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4.1.2.1 (2015 NBC)

Use and Occupancy Importance Category	Importance Category
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including: • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings	Low(1)
All buildings except those listed in Importance Categories Low, High and Post-disaster	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is: • as an elementary, middle or secondary school • as a community centre Manufacturing and storage facilities containing toxic, explosive or other hazardous substances in sufficient quantities to be dangerous to the public if released(1)	High
Post-disaster buildings are buildings that are essential to the provision of services in the event of a disaster, and include: • hospitals, emergency treatment facilities and blood banks • telephone exchanges • power generating stations and electrical substations • control centres for air, land and marine transportation • public water treatment and storage facilities, and pumping stations	Post-disaster

4.1.2.1 (2020 NBC)

Type of Building	Importance Category
A Low Importance Category building is a building that represents a low direct or indirect hazard to human life in the event of structural failure.	Low
A Normal Importance Category building is a building that does not meet the criteria for a Low Importance Category building, High Importance Category building or post-disaster building.	Normal
A High Importance Category building is a building that provides a greater degree of safety to human life than a Normal Importance Category building. Community centres and elementary, middle and secondary schools are High Importance Category buildings.	High
A post-disaster building.	Post-disaster

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	• sewage treatment facilities and buildings having critical national defence functions • buildings of the following types, unless exempted from this designation by the authority having jurisdiction:(2) • emergency response facilities • fire, rescue and police stations, and housing for vehicles, aircraft or boats used for such purposes • communications facilities, including radio and television stations		
4.1.2.2	**Loads Not Listed** **1)** Where a building or structural member can be expected to be subjected to loads, forces or other effects not listed in Article 4.1.2.1., such effects shall be taken into account in the design based on the most appropriate information available.	4.1.2.2	**Loads Not Listed** **1)** Where a building or structural member can be expected to be subjected to loads, forces or other effects not listed in Article 4.1.2.1., such effects shall be taken into account in the design based on the most appropriate information available. (See Note A-4.1.2.2.(1).)
4.1.3.4	**Serviceability** **1)** A building and its structural components shall be checked for serviceability limit states as defined in Clause 4.1.3.1.(1)(a) under the effect of service loads for serviceability criteria specified or recommended in Articles 4.1.3.5. and 4.1.3.6. and in the standards listed in Section 4.3. (See Note A-4.1.3.4.(1).)	4.1.3.4	**Serviceability** **1)** A building and its structural components shall be checked for serviceability limit states as defined in Clause 4.1.3.1.(1)(a) under the effect of service loads for serviceability criteria specified or recommended in Articles 4.1.3.5. and 4.1.3.6. and in the standards listed in Section 4.3. (See Note A-4.1.3.4.(1).) **2)** The effect of service loads on the serviceability limit states shall be determined in accordance with this Article and the load combinations listed in Table 4.1.3.4., the applicable combination being that which results in the most critical effect. **3)** Other load combinations that must also be considered are the principal loads acting with the companion loads taken as zero. **4)** Deflections calculated for load types P, T and H, if present, with load factors of 1.0 shall be included with the calculated deflections due to principal loads. **5)** The determination of the deflection shall consider the following: a) for materials that result in increased deformations over time under sustained loads, the deflection calculation shall consider the portion of live load, L, that is sustained over time, Ls, and the portion that is transitory, Lt, and b) the calculated deflection due to dead load, D, and sustained live load, Ls, shall be increased by a creep factor as specified in the standards listed in Section 4.3. to obtain the additional long-term deflection. 6) The determination of the long-term settlement of foundations shall consider the following: a) for foundation soil types that result in increased settlement over time under sustained loads,

<table>
<tr><th>2015 NBC</th><th></th><th>2020 NBC</th><th><u>CHANGES MADE</u></th></tr>
<tr><td>Code Ref.</td><td>Part 4</td><td>Code Ref.</td><td>Part 4</td></tr>
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the additional long-term settlements shall be determined for the portion of live load, L, that is sustained over time, Ls, and the portion that is transitory, Lt, and b) the additional long-term settlements due to dead load, D, and sustained live loads, Ls, shall be calculated from the foundation soil properties provided by a qualified professional geotechnical engineer.

Table 4.1.3.4. Loads and Load Combinations for Serviceability Forming Part of Sentence 4.1.3.4.(2)
<table>
<tr><th>Limit State</th><th>Structural Parameter</th><th>Load Case</th><th>Load Combinations
Principle Loads</th><th>Companian Loads</th></tr>
<tr><td>Deflection for materials not subject to creep</td><td>Deflection of the structure or of components of the structure(1)</td><td>1
2
3</td><td>1.0D + 1.0L
1.0D + 1.0 W
1.0D + 1.0S</td><td>0.3W or 0.35S
0.35L or 0.35S
0.3W or 0.35L</td></tr>
<tr><td>Deflection for materials subject to creep</td><td>Total deflection of the structure or of components of the structure(3)</td><td>1
2
3</td><td>1.0D + 1.0Ls (4) + 1.0Lt (5)
1.0D + 1.0W
1.0D + 1.0S</td><td>0.3W or 0.35S
0.35L(2) or 0.35S
0.3W or 0.35L(2)</td></tr>
<tr><td>Vibration serviceability</td><td>Acceleration</td><td colspan="3">(6)</td></tr>
</table>
Notes to Table 4.1.3.4:
(1) The calculated deflection due to dead load, D, is permitted to be excluded where specified in the standards listed in Section 4.3.
(2) The companion load factor of 0.35 for live load, L, shall be increased to 0.5 for storage areas, equipment areas and service rooms.
(3) The calculated immediate deflection due to dead load, D, is permitted to be excluded where specified in the standards listed in Section 4.3.
(4) Ls = sustained portion of the live load, L.
(5) Lt = transitory portion of the live load, L.
(6) See Note A-Table 4.1.3.4.
</td></tr>
<tr><td>4.1.3.6</td><td>Vibration
2) Where the fundamental vibration frequency of a structural system supporting an assembly occupancy used for rhythmic activities, such as dancing, concerts, jumping exercises or gymnastics, is less than 6 Hz, the effects of resonance shall be investigated</td><td>4.1.3.6</td><td>Vibration
2) Where floor vibrations caused by resonance with operating machinery or equipment are anticipated, dynamic analysis of the floor system shall be carried out.
(See Note A-4.1.3.6.(2).)</td></tr>
</table>

