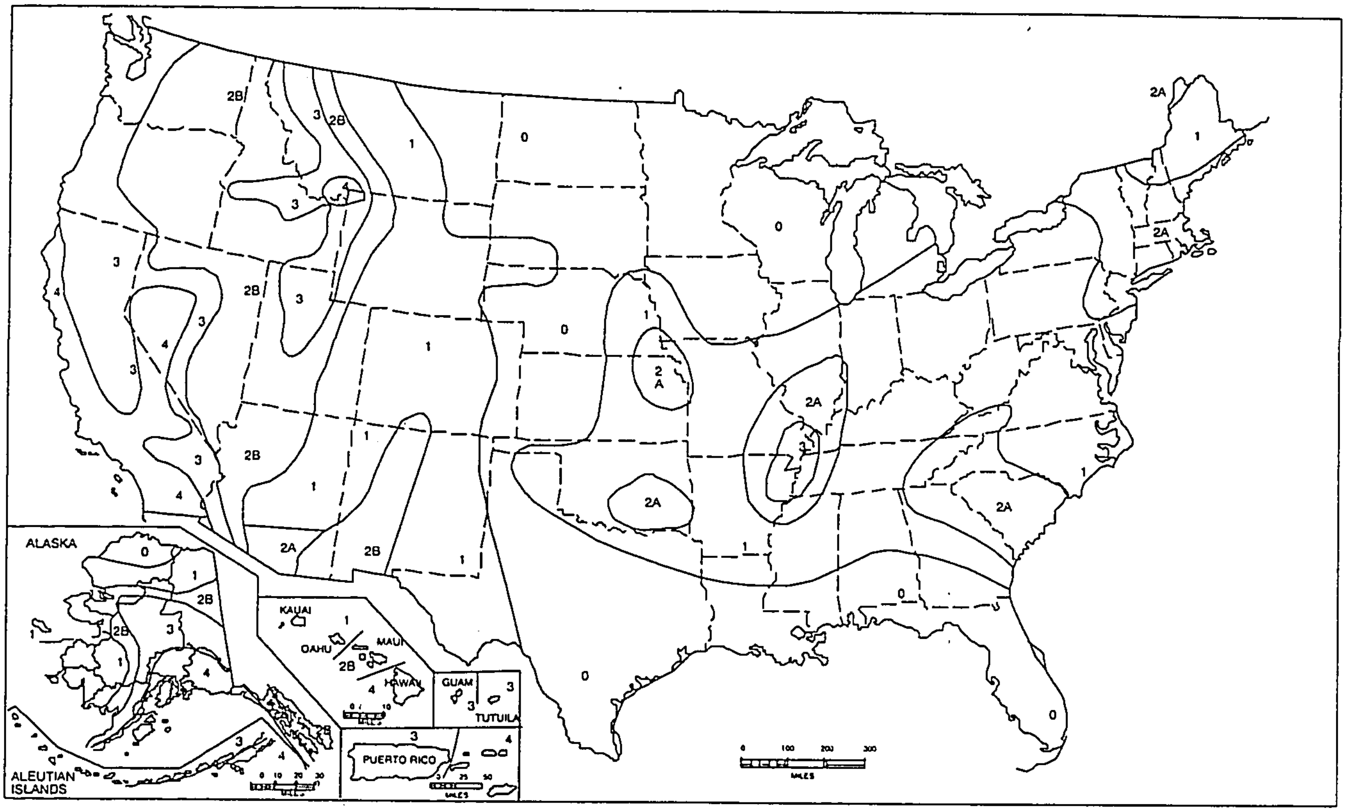


Figure C-1 Seismic zone map of the Unites States, as given in Figure 16-2 of UBC-97

APPENDIX D

Section 12 of NZS3101:1995 (modified 1998) and Relevant Figures from its Commentary (Provided by Professor T. Paulay, 15 February 2000)

