Limit State: Limit states are those "states" or "condition" of structure in which (i) it ceases to fulfill its intended function (ii) restricts its intended uses.

Limit state Design: Limit state design is the design philosophy in which the designer recognize the various relevant limits states and thus the design the structure such that the probability of the structure exceeding or reaching the limit states are below acceptable level.

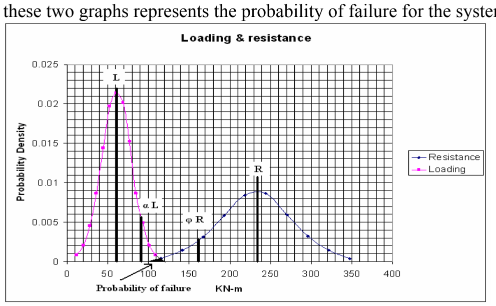
Explain the difference between ultimate limit states and serviceability limit states.

a)"Limit states means those conditions of a building structure that result in the building ceasing to fulfill the function for which it was designed (those limit states
concerning safety are called ultimate limit states (ULS) and include exceeding the load carrying capacity, overturning, sliding and fracture; those limit states that restrict te intended use and occupancy of the building are called serviceability limit states (SLS) and include deflection, vibration, permanent deformation and local structural damage such as cracking.

b. Serviceability limit states may also include aspects of durability.

Explain the probability basis of limit states design of structures, and the significance of load factor and resistance factor.

1) a) i) The probabilistic design concept is based on probability theory and statistical analysis of data. The data analyzed for limit states design is loading and resistance.
Based on this analysis, a graph can be plotted with the probability distributions of the loading and the resistance. The area underneath these two graphs represents the probability of failure for the system.



If the mean value for the loading is used in calculations then the probability of having a loading greater than this value would be 50%. However, if a factor is used which is greater than 1, and that represents the uncertainty in the computed loading then the product of the mean loading and this factor will result in the in an amplified load. The use of this amplified load will ensure that the probability of having a higher load will be less than 50%. The same holds true for the resistance if a resistance factor less than 1 is used. In this case, the product of the mean resistance and the resistance factor creates a factored resistance and the probability of having an actual resistance less than the factored resistance is reduced. It is important to note that the probability of failure can never be reduced to zero however by using the load and resistance factors the probability of having less resistance then expected is reduced while the probability of having a higher load then expected is also reduced to acceptable levels. This gives the basic design
philosophy of:

ϕ: Resistance factor
ϕR > αL R: Resistance
α: Load factor
L: Load

ii) The principle load factor as "principle-load factor means a factor applied to the principle loaf in a load combination to account for the variability of the load and load pattern and the analysis of its effects"1. The companion load factor is similarly defined in section 4.1.3.1.1 g) as "companion-load factor means a factor that, when applied to a companion load in the load combination, gives the probable magnitude of a companion load acting simultaneously with the factored principle load.

Explain the concept of companion-action principle, which is the basis for the derivation of the load combination cases for limit states design of buildings in Canada

The companion-action principle is based on the need to design structures for a variety of potential load effects. These load effects must all be considered at their maximums and the possibility of multiple load effects acting simultaneously must also be considered. However, the probability of multiple maximum load effects acting together must also be taken into account. To this effect the NBCC defines the companion-load factor is "companion-load factor means a factor that, when applied to a companion load in the load combination, gives the probable magnitude of a companion load acting simultaneously with the factored principle load".

In limit states design, the typical load factor applied to specified dead load is 1.25 and to specified live load is 1.5 in factored load combination calculations of ultimate limit states. Explain what is the purpose of load factors in limit states design in general, and why there is a difference in the two load factors for dead load and live load.

a. Due to the long service lives of structures there is uncertainty in that any future load applied to the structure does not exceed the load the structure was designed to withstand safely.

b. The reason the load factor of the dead load is lower than the load factor for the live load is that there is more confidence in the engineer's ability to estimate an accurate dead load for the structure, then to predict an accurate live load for the structure. The self-weight of concrete, steel members, ceiling tiles, flooring, electrical conduits, etc., are all well defined by physical properties such as unit weight and dimensions, which can be calculated with higher level of certainty than other transient load effects. Whereas, the live load, due to the weight of occupants, of would have a large variation Therefore, due to the relative uncertainties of the dead and live loads, the dead load factor is smaller than the live load factor.

The load factor for specified dead load may be taken as 0.9 instead of 1.25, why?