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	by means of a dynamic analysis. (See Note A-4.1.3.6.(2).)		
4.1.4.1	**Dead Loads** 1) The specified dead load for a structural member consists of a) the weight of the member itself, b) the weight of all materials of construction incorporated into the building to be supported permanently by the member, c) the weight of partitions, d) the weight of permanent equipment, and e) the vertical load due to earth, plants and trees. 2) Except as provided in Sentence (5), in areas of a building where partitions, other than permanent partitions, are shown on the drawings, or where partitions might be added in the future, allowance shall be made for the weight of such partitions. 3) The partition weight allowance referred to in Sentence (2) shall be determined from the actual or anticipated weight of the partitions placed in any probable position, but shall be not less than 1 kPa over the area of floor being considered. 4) Partition loads used in design shall be shown on the drawings as provided in Clause 2.2.4.3.(1)(d) of Division C. 5) In cases where the dead load of the partition is counteractive, the load allowances referred to in Sentences (2) and (3) shall not be included in the design calculations. 6) Except for structures where the dead load of soil is part of the load-resisting system, where the dead load due to soil, superimposed earth, plants and trees is counteractive, it shall not be included in the design calculations. (See Note A-4.1.4.1.(6).)	**4.1.4.1**	**Dead Loads** 1) The specified dead load for a structural member consists of a) the weight of the member itself, b) the weight of all materials of construction incorporated into the building to be supported permanently by the member, c) the weight of partitions, d) the weight of permanent equipment, and e) the vertical load due to soil, superimposed earth, plants and trees. 2) In areas of a building for which partitions are shown on the drawings, the weight of partitions referred to in Clause (1)(c) shall be taken as the actual weight of such partitions. (See Note A-4.1.4.1.(2).) 3) In areas of a building for which partitions are not shown on the drawings, the weight of partitions referred to in Clause (1)(c) shall be a partition weight allowance determined from the anticipated weight and position of the partitions, but shall not be less than 1 kPa over the area of floor being considered. (See Note A-4.1.4.1.(3).) 4) The weights of partitions and partition weight allowances used in the design shall be shown on the drawings as provided in Clause 2.2.4.3.(1)(d) of Division C. 5) Where the *partition* weight allowance referred to in Sentence (3) is counteractive to other loads, it shall not be included in the design calculations. 6) Except for structures where the *dead load* of *soil* is part of the load-resisting system, where the *dead load* due to *soil*, superimposed earth, plants and trees is counteractiveto other loads, it shall not be included in the design calculations. (See Note A-4.1.4.1.(6).)
4.1.5.3	Office areas (not including record storage and computer rooms) located in Basement and the first storey — **4.8** Floors above the first storey — **2.4**	**4.1.5.3**	Office areas(1) (not including record storage and computer rooms) located in Basements, and floors, including mezzanines, with direct access to the exterior at ground level — 4.8 Other floors — 2.4
4.1.5.5	4) Roof parking decks shall be designed for either the uniformly distributed *live loads* specified in Table 4.1.5.3., the concentrated *live loads* listed in Table 4.1.5.9., or the roof snow load, whichever produces the most critical effect in the members concerned.	**4.1.5.5**	4) Roof parking decks and exterior areas accessible to vehicular traffic shall be designed a) for the appropriate load combination listed in Sentence 4.1.3.2.(2) with a *live load*, L, consisting of either a uniformly distributed *live load* as specified in Table 4.1.5.3. or a concentrated *live load* as listed in Table 4.1.5.9., whichever produces the most critical effect, and a companion snow load, S, as

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			prescribed in Subsection 4.1.6., but with the companion-load factor reduced to 0.2, and b) such that the load combination in Clause (a) is not less than the snow and rain loads prescribed in Subsection 4.1.6. with the *live load* taken as zero. 5) Roof parking decks that are used for the long-term storage of vehicles shall be designed for the appropriate load combination listed in Sentence 4.1.3.2.(2) with a *live load*, L, consisting of either a uniformly distributed *live load* as specified in Table 4.1.5.3. or a concentrated *live load* as listed in Table 4.1.5.9., whichever produces the most critical effect, and a snow load, S, as prescribed in Subsection 4.1.6.
4.1.5.8	1) An area used for *assembly occupancies* designed for a *live load* of less than 4.8 kPa and roofs designed for the minimum loading specified in Table 4.1.5.3. shall have no reduction for tributary area. New in 2020	**4.1.5.8**	1) One- and two-way floor slabs shall have no reduction for tributary area applied to *live load*. 5) Where the specified *live load* for a floor is reduced in accordance with Sentence (3) or (4), the structural drawings shall indicate that a *live load* reduction factor for tributary area has been applied and which structural elements are impacted by this factor.
4.1.5.14	1) The minimum specified horizontal load applied outward at the minimum required height of every required *guard* shall be a) 3.0 kN/m for open viewing stands without fixed seats and for *means of egress* in grandstands, stadia, bleachers and arenas, b) a concentrated load of 1.0 kN applied at any point, so as to produce the most critical effect, for access ways to equipment platforms, contiguous stairs and similar areas where the gathering of many people is improbable, and c) 0.75 kN/m or a concentrated load of 1.0 kN applied at any point so as to produce the most critical effect, whichever governs for locations other than those described in Clauses (a) and (b). 2) The minimum specified horizontal load applied inward at the minimum required height of every required *guard* shall be half that specified in Sentence (1). 3) Individual elements within the *guard*, including solid panels and pickets, shall be designed for a load of 0.5 kN applied outward over an area of 100 mm by 100 mm located at any point in the element or elements so as to produce the most critical effect. 4) The size of the opening between any two adjacent vertical elements within a *guard* shall not exceed the limits required by Part 3 when each of these elements is subjected to a specified *live load* of 0.1 kN applied in opposite directions in the in-plane	**4.1.5.14**	1) The minimum horizontal specified *live load* applied outward at the minimum required height of every required *guard* shall be a) 3.0 kN/m for open viewing stands without fixed seats and for *means of egress* in grandstands, stadia, bleachers and arenas, b) 1.0 kN applied at any point, so as to produce the most critical effect, for access ways to equipment platforms, contiguous stairs and similar areas where the gathering of many people is improbable, and c) 0.75 kN/m or 1.0 kN applied at any point so as to produce the most critical effect, whichever governs, for locations other than those described in Clauses (a) and (b). 2) The minimum horizontal specified *live load* applied inward at the minimum required height of every required *guard* shall be half that specified in Sentence (1). 3) Individual elements within the *guard*, including solid panels and pickets, shall be designed for a horizontal specified *live load* of 0.5 kN applied outward over an area of 100 mm by 100 mm located at any point on the element or elements so as to produce the most critical effect. 4) The size of the opening between any two adjacent vertical elements within a *guard* shall not exceed the limits required by Part 3 when each of these elements is

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	direction of the *guard* so as to produce the most critical effect. 5) The loads required in Sentence (3) need not be considered to act simultaneously with the loads provided for in Sentences (1), (2) and (6). 6) The minimum specified load applied vertically at the top of every required *guard* shall be 1.5 kN/m and need not be considered to act simultaneously with the horizontal load provided for in Sentence (1). 7) Handrails and their supports shall be designed and constructed to withstand the following loads, which need not be considered to act simultaneously: a) a concentrated load not less than 0.9 kN applied at any point and in any direction for all handrails, and b) a uniform load not less than 0.7 kN/m applied in any direction to handrails not located within *dwelling units*.		subjected to a horizontal specified *live load* of 0.1 kN applied in opposite directions in the in-plane direction of the *guard* so as to produce the most critical effect. 5) The specified *live loads* required in Sentence (3) need not be considered to act simultaneously with the loads provided for in Sentences (1), (2), (6) and (7). 6) The minimum specified *live load* applied vertically at the top of every required *guard* shall be 1.5 kN/m and need not be considered to act simultaneously with the horizontal specified *live load* provided for in Sentences (1), (3) and (7). 7) Handrails and their supports shall be designed and constructed to withstand the following minimum specified *live loads*, which need not be considered to act simultaneously: a) 0.9 kN applied at any point and in any direction for all handrails, and b) 0.7 kN/m applied in any direction for handrails not located within *dwelling units*.
4.1.6.1	Specified Load Due to Rain or to Snow and Associated Rain 1) The specified load on a roof or any other *building* surface subject to snow and associated rain shall be the snow load specified in Article 4.1.6.2., or the rain load specified in Article 4.1.6.4., whichever produces the more critical effect.	**4.1.6.1**	Specified Load Due to Rain or to Snow and Associated Rain 1) The specified load on a roof or any other *building* surface subject to snow and associated rain shall be the snow load specified in Article 4.1.6.2., or the rain load specified in Article 4.1.6.4., whichever produces the more critical effect. (See Note A-4.1.6.1.(1).)
4.1.6.2	where l_c = characteristic length of the upper or lower roof, defined as 2w−w_2/l, in m, w = smaller plan dimension of the roof, in m, and l = larger plan dimension of the roof, in m, or b) conform to Table 4.1.6.2.-B, using linear interpolation for intermediate values of . (See Note A-4.1.6.2.(2).)	**4.1.6.2**	where l_c = characteristic length of the upper or lower roof, defined as 2w − w_2/l, in m, w = smaller plan dimension of the roof, in m, and l = larger plan dimension of the roof, in m, b) conform to Table 4.1.6.2.-B, using linear interpolation for intermediate values of , or c) be taken as equal to 1 for any roof structure with a mean height of less than 1 + S_s/γ, inm, above *grade*, where γ is the specific weight of snow determined in accordance with Article 4.1.6.13. (See Note A-4.1.6.2.(2).)
4.1.6.4	4) Where scuppers are provided and where the position, shape and deflection of the loaded surface make an accumulation of rainwater possible, the loads due to rain shall be the lesser of either the one-day rainfall determined in conformance with Subsection 1.1.3. or a depth of rainwater equal to 30 mm above the level of the scuppers, applied over the horizontal projection of the surface and tributary areas	**4.1.6.4**	4) Where scuppers are provided as secondary drainage systems and where the position, shape and deflection of the loaded surface make an accumulation of rainwater possible, the loads due to rain shall be the lesser of either the one-day rainfall determined in conformance with Subsection 1.1.3. or a depth of rainwater equal to 30 mm above the bottom of the scuppers, applied over the horizontal projection of