12 WALLS

12.1 Notation

A_c^* area of concrete core extending over the outer c' length of the neutral axis depth which is subjected to compression, measured to center of peripheral hoop legs, mm^2

A_g gross area of section, mm^2

A_g^* gross area of concrete section extending over outer c' length of the neutral axis depth which is subjected to compression, mm^2

A_r h_w/L_w aspect ratio of wall

A_s area of vertical wall reinforcement spaced horizontally at s_v, mm^2

A_{sh} total effective area of hoop bars and supplementary cross ties distributed over length h" in the direction under consideration, with vertical spacing s_h, mm^2

A_{te} area of one leg of stirrup-tie, mm^2

A_{wb} gross area of boundary element, mm^2

b width or thickness of wall section, mm

b_m critical thickness of wall, mm

c computed distance of neutral axis from the compression edge of the wall section at the ultimate limit state, mm

c' length of wall section defined by equation 12-9 to be confined by transverse reinforcement, mm

c_c distance of the critical neutral axis from the compression edge of the wall section at the ultimate limit state, mm

f_c' specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yh} lower characteristic yield strength of non-prestressed hoop or supplementary tie reinforcement, MPa

h overall depth of member

h'' dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to center of peripheral hoop, mm

h_w total height of wall from base to top, mm

L_d development length

L_n the clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span, mm

L_w horizontal length of wall, mm

N_{nw} nominal axial load-carrying capacity of a bearing wall designed by 12.3.7, N

ρ_l the ratio of vertical wall reinforcement area to the unit of horizontal gross concrete section = A_s/bs_v

s_h center-to-center spacing of horizontal hoop sets, mm

s_v horizontal spacing of vertical reinforcement along the length of a wall, mm

V_n total nominal shear stress, MPa

μ displacement ductility capacity relied on the design

ΣA_b sum of area of longitudinal bars, mm^2

ϕ_o ratio of moment of resistance at overstrength to moment resulting from specified earthquake forces, where both moments refer to the base section of wall

12.2 Scope

Provisions of this section apply to design of walls subjected to axial load, with or without flexure, and shear.

12.3 General principles and requirements

12.3.1.1 *General design principles*

Walls shall be designed for any vertical loading and/or lateral in-plane and face forces to which they may be subjected.

12.3.1.2

Proper provisions shall be made to eccentric loads

12.3.1.3

Unless designed in accordance with this section, walls subjected to combined flexure and axial loads shall be designed in accordance with the requirements of 8.3.1

12.3.2 *Dimensions Limitations*

12.3.2.1

Thickness of walls subjected to compression stress over the gross area of the wall due to the ultimate limit state axial load exceeding $0.2f_c$' shall not be less than 1/25 the unsupported height.

12.3.2.2

Walls shall not be less than 100 mm thick for the uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), minimum thickness shall be increased by 25 mm. Bearing walls for two-story buildings may be 100 mm thick for the total wall height, provided that the compression stress over the gross area of the wall due to the ultimate state axial load does not exceed $0.2f_c$'

12.3.2.3

Exterior basement walls and foundation walls shall not be less than 100 mm thick

12.3.2.4

Overall thickness of non-loadbearing wall panels and enclosure wall shall not be less than 100 mm, nor less than 1/30 the distance between supporting or enclosing members.

12.3.2.5

Where walls consist of studs or ribs tied together by other reinforced concrete members at each floor roof level, such studs or ribs may be considered as columns.

12.3.2.6

The length of the wall to be considered as effective for each concentrated load or reaction shall not exceed centre-to-centre distance between such loads, nor width of bearing plus 4 times the wall thickness.

12.3.2.7

Limits of thickness and quantity of reinforcement required by 12.3.2 and 7.3.31 respectively may be waived where, instead of the empirical rules of 12.3.6, rational analysis or test shows adequate strength and stability at the ultimate limit state

12.3.3 *Anchorage of walls*

Walls shall be anchored to floors, roofs, columns, pilasters, buttresses, intersecting walls and foundations.