"The counteracting factored dead load, 0.9D in load cases 2, 3, and 4 and 1.0D in load case 5, shall be used when the dead load acts to resist overturning, uplift, sliding, failure due to stress reversal, and to determine anchorage requirements and the factored resistance of members."

	Principal loads	Companion loads
1	D	0
2	D+L	0.5(0.9S) or 0.4(0.75W)
3	D+0.9S	0.5L or 0.4(0.75W)
4	D+0.75 W	0.5L or 0.5(0.9S)

	Principal loads	Companion loads
1	1.4D	0
2	(1.25 or 0.9)D+1.5L	0.5S or 0.4W
3	(1.25 or 0.9)D+1.5S	0.5L or 0.4W
4	(1.25 or 0.9)D+1.4W	0.5L or 0.5S
5	1.0D+1.0E	0.5L+0.25S

In load combination case involving earthquake, the load factors for dead load and earthquake load are both taken as 1.0, why?

For the other load cases the load factors are determined by performing a statistical analysis and then calibrating the load factors to ensure a sufficiently low probability of failure. Due to the fact that a major earthquake is a very rare occurrence, there is not enough data to perform a proper statistical analysis using the same methodology as the other loads, such as snow and wind, to determine the load factors. Therefore in the evaluation of earthquake load it is the uncertainty with regard to the earthquake load intensity is accounted for in the seismic hazard modeling procedure by the seismologists to determine the probability and magnitude of an earthquake and their levels of uncertainty as well. Therefore, in the calculation of load combination case involving the earthquake load, E, a load factor of one if used. And, due to the method described for determining the earthquake load, E, the factor for the dead load cannot be properly calibrated and is therefore left as unity

Explain the definition of return period in the context of limit state design of buildings for snow and wind load.

The return period for wind and snow load expressed in years, is the period of time during which there will be at least one incident that applies the maximum specified wind and snow load on the structure.

Explain why an importance factor of 0.9 is applied to snow load, and an importance factor of 0.75 to wind load, in serviceability limit states check of Limit State Design.

Serviceability limit states are the limit states which restrict the use and occupancy of a building (normal service loads). Ultimate limit states are the limit states which are concerned with the safety of the building (overturning, sliding and fracture). Factors used for ultimate limit states are for major events and therefore consider a large return period of 1 in 50 years for both wind and snow; this must be reduced to a reasonable return period as to check for loads which occur more frequently. Thus, the return period for snow is reduced to 1 in 30 years and the return period for wind is reduced to 1 in 10 years by using importance factors of 0.9 and 0.75, respectively.

Un-factored axial load due to live load only acting on column B3

10

a) Between floor level 1 and 2 (i.e. under level 2)

$$P_{LL,Reduced} = (49 \times 1 + 49 \times 4.8) \times 1 + (5 \times 49 \times 2.4)\left(0.3 + \sqrt{\frac{9.8}{5\times49}}\right) = 578.2\ kN$$

3

$$P_{LL,Reduced} = \mathbf{579\ kN}$$

10

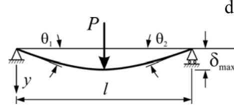
b) Between floor level -1 and 1 (i.e. under level 1)

Required Section Modulus

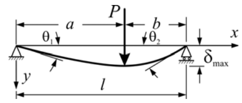
Mmax/S ≤ φfy à S ≥ Mmax/(φfy)

Deflection of beam (to calculate moment of Inertia)

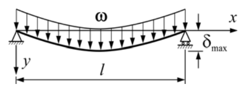
def ≤ L/360



$$def = \frac{PL^3}{48EI}$$



$$def = \frac{Pb}{24EI}(3l^2 - 4b^2)$$



$$def = \frac{Pb}{24EI}(3l^2 - 4b^2)$$

The load combination cases in limit sates design are derived based on the companion action principle. What are the basis and format of the companion action principle? Give one example scenario in your explanation.

− load combinations + companion loads or
− fixed load + (factor greater than 1)*primary variable load + (factor less than one)*companion load
− Statistical reliability analyses or Probability of loads acting in unison
− Clear set of load combinations with a direct physical meaning or Ease of use/simplicity

LLRF

-Type A: Occupancy: LLRF = 0.5 + $\sqrt{\frac{20}{A}}$ A is TA in m^2

Case 1: Assembly area with L.L >= 4.8 KPa
And TA > 80 mm^2

Case 2: area for storage, manufacturing, retail or garage or foot bridge No L.L Condition (i.e. specified LL from table)