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			the surface and tributary areas.
4.1.6.5	where C_{a0} = peak value of C_a at $x = 0$ determined in accordance with Sentences (3) and (4) and as shown in Figure 4.1.6.5.-B, x = distance from roof step as shown in Figure 4.1.6.5.-A, and x_d = length of drift determined in accordance with Sentence (2) and as shown in Figure 4.1.6.5.-A. Notes to Figure 4.1.6.5.-A: (1) If $a > 5$ m or $h \leq 0.8S_s$/©, drifting from the higher roof need not be considered. (2) For lower roofs with parapets, $C_s = 1.0$, otherwise it varies as a function of slope 〈 as defined in Sentences 4.1.6.2.(5) and (6). 3) The value of C_{a0} for each of Cases I, II, and III shall be the lesser of where h_p = height of the roof perimeter parapet of the source area, to be taken as zero unless all the roof edges of the source area have parapets 4) The value of C_{a0} shall be the highest of Cases I, II and III, considering the different roof source areas for drifting snow, as specified in Sentence (3) and Figure 4.1.6.5.-B. New in 2020	**4.1.6.5**	where C_{a0} = peak value of C_a at $x = 0$ determined in accordance with Sentences (3) to (5) and as shown in Figure 4.1.6.5.-B, x = distance from roof step as shown in Figure 4.1.6.5.-A, and x_d = length of drift determined in accordance with Sentence (2) and as shown in Figure 4.1.6.5.-A. Notes to Figure 4.1.6.5.-A: (1) If $a > 5$ m or $h \leq 0.8S_s$/©, drifting from the higher roof need not be considered. (2) If $h \geq 5$ m, the value of C_{a0} for Case I is permitted to be determined in accordance with Sentence 4.1.6.5.(4). 3) Except as provided in Sentence (4), the value of C_{a0} for each of Cases I, II and III shall be the lesser of where h_p = height of the roof perimeter parapet of the source area, to be taken as zero unless all the roof edges of the source area have parapets. 4) Where $h \geq 5$ m, the value of C_{a0} for Case I is permitted to be taken as $C_{a0} = \left(\frac{25-h}{20}\right)\left(\frac{F}{C_b}-1\right)+1$ for $5\text{ m} \leq h \leq 25\text{ m}$, and $C_{a0} = 1$ for $h > 25$ m 5) The value of C_{a0} shall be the highest of Cases I, II and III, considering the different roof source areas for drifting snow, as specified in Sentences (3) and (4) and Figure 4.1.6.5.-B.
4.1.6.8	2) The drift loads on the lower level roof against the two faces of an inside corner of an upper level roof or a parapet shall be calculated for each face and applied as far as the bisector of the corner angle as shown in Figure 4.1.6.8.-B.	**4.1.6.8**	2) The drift loads on the lower level roof against the two faces of an inside corner of an upper level roof or a parapet shall be calculated for each face and the higher of the two loads shall be applied where the drifts overlap as shown in Figure 4.1.6.8.-B

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	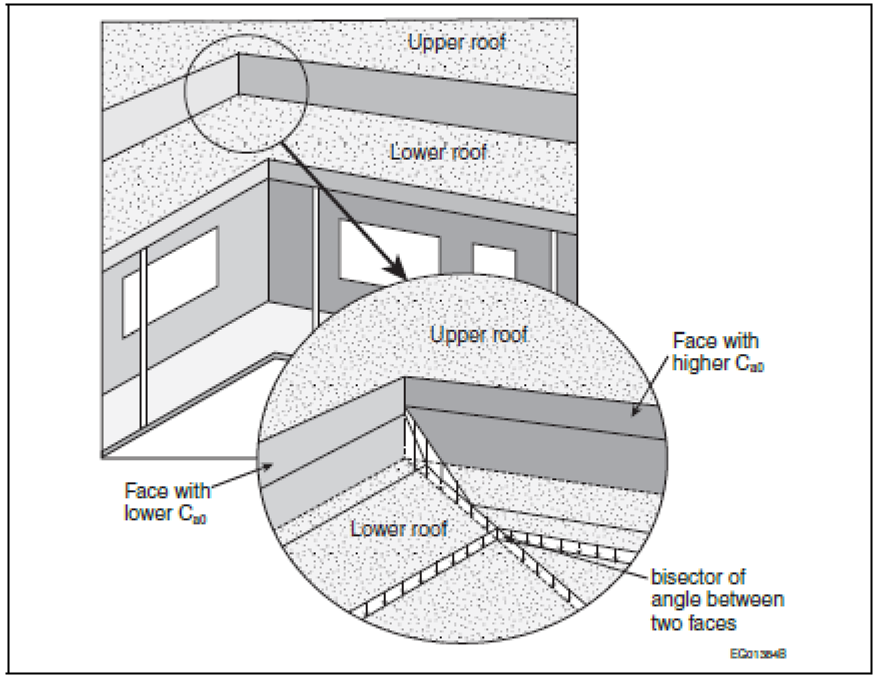		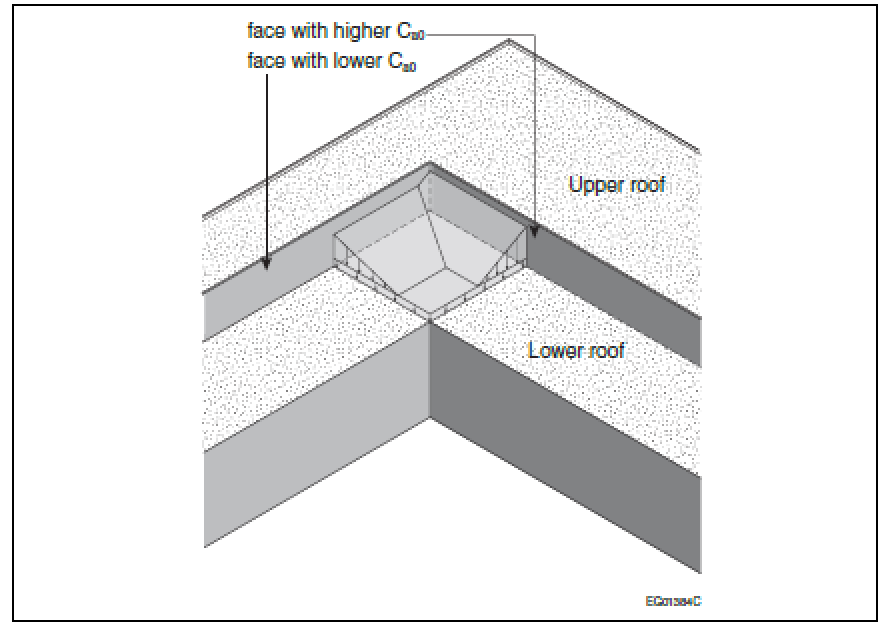
4.1.6.16	New in 2020	**4.1.6.16**	Roofs with Solar Panels (See Note A-4.1.6.16.) 1) Where solar panels are installed on a roof, the snow loads, S, shall be determined in accordance with Sentences (2) to (6) or with the requirements for roofs without solar panels, whichever produces the most critical effect. 2) For the purposes of this Article, solar panels shall be classified as a) Parallel Flush, where the panels are installed parallel to the roof surface with their upper surface less than or equal to $C_bC_wS_s/\gamma$ above the roof surface, b) Parallel Raised, where the panels are installed parallel to the roof surface with their upper surface greater than $C_bC_wS_s/\gamma$ above the roof surface, or c) Tilted, where the panels are installed at an angle to the roof surface with their highest edge greater than $C_bC_wS_s/\gamma$ above the roof surface. 3) For sloped roofs with solar panels, the snow loads, S, shall be determined in accordance with the requirements for roofs without solar panels, except that the slope factor, C_s, shall be