12.3.4 *Foundation walls*

12.3.4.1

Walls designed as foundation beams shall have top and bottom reinforcement as required for moment in accordance with the provisions of 8.3. Design for shear shall be in accordance with provisions of section 9.

12.3.4.2

Portions of foundation walls exposed above ground shall also meet the requirements of section 7 and 12.3.2

12.3.5 *Ties and vertical reinforcement*

Vertical reinforcement need not to be enclosed by lateral ties if the vertical reinforcement area is not greater than 0.01 times the gross concrete area in any locality of the wall section

12.3.6 *Empirical Design*

Gravity load bearing walls that are not required to be designed to respond in a ductile manner may be designed by the empirical provisions of 12.3.7 if the resultant of the axial load at the ultimate limit state is located within the middle third of the overall thickness of the wall and all limits of 12.3.2 are satisfied

12.3.7 *Axial load strength*

Nominal axial load strength, N_{nw}, of a wall, within limitations of 12.3.6, shall be computed by:

$$N_{nw} = 0.55 f_c^{'} A_g \left[1 - \left(\frac{KL_n}{32b} \right)^2 \right]$$

(Eq. 12-1)

The effective length factor k shall be:

For walls braced at the top and bottom against lateral translation and:

(a) Restrained against rotation at the top and/or the bottom 0.8

(b) Unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

12.4 Additional design requirements for earthquake effects

12.4.1 *General Seismic design requirements*

12.4.1.1

Cantilever or coupled structural walls shall be considered as integral units. The strength of flanges, boundary members and webs shall be evaluated on the basis of compatible interaction between these elements using rational analysis. Due allowance for openings in components shall be made.

12.4.1.2

In the design of ductile walls subjected to seismic forces at the ultimate limit state, the requirements of 4.4.6 shall also be satisfied

12.4.1.3

Walls of limited ductility at the ultimate state shall be designed in accordance with section 17

12.4.2 *Dimensional limitations*

12.4.2.1

The safeguard against premature out-of-plane buckling in the potential plastic hinge region of walls with thin sections, more than 2 stories high the limitation of this clause shall apply

12.4.2.2

Where one layer of reinforcement near each face of a wall is used, the thickness in the boundary region of the wall section, exceeding at least over the full height of the first story, shall not be less than:

$$b_w = \frac{K_m(\mu+2)(A_r+2)L_w}{1700\sqrt{\xi}}$$

(Eq. 12-2)

Where K_m = 1.0, unless it can be shown that for long walls

$$K_m = \frac{L_n}{(0.25+0.055A_r)L_w}$$

(Eq. 12-3)

And where

$$\xi = 0.3 - \frac{p_l f_y}{2.5 f_c'} > 0.1$$

(Eq. 12-4)

12.4.2.3

Walls with a single layer of reinforcement shall not be used when $\mu > 4$, and where used, the thickness of the boundary region, b_m, defined in 12.4.2.4 shall be not less than 1.25 times that required by 12.4.2.2.

12.4.2.4

Where 12.4.2.2 and 12.4.2.3 control the thickness of the wall in the boundary element region, an enlarged boundary element shall be provided with gross area, A_{wb}, satisfying the following limitations:

$$b_m^2 \leq A_{wb} \geq \frac{b_m L_w}{10}$$

(Eq. 12-5)

12.4.2.5

The ratio of the wall thickness in the boundary regions to the clear height o a story for walls designed for ductility shall not be less than $0.04(1 + \mu/10)$.