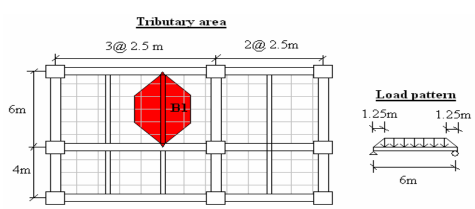
-Type B: Occupancy: LLRF = 0.3 + $\sqrt{\frac{9.8}{B}}$ B is TA in mm^2

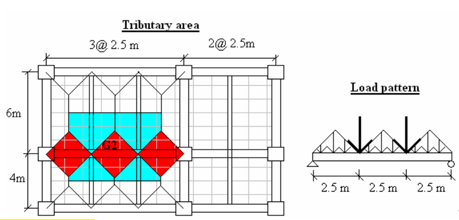
For other occupancy than type A (eg: office) with TA >= 20 m^2

Level	Occupancy	T.A (m^2)	Live Load Intensity (kN/m^2)	Live Load (kN) Other	Live Load (kN) Type B	Live Load (kN) Type A	Cumulative Live Load (kN) Other	Cumulative Live Load (kN) Type B	Cumulative Live Load (kN) Type A	Cumulative Tributary Area (m^2) Other	Cumulative Tributary Area (m^2) Type B	Cumulative Tributary Area (m^2) Type A	LLRF Type B	LLRF Type A	Reduced Live Load (kN) Other	Reduced Live Load (kN) Type B	Reduced Live Load (kN) Type A	Total Reduced Live Load (kN)
8	roof	49	1	49			49	0	0	49	0	0	1	1	49	0	0	49.0
7	office	49	2.4		117.6		49	117.6	0	49	49	0	0.7472	1	49	87.87	0	136.9
6	office	49	2.4		117.6		49	235.2	0	49	98	0	0.6162	1	49	144.94	0	193.9
5	office	49	2.4		117.6		49	352.8	0	49	147	0	0.5582	1	49	196.93	0	245.9
4	office	49	2.4		117.6		49	470.4	0	49	196	0	0.5236	1	49	246.30	0	295.3
3	office	49	2.4		117.6		49	588	0	49	245	0	0.5	1	49	294	0	343.0
2	retail	49	4.8			235.2	49	588	235.2	49	245	49	0.5	1	49	294	235.2	**578.2**
1	retail	49	4.8			235.2	49	588	470.4	49	245	98	0.5	0.9518	49	294	447.71	**790.7**
-1	garage	49	2.4			117.6	49	588	588	49	245	147	0.5	0.8689	49	294	510.89	**853.9**

Revised →

Tributary Areas
Two Way Slab:





SNOW LOAD

1) $S = I_S\left[S_S\left(C_b C_W C_S C_a\right) + S_r\right]$

Where

I_S = importance factor for snow load provided in Table 4.1.6.2

S_S = 1/50 year ground snow load from appendix

C_b = basic roof show load factor in Sentence(2)

C_W = wind exposure factor in Sentences(3)&(4)

C_S = slope factor in Sentences (5), (6) & (7)

C_a = shape factor in Sentence (8)

1) Importance Factor for Snow: (and wind is same except SLS is 0.75)

Importance Category	Importance Factor, I_s	
	ULS	SLS
Low	0.8	0.9
Normal	1.0	0.9
High	1.15	0.9
Post Disaster	1.25	0.9

2) S_r and S_s are given in Table C-2

3) Obtain your C_w

C_w: a building that is located in an exposed setting would be subjected to unfactored wind, which tends to remove more snow off the roof.

For buildings in the low and normal importance categories the wind exposure factor given may be reduced to 0.75 or, in exposed areas north of the tree line= 0.5 where

i) The building is exposed on all sides to wind over open terrain and is expected to remain so during its life,

ii) The area of roof under consideration is exposed to the wind on all sides with no significant obstructions on the roof, such as parapet walls, within a distance of at least 10 times the height of the obstruction and $C_b C_W C_S / \gamma$ metres, and

iii) The loading does not involve the accumulation of snow due to drifting form adjacent surfaces. (All above for open terrain cases)

Otherwise C_w = 1.0 (For rough terrain)

4) Calculate the characteristic length l_c

Where l_C = Characteristic length of the upper or lower roof, defined as $\mathbf{2W} - \frac{w^2}{l}$ (m).

w = smaller plan dimension or the roof in metres

l = larger plan dimension of the roof, in metres

5) Calculate your C_b (You need C_w and l_c)

C_b: The basic roof snow load factor shall be 0.8, except that for large roofs it shall be

i) $1.0-(30/l_C)^2$, for roofs with $C_W = 1.0$ and $l_C \geq 70m$, or

ii) $1.3-(140/l_C)^2$, for roofs with $C_W = 0.75$ or 0.5 and $l_C \geq 200m$

$C_b > 0.8$ for large roofs to account for the observation of wind is less effective in blowing off snow from roof due to the large amount of snow involved.