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			a) taken as 1.0 for roof areas extending upslope from the downslope edge of a panel or array of panels at an angle of 45° from each side edge of the panel or array, and b) as specified in Sentences 4.1.6.2.(5) to (7) for all other roof areas. (See Note A-4.1.6.16.(3).) 4) For sloped roofs with Parallel Flush solar panels, the snow loads, S, shall be determined in accordance with the requirements for roofs without solar panels, except that a) C_s shall be determined in accordance with Sentence (3), b) where the gap width, w_g, between the panels along the roof slope is greater than or equal to the panel width, w_p, along the roof slope, the accumulation factor, C_a, shall be taken as i) 0.0 for the panels, ii) 2.0 for roof areas within a distance of w_p downslope from a downslope panel edge, and iii) 1.0 for all other roof areas (see Note A-4.1.6.16.(4)(b)), and c) where the gap width, w_g, between the panels along the roof slope is less than the panel width, w_p, along the roof slope, C_a shall be taken as i) 0.0 for panel areas within a distance of w_g downslope from an upslope panel edge, ii) 1.0 for other panel areas, iii) 2.0 for roof areas in gaps between the panels, and iv) 1.0 for all other roof areas (see Note A-4.1.6.16.(4)(c)). 5) For roofs with Parallel Raised solar panels, the snow loads, S, shall be determined in accordance with the requirements for roofs without solar panels, except that a) where the roof is flat, C_a shall be taken as i) 1.0 for the panels, ii) 1.0 for roof areas not under the panels, iii) 1.0 for roof areas under the panels within a distance of $\min(2h_g, 2w_g)$ from a panel edge, where h_g is the gap height between the lower surface of the panels and the roof surface, and w_g is the gapwidth between the panels, and

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			iv) 0.0 for other roof areas under the panels (see Note A-4.1.6.16.(5)(a)), and b) where the roof is sloped, the snow loads, S, derived from Clause (a) shall be used, except that i) C_s shall be determined in accordance with Sentence (3), ii) S shall be taken as 0.0 on the panels, and iii) S for all roof areas shall be taken as the sum of S on the panels, as derived from Subclause (a)(i) and shifted by a distance of w_p downslope onto the roof, where w_p is the panel width along the roof slope, and S on the roof areas, as derived from Subclauses (a)(ii) to (a)(iv) (see Note A-4.1.6.16.(5)(b)). 6) For flat roofs with Tilted solar panels, the snow loads, S, shall be determined in accordance with the requirements for roofs without solar panels, except that a) C_a shall be taken as 0.0 for the panels, b) C_a shall be taken as 1.0 for roof areas beyond a distance of $5(h – C_bC_wS_s/\gamma)$ from the lowest edge of the panels, where h is the height of the highest edge of the panels above the roof surface, c) except as provided in Clauses (d) and (e), for roof areas within a distance of $5(h – C_bC_wS_s/\gamma)$ from the lowest edge of the panels, C_a shall be taken as i) 1.25 for $(h_g – C_bC_wS_s/\gamma) \leq 0.3$ m, where h_g is the gap height between the lowest edge of the panels and the roof surface, ii) $1.294 – 0.1471(h_g – C_bC_wS_s/\gamma)$ for$0.3 < (h_g – C_bC_wS_s/\gamma) \leq 2.0$ m, and iii) 1.0 for $(h_g – C_bC_wS_s/\gamma) > 2.0$ m (see Note A-4.1.6.16.(6)(c)), d) except as provided in Clause (e), C_a shall be taken as 2.0 for roof areas within a distance of w_{ph} beyond the lowest edge of the panels, where w_{ph} is the horizontal projection of the panel width, w_p, along the sloped panel edges, and e) where the panels, panel supports or back plates obstruct snow from sliding under the panels, the load of the increased volume of snow in the gaps between the panels shall be considered to be uniformly distributed. (See Note A-4.1.6.16.(6).)

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4.1.7.2	3) A *building* shall be classified as very dynamically sensitive if a) its lowest natural frequency is less than or equal to 0.25 Hz, or b) its height is more than 6 times its minimum effective width as defined in Clause (2)(c)	**4.1.7.2**	3) A *building* shall be classified as very dynamically sensitive if a) its lowest natural frequency is less than or equal to 0.25 Hz, or b) it contains a human occupancy, and its height is more than 6 times its minimum effective width as defined in Clause (2)(c).
4.1.7.5	1) Applicable values of external pressure coefficients, C_p, are provided in a) Sentences (2) to (5), and b) Article 4.1.7.6. for certain shapes of low *buildings*. 5) For the design of balcony *guards*, the internal pressure coefficient, C_{pi}, shall be taken as zero and the value of C_p shall be taken as ±0.9, except that within a distance equal to the larger of 0.1W and 0.1D from a *building* corner, C_p shall be taken as ±1.2. New in 2020	**4.1.7.5**	1) Applicable values of external pressure coefficients, C_p, are provided in a) Sentences (2) to (9), and b) Article 4.1.7.6. for certain shapes of low *buildings*. 5) Except as provided in Sentence (6), for the design of balcony *guards*, the internal pressure coefficient, C_{pi}, shall be taken as zero and the value of C_p shall be taken as ±0.9, except that, within a distance equal to the larger of 0.1W and 0.1D from a *building* corner, C_p shall be taken as ±1.2. 6) Where the top of the balcony *guard* is 2.0 m or less below the roof surface, the values of C_p shall be taken as equal to those determined for parapets in Sentences (7) and (8). 7) To determine the contribution from parapets to the wind loads on the main structural system, the values of C_p shall be taken as a) on the outer faces, equal to those on the walls below, b) on the inner face of the windward parapet, equal to that on the upwind edge of a roof surface at the level of the top of the parapet, and c) on the inner faces of the other parapets, zero. 8) For the structural design of parapets themselves, the values of C_p shall be taken as equal to those specified in Sentence (7), except that the value of C_p on the inner face of the leeward parapet shall be taken as equal to that on the outer face of the windward parapet. 9) For the design of cladding on parapets, the values of C_p shall be taken as a) on the outer vertical surfaces, equal to those on the cladding on the walls below, and b) on the inner and top surfaces, equal to those on the cladding of a roof surface at the level of the top of the parapet.
4.1.7.6	*New in 2020*	**4.1.7.6**	10) The wind loads on balcony *guards* on low *buildings* shall be as specified in Sentences 4.1.7.5.(5) and (6). 11) The wind loads on parapets on low *buildings* shall be as specified in