12.4.2.6

Where flanges on either side of the web with width greater than 3 times the flanges thickness, satisfying 12.4.2.4 are used, the flange thickness shall not be less than that required by 12.4.2.5

12.4.3 *Longitudinal reinforcement*

12.4.3.1

The ratio ρ_l of longitudinal reinforcement over any part of a wall shall not be less than $0.7/f_y$ nor more than $16/f_y$

Where lapped splices in boundary elements are unavoidable, the total reinforcement ratio, including the area of splices, shall not exceed $21/f_y$

12.4.3.2

In walls thicker than 200 mm at least 2 layers of reinforcement shall be used, one near each face of the wall.

12.4.3.3

The diameter of the bars used in any part of a ductile wall shall not exceed one tenth of the thickness of the wall

12.4.3.4

Where wide flanges are present in relatively short walls, only the vertical reinforcement placed within a flange width, each side of the web, equal to one half the distance from the section under consideration to the top of the wall shall be considered effective in resisting flexure

12.4.4 *Transverse reinforcement*

12.4.4.1

The requirements for minimum reinforcement ratio, placing of reinforcement, diameter of transverse bars used and their placing shall be in accordance with 7.3.31

12.4.4.2

Transverse reinforcement shall be provided to resist shear resulting from earthquake forces in accordance with 9.3.14.9 and shall be adequately anchored at the wall edges or in the boundary elements as required by 7.5.6 for stirrups in beams or by standard hooks as close to the end of the wall as is practicable

12.4.4.3

In region of potential compression yielding of the longitudinal reinforcement within a wall with 2 layers of reinforcement, where the longitudinal reinforcement ratio ρ_l computed from

$\rho_l = A_s/bs_v$

(Eq. 12-6)

exceed $2/f_y$, transverse tie reinforcement satisfying the following requirements shall be provided:

(a) Ties suitably shaped shall be so arranged that each longitudinal bar or bundle of bars, placed close to the wall surface, is restrained against buckling by a 90° bend or at least a 135° standard hook of a tie. Where 2 or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement.

(b) The area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, shall be computed from equation 8-23 where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 12.4.4.3(a). Longitudinal bars centred more than 75 mm from the inner face of stirrup ties need not be considered in determining the value of ΣA_b

(c) The spacing of the ties along the longitudinal bars shall not exceed 6 times the diameter of the longitudinal bar to be restrained.

12.4.4.4

In those regions of the wall section where the yield strength of the longitudinal bars in tension or compression, required for the nominal strength of the section in accordance with the specified bending moment envelope, cannot be developed and the reinforcement content from equation 12-6 exceeds $2/f_y$, lateral reinforcement shall be provided so that:

(a) The diameter of the ties is not less than ¼ of that of the longitudinal bar to be restrained;

(b) The spacing of ties is not more than 12 times the diameter of the longitudinal bar.

12.4.4.5

Where the neutral axis depth in the potential yield region of a wall, computed for the appropriate design forces for the ultimate limit state, exceeds:

$c_c = (0.3\phi_c / \mu)L_w$

(Eq. 12-7)

the following requirement shall be satisfied in that part of the wall section which is subjected to compression due to the design forces:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used as in confined columns so that:

$$A_{sh} = \left(\frac{\mu}{40} + 0.1\right) s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07\right)$$

(Eq. 12-8)

(b) The length of the confined region of the compressed wall section c' shall be such that:

$c' \geq c - 0.7c_c$ (Eq. 12-9)

but not less than 0.5c

(c) Longitudinal bars shall be restrained against possible buckling in accordance with 12.4.4.3(a) and (b)

(d) The centre-to-centre spacing of hoops along longitudinal bars shall not exceed 6 times the diameter of the longitudinal bar, or ½ of the wall thickness in the confined region.

(e) Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following 2 cases of bars are exempt from this requirement:

i. Bras or bundles of bars that lie between 2 laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed ½ of the adjacent lateral dimension of the cross section.

ii. Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside hoops.

(f) The potential yield region of the wall, over which the requirements for hoops in accordance with 12.4.4.5(a) to (d) is to be satisfied, shall be assumed to extend above the critical section by L_w or 1/6 of the height of the wall measured to the top of the wall, whichever is larger.