6) Calculate your C_s (you need the angle to do this)

C_s:

i) 1.0 where the roof slope, $\alpha \leq 30°$

ii) $(70° - \alpha)/40°$ where $30° \leq \alpha \leq 70°$

iii) 0 where $\alpha > 70°$

C_s: The slope factor C_s, for unobstructed slippery roofs where snow and ice can slide completely off the roof shall be (eg: glass/metal):

i) 1.0 where the roof slope, $\alpha \leq 15°$

ii),$(60 - \alpha)/45°$ where $15° \leq \alpha \leq 60°$

iii) 0 where $\alpha > 60°$

C_s: The slope factor C_S shall be 1.0 when used in conjunction with shape factors for increased snow loads

8)Calculate your C_{a0} (depending on your roof)

C_a: The shape factor C_a shall be 1.0 except that where appropriate for the shape of the roof it shall be assigned other values that account for

i) Non-uniform snow loads on gable, arched or curved roofs and domes,

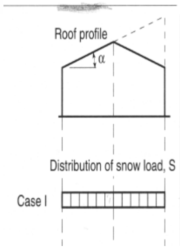
ii) Increased snow loads in valleys,

iii) Increased non-uniform snow loads due to snow drifting onto a roof that is at a level lower than other parts of the same building or at a level lower than another building within 5m of it,

iv) Increased non-uniform snow loads on areas adjacent to roof projections, such as penthouses, large chimneys and equipment, and,

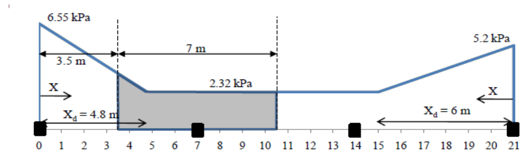
v) Increased snow or ice loads due to snow sliding or melt water draining form adjacent roofs

Figure G-1 Snow distributions and snow loading factors for gable, flat and shed roofs: use CW 0.75 open terrain case 1



Load case	Roof slope α	Factors C_w	C_s	C_a
I	$0° \leq \alpha \leq 90°$	$1.0^{(3)}$	$f(\alpha)^{(1)}$	1.0
	$15° < \alpha < 90°$	1.0	$f(\alpha)^{(1)}$	0.25 + α/20

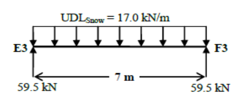
Shaded area in the following figure is acting as a UDL along the length of the beam between E3 and F3 due to snow load.

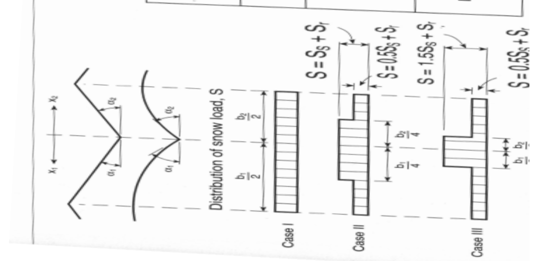


Snow load profile along P1- P2

X = 3.5 m → S = (6.55-0.881X) = 3.467 kPa

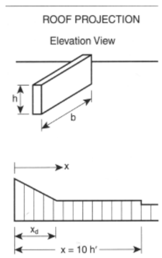
UDL_{Snow} = 2.32 * 7 + (3.467-2.32)*(4.8-3.5)/2 = 17.0 kN/m





Load case	Roof slope α	Factors C_w	C_s	C_a
I	$\alpha \leq 90°$	$1.0^{(2)}$	$f(\alpha)^{(1)}$	1.0
II	$10° < \alpha \leq 90°$	1.0	1.0	For $0 < x \leq b/4$: $C_a = 1/C_0$; For $b/4 < x \leq b/2$: $C_a = 0.5/C_0$
III	$10° < \alpha \leq 90°$	1.0	1.0	For $0 < x \leq b/8$: $C_a = 1.5/C_0$; For $b/8 < x \leq b/2$: $C_a = 0.5/C_0$