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Code Ref.	Part 4	Code Ref.	Part 4
			Sentences 4.1.7.5.(7) to (9).
4.1.7.7	Internal Pressure Coefficient 1) The internal pressure coefficient, C_{pi}, shall be as prescribed in Table 4.1.7.7. New in 2020	**4.1.7.7**	1) The internal pressure coefficient, C_{pi}, for *buildings* shall be as prescribed in Table 4.1.7.7. 2) The internal pressure coefficient, C_{pi}, for cladding on parapets shall be –0.70 to +0.70. (See Note A-4.1.7.7.(2).)
4.1.7.9	Full and Partial Wind Loading 1) Except where the wind loads are derived from the combined C_pC_g values determined in accordance with Article 4.1.7.6., *buildings* and structural members shall be capable of withstanding the effects of the following loads: a) the full wind loads acting along each of the 2 principal horizontal axes considered separately, b) the wind loads described in Clause (a) but with 100% of the load removed from any one portion of the area, c) the wind loads described in Clause (a) but with both axes considered simultaneously at 75% of their full value, and d) the wind loads described in Clause (c) but with 50% of these loads removed from any portion of the area. (See Note A-4.1.7.9.(1).)	**4.1.7.9**	Full and Partial Wind Loading 1) Except where the wind loads are derived from the combined C_gC_p values determined in accordance with Article 4.1.7.6., *buildings* and structural members shall be capable of withstanding the effects of the following loads: a) the full wind loads acting along each of the 2 principal horizontal axes considered separately, b) 75% of the wind loads described in Clause (a) but offset from the central geometric axis of the *building* by 15% of its width normal to the direction of the force to produce the worst load effect, c) 75% of the wind loads described in Clause (a) but with both axes considered simultaneously, and d) 56% of the wind loads described in Clause (a) but with both axes considered simultaneously and offset from the central geometric axis of the *building* by 15% of its width normal to the direction of the force. (See Note A-4.1.7.9.(1).)
4.1.7.12	Wind Tunnel Procedure 1) Except as provided in Sentences (2) and (3), wind tunnel tests on scale models to determine wind loads on *buildings* shall be conducted in accordance with ASCE/SEI 49, "Wind Tunnel Testing for Buildings and Other Structures." 2) Where an adjacent *building* provides substantial sheltering effect, the wind loads for the main structural system shall be no lower than 80% of the loads determined from tests referred to in Sentence (1) with the effect of the sheltering *building* removed as applied to a) the base shear force for *buildings* with a ratio of height to minimum effective width, as defined in Sentence 4.1.7.2.(2), less than or equal to 1.0, or b) the base moment for *buildings* with a ratio of height to minimum effective	**4.1.7.12**	Attached Canopies on Low Buildings with a Height H ≤ 20 m (See Note A-4.1.7.12.) 1) For the purposes of this Article, "attached canopy" shall mean a horizontal canopy with a maximum slope of 2% that is attached to a *building* wall at any height, h_c, above ground level. 2) The specified external wind pressure, p, and the specified net external wind pressure, p_{net}, for attached canopies on exterior walls of low *buildings* with a height H ≤ 20 m shall be determined as follows: $p = I_W q C_e C_t C_g C_p$, and $p_{net} = I_W q C_e C_t (C_g C_p)_{net}$

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	width greater than 1.0. 3) For the design of cladding and secondary structural members, the exterior wind loads determined from the wind tunnel tests shall be no less onerous than those determined by analysis in accordance with Article 4.1.7.3. using the following assumptions: a) $C_p = \pm 0.72$ and $C_g = 2.5$, where the *building*'s height is greater than 20 m or greater than its minimum effective width, and b) C_pC_g = 80% of the values for zones w and r provided in Article 4.1.7.6., where the *building*'s height is less than or equal to 20 m and no greater than its minimum effective width.		where p = specified external wind pressure acting statically and in a direction normal to the upper or lower surface of the canopy, considered positive when acting towards the surface and negative when acting away from the surface, p_{net} = specified net external wind pressure acting statically on the canopy, considered positive when acting in a downward direction and negative when acting in an upward direction, I_W, q, C_e, C_t = as defined in Sentence 4.1.7.3.(1), C_gC_p = gust pressure coefficient on the upper or lower surface of the canopy, as given in Figure 4.1.7.12.-A, and $(C_gC_p)_{net}$ = net gust pressure coefficient on the canopy, considering simultaneous contributions from the upper and lower surfaces of the canopy, as given in Figure 4.1.7.12.-B.

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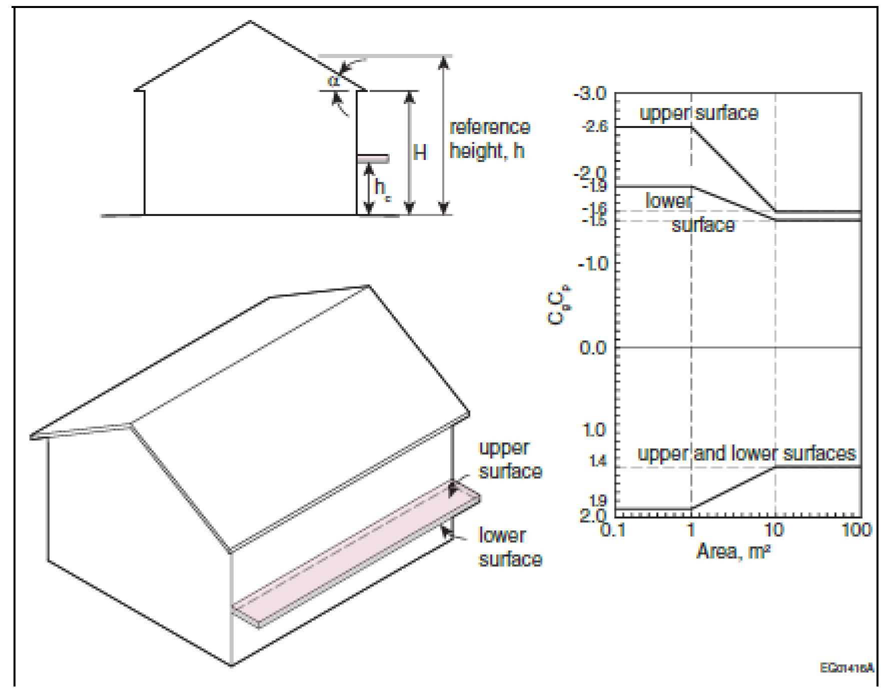
Figure 4.1.7.12.-A
Gust pressure coefficients on the upper and lower surfaces of attached canopies with no gap between the canopy and the building
Forming Part of Sentence 4.1.7.12.(2)



Notes to Figure 4.1.7.12.-A:

(1) The coefficients apply for any roof slope, ⟨.

(2) The reference height, h, is the mid-height of the roof or 6 m, whichever is greater.

(3) Positive C_gC_p values denote forces acting towards the upper or lower surface of the canopy, whereas
negative C_gC_p values denote forces acting away from the surface. Each structural element must be designed
to resist both the positive and negative forces.

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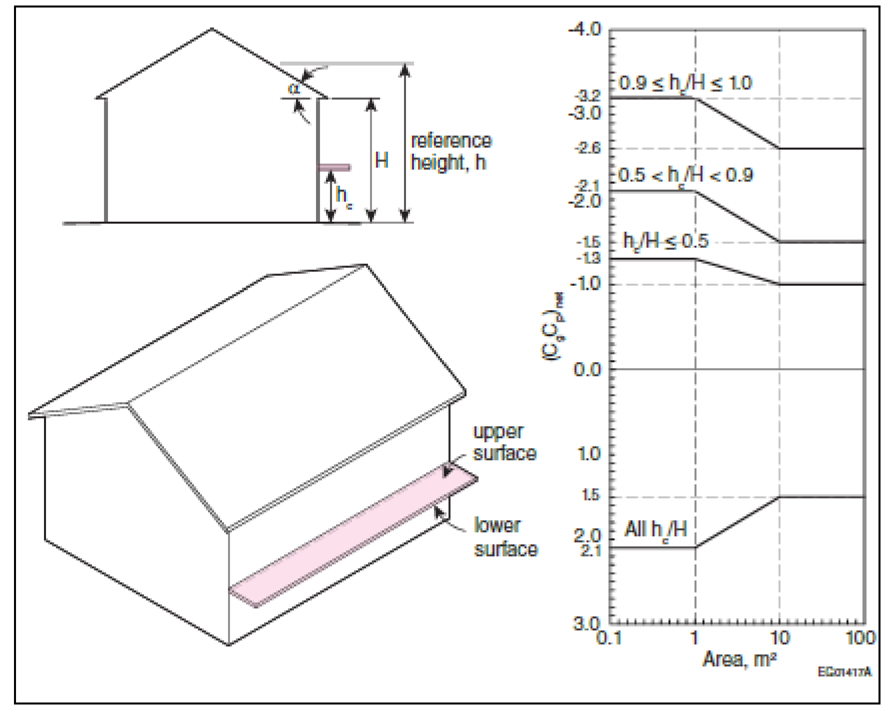
Figure 4.1.7.12.-B
Net gust pressure coefficients on attached canopies, considering simultaneous contributions from the upper and lower surfaces of the canopy
Forming Part of Sentence 4.1.7.12.(2)



Notes to Figure 4.1.7.12.-B:

(1) The coefficients apply for any roof slope, ⟨.