(g) The region to be confined shall contain more than one layer of reinforcement.

12.4.4.6

The neutral axis depth in the potential yield regions of walls with a single layer of reinforcement, computed for the approximate design forces for the ultimate limit state, shall not exceed 0.8 times the value obtained with equation 12-7.

12.4.5 *Shear strength*

12.4.5.1

The evaluation of shear strength of, and the determination of shear reinforcement for, walls shall be in accordance with section 9. The maximum design shear force in a wall shall be based on the sum of all the lateral seismic forces introduced at levels above the assumed base level of the structure.

12.4.5.2

The height of the end region of the wall, for which the special shear stress limitations apply, shall be taken as the length of the wall L_w or 1/6 of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not be taken larger than $2L_w$.

12.4.6 *Walls with openings*

12.4.6.1

Openings in structural walls shall be so arranged that unintentional failure planes across adjacent openings, do not reduce the shear or flexural strength of the structure. For ductile cantilever walls with irregular openings appropriate analysis such as based on strut-and-tie models shall establish rational paths for internal forces. Capacity design procedures shall be used to ensure that the horizontal shear reinforcement will not yield before the flexural strength of the wall is developed.

12.4.6.2

Ductile coupled walls shall be connected by ductile coupling beams of stabs. In coupling beams, the entire earthquake induced shear and flexure derived in accordance with 9.4.1.1, shall be resisted by diagonal reinforcement in both directions unless the earthquake induced shear stress is less than:

$$V_n = 0.12 \frac{L_n}{h} \sqrt{f_c^{'}}$$
(Eq. 12-10)

The diagonal reinforcement shall be enclosed by rectangular ties or spirals that satisfy the requirements of 12.4.4.3.

12.4.7 *Special splice and anchorage requirements*

12.4.7.1

The splicing of the principle vertical flexural tension reinforcement in potential areas of yielding in ductile walls shall be avoided if possible. Not more than 1/3 of such reinforcement shall be spliced at the same location where yielding can occur.

12.4.7.2

The stagger between lapped splices shall be not less than twice the splice length, and at least one leg of lateral tie, spaced not further than 10 times the diameter of a longitudinal bar, satisfying the requirements of 7.5.1.2, shall surround lapped bars larger than 16 mm.

12.4.7.3

Mechanical connections and welded splices satisfying the requirements of 7.3.16.5 may be used in potential areas of yielding in walls, provided that not more than ½ of the reinforcement shall be spliced at one section, and the stagger shall not be less than 600 mm. Welded splices satisfying 7.3.16.5(a) or mechanical connections meeting the requirements for stiffness of 7.5.1.3 need not be staggered.

12.4.7.4

When by capacity design procedure or otherwise it can be shown that yielding of wall reinforcement could not occur, only the requirements of 7.3.16.6 need be satisfied

12.4.7.5

When three or more diagonal or horizontal bars of a coupling beam are anchored in adjacent structural walls, the development length shall be $1.5L_d$