Figure G-8 Snow distribution and snow loading factors for areas adjacent to roof obstructions



$$X_d = 2h \text{ but } 3\text{ m} \leq X_d \leq 9\text{ m}$$

$$h' = h - \frac{C_b C_w S_s}{\gamma}$$

$$C_a(0) = 0.67\frac{\gamma h}{C_b S_s}$$

when $C_a(0) < 0.8/C_b$, use $0.8/C_b$

when $C_a(0) > 2/C_b$, use $2/C_b$

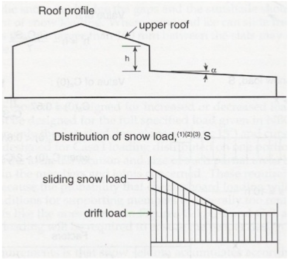
b, metres	x	Factors C_w	C_s	C_a
$\leq 3\ S_s/\gamma^{(2)}$	All	$1.0^{(3)}$	$f(\alpha)^{(1)}$	1.0
$> 3\ S_s/\gamma$	0	1.0	$f(\alpha)^{(1)}$	$C_a(0)$
$> 3\ S_s/\gamma$	$0 < x \leq X_d$	1.0	$f(\alpha)^{(1)}$	$C_a(0) - \frac{(C_a(0)-1)\,x}{X_d}$
$> 3\ S_s/\gamma$	$X_d < x \leq 10\ h'$	1.0	$f(\alpha)^{(1)}$	1.0
	$> 10\ h'$	$1.0^{(3)}$	$f(\alpha)^{(1)}$	1.0

i) Varies as a function of α as defined in 4.1.6.2 (5)&(6)

ii) If $b < 3S_S/\gamma$ then the effect of the obstruction on the snow loading can be ignored

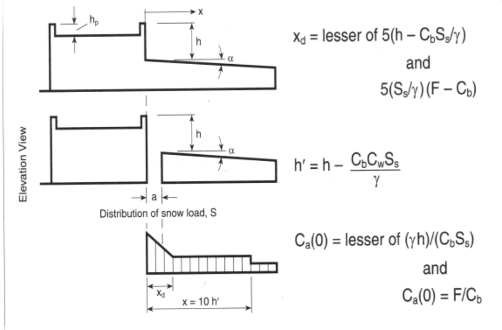
iii) See 4.1.6.2. (4) use for Cw = 0.75

Figure G-7 Snow distribution on lower roofs with a sloped upper roof:



Note: Sliding snow load is equal to 50% of the total weight of the Case I snow load of figure G-1 from the portion of the upper roof that slopes toward the lower roof.

Figure G-5 Snow distributions and snow loading factors for lower levels of adjacent roofs:



x	Factors(2)		
	C_w	C_s	C_a
0	1.0	$f(\alpha)$(1)	$C_a(0)$
$0 < x \le x_d$	1.0	$f(\alpha)$(1)	$C_a(0) - \frac{(C_a(0) - C_a(x_d))\ x}{x_d}$
$x_d < x \le 10\ h'$	1.0	$f(\alpha)$(1)	1.0
$> 10\ h'$	1.0(3)	$f(\alpha)$(1)	1.0

Note: F is the greater of 2 or $0.35\left(\gamma l_c / S_s - 6(\gamma h_p / S_s)^2\right)^{0.5} + C_b$

i) Varies as a function of α as defined in 4.1.6.2 (5)&(6)

ii) If a>5m or $h \le 0.8S_s/\gamma$, drifting need not be considered. Where the upper roof is very large, the limiting gap, a, of 5m should be confirmed by model tests.

iii) See 4.1.6.2.(4)

Importance Categories:

Low: Low human occupancy i.e. Barn, Warehouse

Normal: Everything Else

High: Might be used as a post disaster building i.e. schools

Post Disaster: Essential facilities i.e. Hospitals, fire and police

For live load calculation if building is of low importance live load can be reduced by 0.8**L**.

WIND LOAD

Wind Loading 4.1.7.

11) The specified external pressure or suction due to wind on part or all of a surface of a building shall be calculated using the formula

$p = I_w q C_e C_g C_p$

Where

p = specified equivalent static pressure normal to the surface

I_w = Importance factor for wind load, as provided in table 4.1.7.1.

q= reference velocity pressure, measured at 10m above ground as provided in Sentence (4)

C_e = Exposure factor, as provided in Sentence (5)

C_g = Gust effect factor, as provided in Sentence (6)

C_p = External pressure coefficient

Low rise or high category?

Low rise if H/Ds <1.0 where H = total height and Ds= smaller dimension of the plan view, and h <20m

For High Rise Building

4) The reference velocity pressure, q, shall be the 1/50 year wind pressure (Ottawa q=0.41KPa) for ULS, strength design of structural members. 1/10 years for SLS, design of members for deflection & vibration & cladding.