(2) The reference height, h, is the mid-height of the roof or 6 m, whichever is greater.

(3) Positive $(C_gC_p)_{net}$ values denote net forces acting in a downward direction on the canopy, whereas negative
$(C_gC_p)_{net}$ values denote net forces acting in an upward direction on the canopy. The canopy must be designed
to resist both the positive and negative net forces.

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Code Ref.	Part 4	Code Ref.	Part 4
4.7.13	New in 2020	**4.1.7.13**	Roof-Mounted Solar Panels on Buildings of Any Height (See Note A-4.1.7.13.) 1) Where solar panels are installed on a roof, the roof wind loads shall account for the wind loads on the solar panels, as determined in accordance with Sentences (2) to (7), or shall be determined in the same way as for the roof without solar panels, whichever approach results in the most critical effect. 2) For an array of solar panels where the panels are installed close and parallel to the roof surface with their upper surface not more than 250mmabove the roof surface and with gaps around the panels of not less than 6 mm, the net positive or negative pressure difference between the upper and lower surfaces of a panel or the array shall be calculated as follows: $p = I_W q C_e C_t C_g C_p E \gamma_a$ where I_W, q,C_e, C_t, C_g, C_p = as defined in Sentence 4.1.7.3.(1), determined in the same manner as for the roof cladding, E = edge factor, as provided in Sentence (4), and γ_a = pressure equalization factor, as provided in Sentence (3). 3) The pressure equalization factor, γ_a, in Sentence (2) shall be a) for a panel or an array where the panel chord length, L_p, is greater than 2 m or for a panel or an array that is within a distance of $2h_2$ from the roof edge or ridge, where h_2 is the height of the panel's highest point above the roof surface, taken as 1.0, and b) for other panels or arrays, determined from Figure 4.1.7.13.-A based on the area of the panel or array over which the wind load is being calculated.

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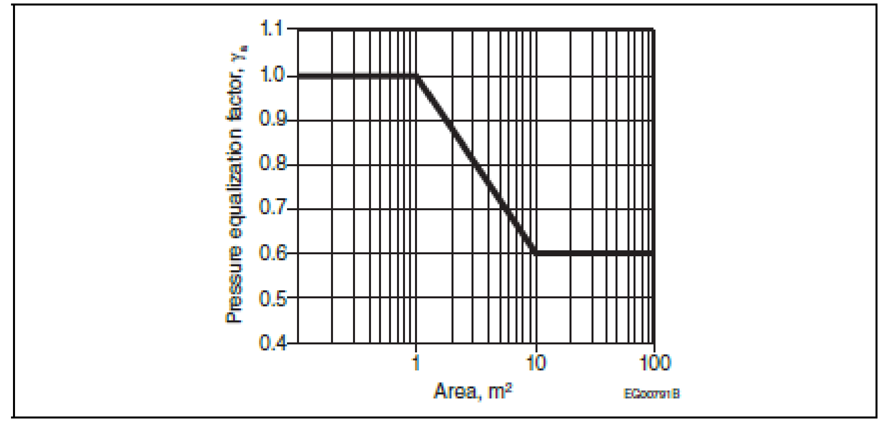
Figure 4.1.7.13.-A
Pressure equalization factor, γ_a, for solar panels or arrays mounted on roofs of buildings of any height
Forming Part of Clause 4.1.7.13.(3)(b)



4) The edge factor, E, in Sentence (2) shall be taken as

a) 1.5 within a distance of $1.5L_p$ from an exposed edge of the array of solar panels, as defined in Sentence (5), and
b) 1.0 elsewhere.

5) For the purposes of Clause (4)(a), an exposed edge of the array of solar panels shall be considered to occur
a) where the distance to the next row of panels or the distance across a gap in the same row of panels exceeds $4h_2$ or 1.2 m, whichever is greater, or
b) where the distance to the roof edge exceeds $4h_2$ or 1.2 m, whichever is greater, and exceeds 0.5h, where h is the reference height of the roof.

6) For an array of solar panels mounted on a roof with a slope, α, less than or equal to 7°, where the panels are tilted relative to the roof surface, have a chord length, L_p, not greater than 2 m, and are installed such that the height of their lowest point above the roof surface, h_1, is not greater than 0.6 m, the height of their highest point above the roof surface, h_2, is not greater than 1.2 m, and their tilt angle relative to the roof surface, ω, is not greater than 35°, or where the panels are installed parallel to the roof surface with their upper surface greater than 250 mm above the roof surface and with gaps not less than 6 mm between the panels, the net positive or negative pressure difference between the upper and the lower surfaces of a panel or the array shall be calculated as follows

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Code Ref.	Part 4	Code Ref.	Part 4
			$p_{net} = I_W q C_e C_t (C_g C_p)_{net}$ where I_W, q, C_e, C_t = as defined in Sentence 4.1.7.3.(1), determined in the same manner as for the roof cladding, and $(C_g C_p)_{net}$ = net gust pressure coefficient, as provided in Sentence (7). 7) The net gust pressure coefficient, $(C_g C_p)_{net}$, in Sentence (6) shall be calculated as follows: $(C_g C_p)_{net} = \pm \gamma_p \gamma_c E (C_g C_p)_n$ where γ_p = parapet factor, determined as the lesser of 1.2 and $(0.9 + h_{pt}/h)$, γ_c = chord factor, determined as the greater of $(0.6 + 0.2L_p)$ and 0.8, E = as defined in Sentence (2), and $(C_g C_p)_n$ = normalized gust pressure coefficient, determined from Figure 4.1.7.13.-B based on ω and A_N, where h_{pt} = height of the parapet above the roof surface, in m, h = reference height of the roof, in m, L_p = panel chord length, in m, ω = panel tilt angle relative to the roof surface, and A_N = normalized panel or array area, calculated as $\bar{A}_N = \frac{1\,000A}{\max(L_b^2, 25)}$ where A = panel or array area over which the wind load is being calculated, in m^2, and L_b = normalized *building* length, in m, determined as the lesser of $(0.4\sqrt{hW_L})$, h where W_L = longest horizontal dimension of the *building*, in m, and W_S = smallest horizontal dimension of the *building*, in m.