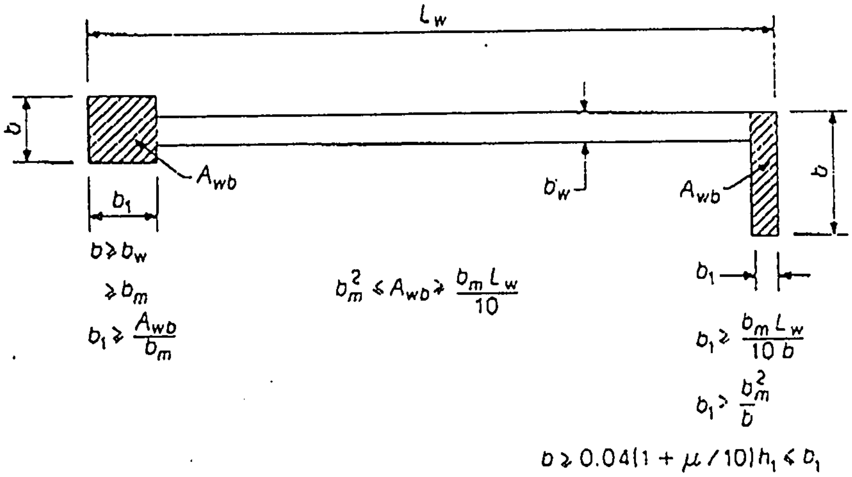


Figure D-1 Minimum dimensions of boundary elements of wall sections in plastic hinge regions (Figure C12.1 of NZS 3101:part2:1995)

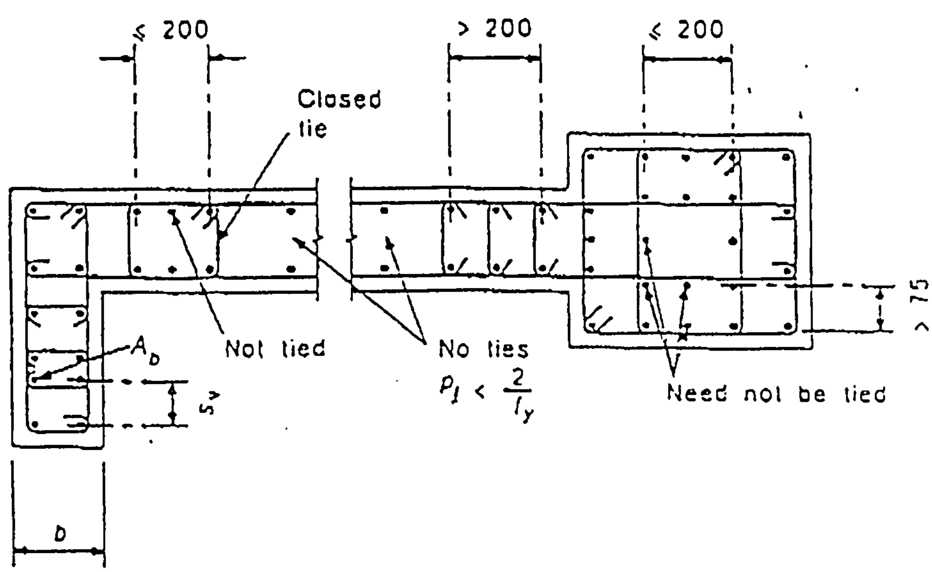


Figure D-2 Example of transverse reinforcement in plastic hinge regions of walls in accordance with Section 12.4.4 (Figure C12.2 of NZS 3101:part2:1995)