The exposure factor, C_e, shall be

i) $C_e = (h/10)^{0.2}$, but $C_e \ge 0.9$ For open terrain, where open terrain is level with only scattered buildings, trees or other obstructions, open water or shorelines, h being the reference height above grade in metres for the surface or part of the surface.

ii) $C_e = 0.7(h/12)^{0.3}$, but $C_e \ge 0.7$ For rough terrain, where rough terrain is suburban, urban or wooded terrain extending upwind from the building uninterrupted for at least 1 Km or 10h whichever is greater.

Where h is the reference height,

Windward side h= z , (where z is the height measured from the floor)

Leeward side h=(H/2)

Roof side h=H

The gust effect factor C_g, shall be one of the following values:

i) For the building as a whole and main structural members, $C_g = 2.0$

ii) For external pressures and suctions on small elements including cladding, $C_g = 2.5$

iii) for internal pressures, $C_{gi} = 2.0$ or a value determined by detailed calculation that takes into account the sizes of the openings in the building envelope, the internal volume and the flexibility of the building envelope or,

External pressure coefficient Cp

Account for the effect of wind induced pressure in the building. It is a non-dimensional ratio of wind pressure at a point of a building to the reference velocity pressure at 10m above ground. (depend on the location of the building surface and depends on the structural form)

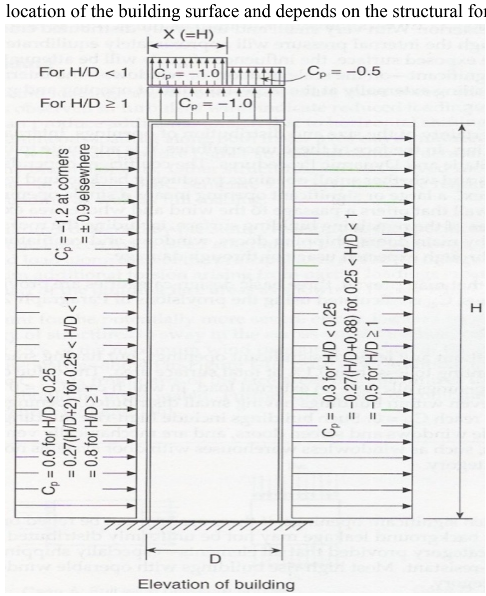
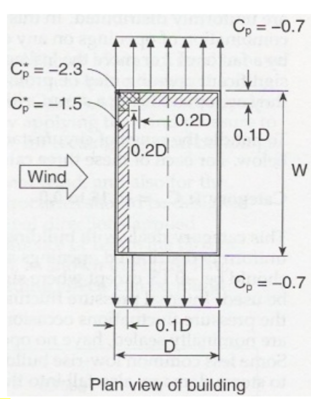


Figure I-15 External pressure coefficients, C_p & C_p^* **for flat-roofed buildings**



For Low Rise

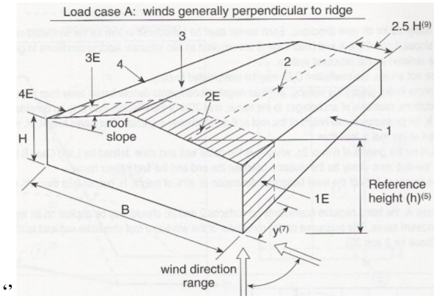
Reference height

If the angle in the gable room < 7 the height is till the eave height and if it is greater than 7 the height is till the midway of gable room and it cannot be less than 6m.

External pressure

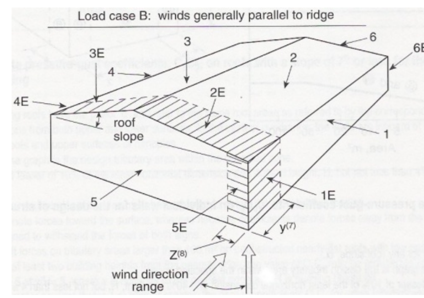
Ce: same 3 cases as high rise with the reference height of the low

Figure I-7 External peak composite pressure-gust coefficients, C_pC_g **for primary structural actions arising from wind load acting simultaneously on all surfaces**