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Code Ref.	Part 4	Code Ref.	Part 4
4.1.8.1	Analysis 1) Except as permitted in Sentence (2), the deflections and specified loading due to earthquake motions shall be determined according to the requirements of Articles 4.1.8.2. to 4.1.8.22. 2) Where $I_E F_s S_a(0.2)$ and $I_E F_s S_a(2.0)$ are less than 0.16 and 0.03 respectively, the deflections and specified loading due to earthquake motions are permitted to be determined in accordance with Sentences (3) to (15), where a) I_E is the earthquake importance factor and has a value of 0.8, 1.0, 1.3 and 1.5 for *buildings* of Low, Normal, High and Post-Disaster importance respectively, b) F_s is the site coefficient based on the average $_{60}$ or s_u, as defined in Article 4.1.8.2., for the top 30 m of *soil* below the footings, pile caps, or mat *foundations* and has a value of i) 1.0 for *rock* sites or when $_{60} > 50$ or $s_u > 100$ kPa, ii) 1.6 when $15 \leq {}_{60} \leq 50$ or 50 kPa $\leq s_u \leq$ 100 kPa, and iii) 2.8 for all other cases, and c) $S_a(T)$ is the 5% damped spectral response acceleration value for period T, determined in accordance with Subsection 1.1.3. $$V_s = F_s S_a(T_s) I_E W_t / R_s$$ where $S_a(T_s)$ =valueofS_a at T_s determined by linear interpolation between the value of S_a at 0.2 s, 0.5 s, and 1.0 s, and $= S_a(0.2)$ for $T_s \leq 0.2$ s, W_t = sum ofW_i over the height of the *building*, where W_i is defined in Article 4.1.8.2., and $R_s = 1.5$, except $R_s = 1.0$ for structures where the *storey* strength is less than that in the *storey* above and for an unreinforced masonry SFRS, where h_n = height above the base, in m, as defined in Article 4.1.8.2., except that V_s shall not be less than $F_s S_a(1.0) I_E W_t / R_s$ and, in cases where $R_s = 1.5$, V_s need not be greater than $F_s S_a(0.5) I_E W_t / R_s$.	**4.1.8.1**	Analysis 1) Except as permitted in Sentence (2), the deflections and specified loading due to earthquake motions shall be determined according to the requirements of Articles 4.1.8.2. to 4.1.8.23. 2) Where $I_E F_s S_a(0.2, X_{450})$ and $I_E F_s S_a(2.0, X_{450})$ are less than 0.16 and 0.03 respectively, the deflections and specified loading due to earthquake motions are permitted to be determined in accordance with Sentences (3) to (15), where a) I_E is the earthquake importance factor and has a value of 0.8, 1.0, 1.3 and 1.5 for *buildings* in the Low, Normal, High and Post-disaster Importance Categories respectively, b) F_s is the site coefficient based on the average $_{60}$ or , as defined in Article 4.1.8.2., for the top 30 m of *soil* below the footings, *pile* caps, or mat *foundations* and has a value of i) 1.0 for *rock* sites or when $_{60} > 50$ or > 100 kPa, ii) 1.6 when $15 \leq {}_{60} \leq 50$ or 50 kPa ≤ ≤ 100 kPa, and iii) 2.8 for all other cases, and c) $S_a(T, X_{450})$ is the 5%-damped spectral acceleration value at period T for site designation X_{450}, as defined in Article 4.1.8.2., determined in accordance with Subsection 1.1.3. and corresponding to a 2% probability of exceedance in 50 years. $$V_s = F_s S_a(T_s, X_{450}) I_E W / R_s$$ where $S_a(T_s, X_{450})$ =valueofS_a (T_s, X_{450}) determined by linear interpolation between the values of $S_a(0.2, X_{450})$, $S_a(0.5, X_{450})$ and $S_a(1.0, X_{450})$, $= S_a(0.2, X_{450})$ for$T_s \leq 0.2$ s, and $= S_a(1.0, X_{450})$ for$T_s \geq 1.0$ s, W =sumofW_i over the height of the *building*, where W_i is defined in Article 4.1.8.2., and $R_s = 1.5$, except $R_s = 1.0$ for structures where the *storey* strength is less than that in the *storey* above and for an unreinforced masonry SFRS, where h_n = height, in m, above the base to level n, as defined in Article 4.1.8.2., and

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	$V_{sp} = 0.1 F_s I_E W_p$		N =total number of *storeys* above exterior *grade* to level n, as defined in Article 4.1.8.2., except that, in cases where $R_s = 1.5$, V_s need not be greater than $F_s S_a(0.5,X_{450}) I_E W/R_s$. $V_{sp} = 0.9 S_a(0.2, X_{450}) F_s I_E W_p$
4.1.8.2	F_a = site coefficient for application in Subsection 4.1.8., as defined in Sentence 4.1.8.4.(7), F(PGA) = site coefficient for PGA, as defined in Sentence 4.1.8.4.(5), F(PGV) = site coefficient for PGV, as defined in Sentence 4.1.8.4.(5), F_s = site coefficient as defined in Sentence 4.1.8.1.(2) for application in Article 4.1.8.1., F(T) = site coefficient for spectral acceleration, as defined in Sentence 4.1.8.4.(5), Some new in 2020	4.1.8.2	F_a = acceleration-based site coefficient for application in standards referenced in Subsection 4.1.8., as defined in Sentence 4.1.8.4.(7), F_s = site coefficient as defined in Sentence 4.1.8.1.(2) for application in Article 4.1.8.1., F_t = portion of V to be concentrated at the top of the structure, as defined in Sentence 4.1.8.11.(7), F_v = velocity-based site coefficient for application in standards referenced in Subsection 4.1.8., as defined in Sentence 4.1.8.4.(7), F_x = lateral force applied to level x, as defined in Sentence 4.1.8.11.(7), M_x = overturning moment at level x, as defined in Sentence 4.1.8.11.(8), N =totalnumberof *storeys* above exterior *grade* to level n, = average standard penetration resistance, in blows per 0.3 m, in the top 30 m of *soil*, corrected to a rod energy efficiency of 60% of the theoretical maximum, PGA(X) = peak ground acceleration, expressed as a ratio to gravitational acceleration, for site designation X, as defined in Sentence 4.1.8.4.(1), PGV(X) = peak ground velocity, in m/s, for site designation X, as defined in Sentence 4.1.8.4.(1), PI = plasticity index for *soil*, R_d = ductility-related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour, as defined in Article 4.1.8.9., R_s = combined overstrength and ductility-related modification factor, as defined

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Code Ref.	Part 4	Code Ref.	Part 4
			in Sentence 4.1.8.1.(7), for application in Article 4.1.8.1., $S_a(T,X)$ = 5%-damped spectral acceleration, expressed as a ratio to gravitational acceleration, at period T for site designation X, as defined in Sentence 4.1.8.4.(1), SC = Seismic Category assigned to a *building* based on its Importance Category and the design spectral acceleration values at periods of 0.2 s and 1.0 s, as defined in Article 4.1.8.5., X = site designation, either X_V or X_S, X_S = site designation in terms of Site Class, where S is the Site Class determined in accordance with Sentence 4.1.8.4.(3), X_V = site designation in terms of V_{s30}, where V is the V_{s30} value calculated from in situ measurements of shear wave velocity, X_{450} = site designation X_V with V_{s30} = 450 m/s, δ_{ave} = average displacement of the structure at level x, as defined in Sentence 4.1.8.11.(10), and δ_{max} = maximum displacement of the structure at level x, as defined in Sentence 4.1.8.11.(10).
4.1.8.4	Site Properties 1) The peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values, $S_a(T)$, for the reference ground conditions (Site Class C in Table 4.1.8.4.-A) for periods T of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s shall be determined in accordance with Subsection 1.1.3. and are based on a 2% probability of exceedance in 50 years.	4.1.8.4	Site Properties 1) For site designation X, as determined in accordance with Sentence (2) or (3), the peak ground acceleration, PGA(X), the peak ground velocity, PGV(X), and the 5%-damped spectral acceleration values, $S_a(T,X)$, at periods T of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s shall a) except as provided in Sentence (4), be determined in accordance with Subsection 1.1.3., and b) except as provided in Article 4.1.8.23., correspond to a 2% probability of exceedance in 50 years. 2) Except as provided in Sentence (3), the site designation referred to in Sentence (1) shall be determined using the average shear wave velocity, V_{s30}, calculated from in situ measurements of shear wave velocity, as follows: a) for the ground profiles described in Table 4.1.8.4.-A, the site designation shall be determined in accordance with the Table, and b) for all other ground profiles, the site designation shall be X_V, where V is the value of V_{s30}. (See Note A-4.1.8.4.(2) and (3).)