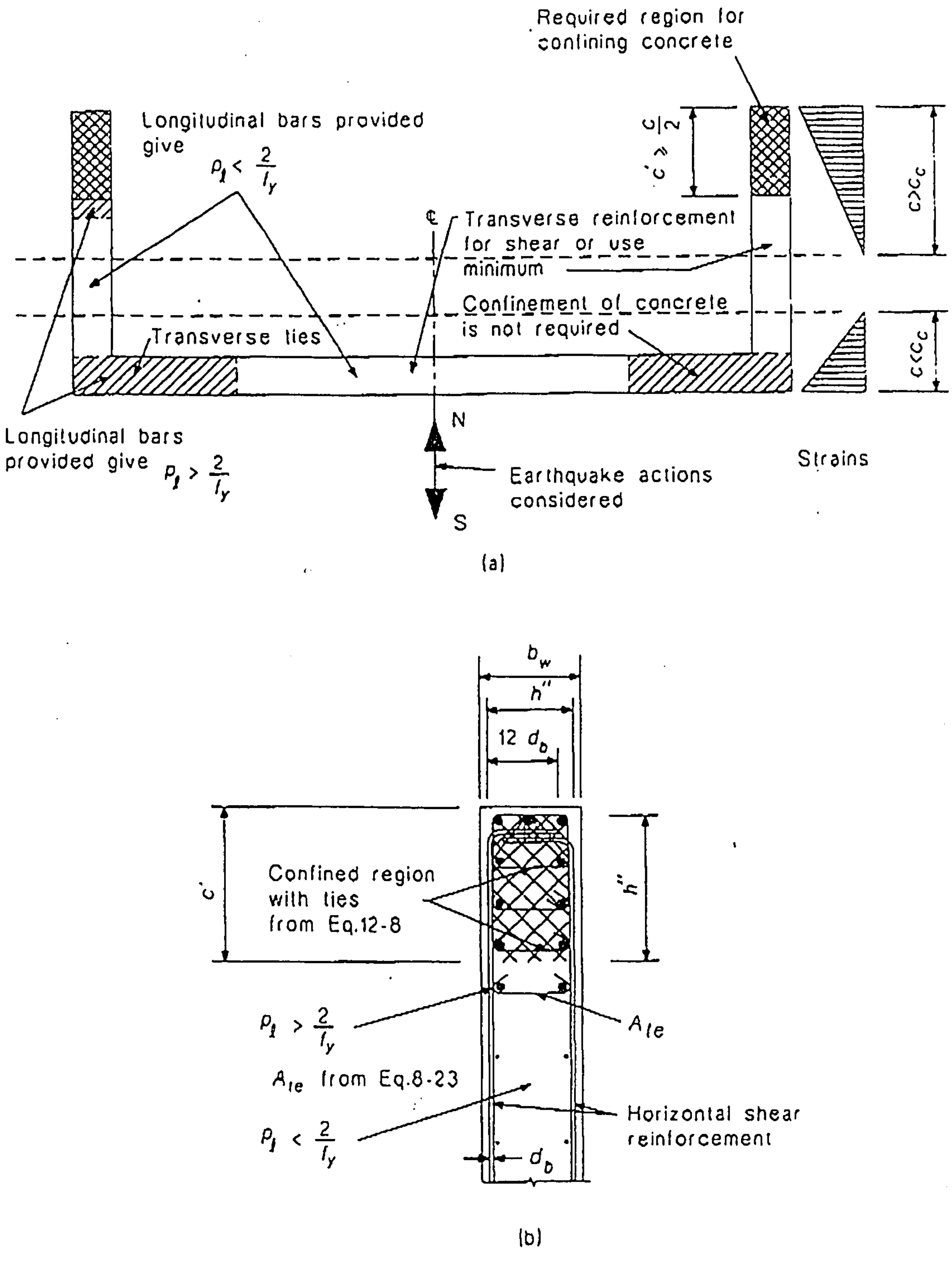


Figure D-3 Regions of transverse reinforcements in accordance with 12.4.4 (Figure C12.3 of NZS 3101:part2:1995)

APPENDIX E

LOAD COMBINATIONS

E.1 Load Combinations of ACI318-99

9.2 – Required strength

9.2.1 – Required strength **U** to resist dead load **D** and live load **L** shall be at least equal to

$$U = 1.4D + 1.7L \qquad (9\text{-}1)$$

9.2.2 – if resistance to structural effects of a specified wind load **W** are included in design, the following combinations of **D**, **L**, and **W** shall be investigated to determine the greatest required strength **U**

$$U = 0.75(1.4D + 1.7L + 1.7W) \qquad (9\text{-}2)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \qquad (9\text{-}3)$$

but for any combination of **D**, **L**, and **W**, required strength **U** shall not be less than Eq. (9-1).