Perpendicular

Roof slope	Building Surfaces							
	1	1E	2	2E	3	3E	4	4E
0º to 5º	0.75	1.15	-1.3	-2.0	-0.7	-1	-0.55	-0.8
20º	1.0	1.5	-1.3	-2.0	-0.9	-1.3	-0.8	-1.2
30º to 45º	1.05	1.3	0.4	0.5	-0.8	-1	-0.7	-0.9
90º	1.05	1.3	1.05	1.3	-0.7	-0.9	-0.7	-0.9



Parallel

Roof slope	Building Surfaces					
	1	1E	2	2E	3	3E
0º to 90º	-0.85	-0.9	-1.3	-2.0	-0.7	-1.0
	4	4E	5	5E	6	6E
0º to 90º	-0.85	-0.9	0.75	1.15	-0.55	-0.8

7) End-zone width **y** should be the greater of 6m or 2z where z is the gable wall end zone defined for load case (b) below. Alternatively, *for buildings with frames, the end zone y may be the distance between the end and the first interior frame.*

8) End-zone width z is the lesser of 10% of the least horizontal dimension or 40% of the height H but not less than 4% of the least horizontal dimension or 1m.

Internal Pressure

$$p_i = I_w q C_e C_{gi} C_{pi}$$

where

p_i = The specified internal pressure acting statically and in a direction normal to the surface.

I_w = importance factor for wind load, as provided in table 4.1.7.1.

q= reference velocity pressure, as provided in Sentence (4)

C_e = exposure factor, as provided in Sentence (5)

C_{gi} = internal gust effect factor, as provided in Sentence (6)

C_{pi} = internal pressure coefficient

Cpi

Category 1: $C_{pi} = -0.15$ to 0.0

For building with no large opening example high rise building, mechanical ventilation. (0: small windows -0.15: fully closed)

Category 2: Cpi = -0.43 to 0.3

Building with significant openings but expected to be closed during stroms eg. Low rise building, high rise building with balcony doors, industrial building with wind resistant door (take whichever gives you the critical case)

Category 3: $C_{pi} = -0.7$ to 0.7

Building with large opening such as shipping doors may be open during strom or non-wind resistant type. (take both)

Cgi =2.0.

Earthquake load

Seismic design objective structures should be design to withstand minor EQ that can be expected to occur several times over the service life of structure without any damage.(2) to withstand moderate earthquake expected to occur over the service life of the structure. (3) structure are designed to withstand major rare large EQ without collapse (I,e, some structure remain intact after major EQ although it may suffer extensive damage that it may have to be demolished).

Life safety objective: so that occupant can safely egress building after EQ

Dynamic effect of structures: vibration responses of structures when subjected to dynamic loading are influenced by number of effect.

(1) Intertia effect F=ma, depends on weight of the building and how it is distributed in the building. Lighter building attract smaller EQ loads. Building with weight evenly distributed over the height will perform better in EQ

(2) Frequency of applied dynamic loading Fundamental mode+ higher mode effect factor. Dynamic structural analysis: given the input motion (harmonic), can find the vibration response of the building

(3) Resonance effect: Vibration response effect of structure can become very large when the frequency of the applied dynamic loading matches the natural frequency of the structure.

Seismic Design of bld is influenced by

(1) Seismic hazard at the building location
(2) Local site condition
(3) Building structure systems (SFRS)

Seismic design approaches

(1) Dynamic analysis procedure
(2) Equivalent static load design procedure.

Base shear formula (V)

$V = S(T_a)M_vI_EW / (R_dR_o)$

$S(T_a)$=Design spectral response acceleration (expressed in unit of g)

W=Seismic weight of the building (w= dead load + 25% of the specified snow load + 60% of load of the storage area.

Mv= Higher mode effect factor

Ie=Importance factor for EQ

Rd = ductility factor

Ro= Over strength factor.

(1) Ie Imp. factor

Bld cater	ULS
Low	0.8
Normal	1.0
High	1.3
Post disaster	1.5

Constraints for V

Lower Limit	V not less than $S(4.0)M_vI_EW / (R_dR_o)$	For shear walls coupled walls wall frame system
OR	V not less than $S(2.0)M_vI_EW / (R_dR_o)$	For moment-resisting frames, braced frames and other system
Upper Limit	V not greater than $2S(0.2)I_EW / 3(R_dR_o)$	For bldg. with RD≥1.5 & not site class F

To calculate V (base shear)

1) **Calculate the total weight, W**
2) **Get Ie from imp factor table**
3) **For Ta**

	Ta
Moment Resisting Frame	$0.085(h_n)^{3/4}$ for steel $0.075(h_n)^{3/4}$ for concrete
Braced Frame	$0.025h_n$
Shear Wall	$0.05(h_n)^{3/4}$
Other frame	0.1N

Where h_n is the total height including basement and N is number of floor without basements.