9.2.3 – If resistance to specified earthquake loads or forces **E** are included in design, load combinations of 9.2.2 shall apply, except that **1.1E** shall be substituted for **W.**

R9.2.3 – If earthquake effects are considered in design, Eq. (9-2) and (9-3) become:

$$U = 1.05D + 1.28L + 1.40E$$

and

$$U = 0.9D + 1.43E$$

The load combinations above consider service-level seismic forces.
Strength-level seismic forces are now specified by many model codes, standards, and similar documents. Strength-level forces should not be used directly in the above combinations.

E.2 Load Combinations of IBC-2000

1605.2 Load Combinations Using Strength Design or Load Resistance Factor Design:

1605.2.1 Basic Load Combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4D$	(1605.2.1.1)
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	(1605.2.1.2)
$1.2D + 1.6(L_r \text{ or } S) + (f_1L \text{ or } 0.8W)$	(1605.2.1.3)
$1.2D + 1.3W + f_1L + 0.5(L_r \text{ or } S \text{ or } R)$	(1605.2.1.4)
$1.2D + 1.0E + (f_1L \text{ or } f_2S)$	(1605.2.1.5)
$0.9D \pm (1.0E \text{ or } 1.3W)$	(1605.2.1.6)

Where:

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load
= 0.5 for other live loads

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure
= 0.2 for other roof configurations

Exceptions:

1. For concrete structures where load combinations do not include seismic force, the factored load combinations of ACI 318 Section 9.2 shall be used.
2. Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.4 Special Seismic Load Combinations. For both allowable stress design and strength design methods, where specifically required by Sections 1613 through 1622 or by Chapters 18 through 23, elements and components shall be designed to resist the forces sue to Formula 1605.4.1 when seismic load is additive to gravity forces and Formula 1605.4.2 when seismic load counteracts gravity forces.

$$1.2D + f_1L + E_m \qquad (1605.4.1)$$

$$0.9D - E_m \qquad (1605.4.2)$$

Where:

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load

= 0.5 for other live loads

E_m = the maximum effect of horizontal and vertical forces as set forth in Section 1617.1.2.

1617.1 Seismic Load Effect E and E_m. Seismic for use in the load combination of Section 1605 shall be determined as follows.

1617.1.1 Seismic Load Effect E. Where the effects of gravity and seismic loads are additive, seismic load E shall be defined by Formula 1617.1.1-1:

$$E = \rho Q_E + 0.2S_{DS}D \qquad (1617.1.1\text{-}1)$$

where:

E = the combined effect of horizontal and vertical earthquake-induced forces,

ρ = a reliability factor based on system redundancy obtained in accordance with Section 1617.2,

Q_E = the effect of horizontal seismic forces,

S_{DS} = the design spectral response acceleration at short periods obtained from Section 1615.1.3, or Section 1615.2.2.5, and

D = the effect of dead load.

Where the effects of gravity counteract the seismic load, seismic load E shall be defined by formula 1617.1.1-2

$$E = \rho Q_E - 0.2S_{DS}D \qquad (1617.1.1\text{-}2)$$

where E, ρ, Q_E, S_{DS}, and D are as defined above.

Design shall use the load combinations prescribed in Section 1605.2 for strength or load and resistance factor design methodologies, or Section 1605.3 for allowable stress design methods.

1617.1.2 Maximum Seismic Load Effect, E_m. The maximum seismic load effect E_m shall be used in the Special Seismic Load Combinations in Section 1605.4.

Where seismic forces and dead loads are additive, Formula 1617.1.2-1 shall be used.

$$E = \Omega_0 Q_E + 0.2S_{DS}D \qquad (1617.1.2\text{-}1)$$

Where seismic forces and dead loads counteract, Formula 1617.1.2-2 shall be used.

$$E = \Omega_0 Q_E - 0.2S_{DS}D \qquad (1617.1.2\text{-}2)$$

where E, Q_E, S_{DS} and D are as defined above and Ω_0 is the system overstrength factor as given in Table 1617.6. The term $\Omega_0 Q_E$ need not exceed the maximum force that can be transferred to the element by the other elements of the lateral force resisting system.

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