4) **Get the soil class by determining one of these equations down.**

N_{60}, *Commentary J (102)*

Use: N_{60}=total thickness/(Σlayer thickness/layer N_{60});

S_u, Commentary J (102)

Use: S_u=total thickness/(Σlayer thickness/layer S_u); or

V_s, Commentary J (101)

Use: V_s=total thickness/(Σlayer thickness/layer S_u)

Note: According to NBCC 2010 consider only the first 30m of the total layer of thickness.

Table 4.1.8.4.

Site Class	Average Properties in Top 30m: Avg Shear Wave Velocity (V_s)	Avg Stdrd Penetration Resistance (N_{60})	Soil Undrained Shear Strength (S_u)
A	V_s>1500	n/a	n/a
B	760<V_s≤1500	n/a	n/a
C	360<V_s<760	N_{60}>50	S_u>100kPa
D	180<V_s<360	15≤N_{60}≤50	50<S_u≤100
E	V_s<180	N_{60}<15	S_u≤50
F	Other Soil, site-specific data needed		

5) Once the Site Class is determined, determine the site class

Sa(0.2)	0.64
Sa(0.5)	0.31
Sa(1.0)	0.14
Sa(2.0)	0.046

Note: The value of Sa is for Ottawa.

6)Interpolate using Sa(0.2) and Sa(1.0) from table above to find the required Fv and Fa

Table4.1.8.4.B

Site Class	Values of F_a *Next line is values of Sa(0.2) given above*				
	≤0.25	=0.5	=0.75	=1.0	≥1.25
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	Need Site Specific Data *(4.1.8.4(5))*				

Table 4.1.8.4.C

Site Class	Values of F_v *Next line is values of Sa(1.0) given above*				
	≤0.1	=0.2	=0.3	=0.4	≥0.5
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	Need Site Specific Data *(4.1.8.4(5))*				

Linear Interpolation may be needed with the tables

7) **Spectral acceleration at different period using the table below.**

S(T)	FaSa(0.2)	T≤ 0.2
	Lesser of FaSa(0.2) FvSa(0.5)	T=0.5
	FvSa(1.0)	T=1.0
	FvSa(2.0)	T=2.0
	FvSa(2.0)/2	T=4.0

8) **Graph the S(T) Vs T above and do linear interp. To get required S(T)**

9)Find Mv and J

$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting System	M_v for T_a: ≤1.0	≥2.0	≥4.0	J for T_a: ≤0.5	≥2.0	≥4.0
< 8.0	Moment Resisting Frame	1.0	1.0	1.0	1.0	1.0	0.9
	Braced Frame	1.0	1.0	1.0	1.0	0.8	0.8
	Walls	1.0	1.2	1.6	1.0	0.6	0.5
≥8.0	Moment Resisting Frame	1.0	1.2	1.2	1.0	0.7	0.7
	Braced Frame	1.0	1.5	1.5	1.0	0.6	0.6
	Walls	1.0	2.2	3.0	1.0	0.4	0.3

Note: Mv between Ta(1) and Ta(2) and between Ta(2) and (Ta4) the product S(Ta)Mv Shall be obtained by linear interpolation. And For J follow same same mal awal.

10) get V and the constraints of V.

To calculate Of Fx on each floor, Lateral seismic force.

$F_x=(V-F_t)W_xh_x/(\Sigma^n_{i=1} W_xh_x)$

1)to calculate F_t

F_t	0	T_a≤0.7
	$0.07T_aV$	0.7<T_a<3.6
	0.25V	T_a≥3.6

2) get V-F_t

3) get Hx at each floor

4) get Wx at each floor

5)Hx*Wx for each floor and get the sum of it.

And follow equation above to get Fx. Fx should be equal to V. And add Ft to the top floor.

6)Calculate storey shear, V_x. Do cummulitive sum of F_x starting from roof(add F_t !!)

Solve Moment on each floor

1)

J_x	1.0	h_x≥$0.6h_n$
	$J+(1-J)(h_x/0.6h_n)$	h_x<$0.6h_n$

Note: Get J by interp the table for j and M_v

2)use equation below to find moments, multiply the shear(V_x) of each floor by the floor spacing(h_i-h_x), then do cumulative $\Sigma_{ni=1}F_i(h_i-h_x)$ of each floor, then multiply by moment reduction factor(J_x)

$M_x = J_x \Sigma_{ni=1}F_i(h_i-h_x)$;