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Table 4.1.8.4.-A
Site Classification for Seismic Site Response
Forming Part of Sentences 4.1.8.4.(1) to (3)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per Note A-4.1.8.4.(3) and Table 4.1.8.4.-A		
		Average Shear Wave Velocity, $\bar{V}_{s30}$, m/s	Average Standard Penetration Resistance, $\bar{N}_{60}$	*Soil* Undrained Shear Strength, s_u
A	Hard *rock*(1)(2)	$\bar{V}_{s30} > 1500$	n/a	n/a
B	*Rock*(1)	$760 < \bar{V}_{s30} \leq 1500$	n/a	n/a
C	Very dense *soil* and soft *rock*	$360 < \bar{V}_{s30} < 760$	$\bar{N}_{60} > 50$	$s_u > 100$ kPa
D	Stiff *soil*	$180 < \bar{V}_{s30} < 360$	$15 \leq \bar{N}_{60} \leq 50$	50 kPa $< s_u \leq$ 100 kPa
E	Soft *soil*	$\bar{V}_{s30} < 180$	$\bar{N}_{60} < 15$	$s_u < 50$ kPa
		Any profile with more than 3 m of *soil* with the following characteristics: • plasticity index: PI > 20 • moisture content: w ≥ 40%, and • undrained shear strength: $s_u < 25$ kPa		
F	Other *soils*(3)	Site-specific evaluation required		

Notes to Table 4.1.8.4.-A:

(1) Site Classes A and B, hard *rock* and *rock*, are not to be used if there is more than 3 m of softer materials between the *rock* and the underside of footing or mat *foundations*. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials (see Note A-4.1.8.4.(3) and Table 4.1.8.4.-A).

(2) Where has been measured in-situ, the F(T) values for Site Class A derived from Tables 4.1.8.4.-B to 4.1.8.4.-G are permitted to be multiplied by the factor 0.04 + (1500/)..

(3) Other *soils* include:

(a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,

(b) peat and/or highly organic clays greater than 3 m in thickness,

(c) highly plastic clays (PI > 75) more than 8 m thick, and

(d) soft to medium stiff clays more than 30 m thick.

Table 4.1.8.4.-A
Exceptions for Site Designation Using V_{s30} Calculated from In Situ Measurements
Forming Part of Sentence 4.1.8.4.(2)

Ground Profile Characteristics		Site Designation
Average Shear Wave Velocity in Top 30 m, V_{s30}, Calculated from In Situ Measurements, in m/s	Additional Characteristics	
$V_{s30} > 760$	Ground profile contains more than 3 m of softer materials between *rock* and the underside of footing or mat foundations	X_{760}
$V_{s30} > 140$	Ground profile contains more than 3 m of *soil* with all the following characteristics: • plasticity index, PI > 20, • moisture content, w ≥ 40%, and • undrained shear strength, $s_u < 25$ kPa	X_E
$V_{s30} > 140$	Ground profile contains • liquefiable *soil*, quick and highly sensitive clay, collapsible weakly cemented *soil*, or other *soil* susceptible to failure or collapse under seismic loading, • more than 3 m of peat and/or highly organic clay, • more than 8 m of highly plastic *soil* (with PI > 75), or • more than 30 m of soft to medium-stiff clay	X_F
$V_{s30} \leq 140$	n/a	X_F

3) Where V_{s30} calculated from in situ measurements is not available, the site designation referred to in Sentence (1) shall be X_S, where S is the Site Class determined using the energy-corrected average standard penetration resistance, , or the average undrained shear strength, , in accordance with Table 4.1.8.4.-B, and being calculated based on rational analysis. (See Notes A-4.1.8.4.(3) and A-4.1.8.4.(2) and (3).)

Table 4.1.8.4.-B
Site Classes, S, for Site Designation X_S
Forming Part of Sentence 4.1.8.4.(3)

Site Class, S	Ground Profile	Ground Profile Characteristics		
		Average Shear Wave Velocity in Top 30 m, V_{s30}, in m/s(1)	Average Standard Penetration Resistance in Top 30 m, $\bar{N}_{60}$, in Blows per 0.3 m	Average Undrained Shear Strength in Top 30 m, $\bar{s}_u$, in kPa
A	Hard *rock*(2)	$V_{s30} > 1\,500$	n/a	n/a
B	*Rock*(2)	$760 < V_{s30} \leq 1\,500$	n/a	n/a
C	Very dense *soil* and soft *rock*	$360 < V_{s30} \leq 760$	$\bar{N}_{60} > 50$	$\bar{s}_u > 100$

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Code Ref.	Part 4	Code Ref.	Part 4

Table 4.1.8.4.-B
Values of F(0.2) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(0.2)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	0.69	0.69	0.69	0.69	0.69
B	0.77	0.77	0.77	0.77	0.77
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.09	1.00	0.94	0.90
E	1.64	1.24	1.05	0.93	0.85
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.-B:
(1) See Sentence 4.1.8.4.(6).

Table 4.1.8.4.-C
Values of F(0.5) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(0.5)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	0.57	0.57	0.57	0.57	0.57
B	0.65	0.65	0.65	0.65	0.65
C	1.00	1.00	1.00	1.00	1.00
D	1.47	1.30	1.20	1.14	1.10
E	2.47	1.80	1.48	1.30	1.17
F	(1)	(1)	(1)	(1)	(1)

Table 4.1.8.4.-B (Continued)

Site Class, S	Ground Profile	Ground Profile Characteristics		
		Average Shear Wave Velocity in Top 30 m, V_{s30}, in m/s(1)	Average Standard Penetration Resistance in Top 30 m, $\bar{N}_{60}$, in Blows per 0.3 m	Average Undrained Shear Strength in Top 30 m, $\bar{s}_u$, in kPa
D	Stiff *soil*	$180 < V_{s30} \le 360$	$15 < \bar{N}_{60} \le 50$	$50 < \bar{s}_u \le 100$
E	Soft *soil*	$140 < V_{s30} \le 180$	$10 < \bar{N}_{60} \le 15$	$40 < \bar{s}_u \le 50$
		Any ground profile other than Site Class F that contains more than 3 m of *soil* with all the following characteristics: • plasticity index, PI > 20, • moisture content, $w \ge 40\%$, and • undrained shear strength, $s_u < 25$ kPa		
F	Other *soils*(3)	$V_{s30} \le 140$	$\bar{N}_{60} \le 10$	$\bar{s}_u \le 40$
		Any ground profile that contains • liquefiable *soil*, quick and highly sensitive clay, collapsible weakly cemented *soil*, or other *soil* susceptible to failure or collapse under seismic loading, • more than 3 m of peat and/or highly organic clay, • more than 8 m of highly plastic *soil* (with PI > 75), or • more than 30 m of soft to medium-stiff clay		

Notes to Table 4.1.8.4.-B:

(1) See Note A-4.1.8.4.(2) and (3).

(2) Site designations X_A and X_B, corresponding to Site Classes A and B, are not to be used in cases where the ground profile contains more than 3 m of softer materials between *rock* and the underside of footing or mat *foundations*. The appropriate site designation for such cases is X_{760}.

(3) Site-specific geotechnical evaluation is required.

4) Site-specific geotechnical evaluation is required to determine the values of $PGA(X_F)$, $PGV(X_F)$ and $S_a(T,X_F)$ for site designation X_F.

5) Where structures on liquefiable *soils* have a fundamental lateral period, T_a, of 0.5 s or less, the site designation X and the corresponding values of $S_a(T,X)$ and PGA(X) are permitted to be determined in accordance with Sentence (1) by assuming that the *soils* are not liquefiable.

6) The design spectral acceleration, S(T), shall be determined in accordance with Table 4.1.8.4.-C, using log–log or linear interpolation for intermediate values of T. (See Note A-4.1.8.4.(6).)

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Code Ref.	Part 4	Code Ref.	Part 4

Table 4.1.8.4.-D
Values of F(1.0) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(1.0)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	0.57	0.57	0.57	0.57	0.57
B	0.63	0.63	0.63	0.63	0.63
C	1.00	1.00	1.00	1.00	1.00
D	1.55	1.39	1.31	1.25	1.21
E	2.81	2.08	1.74	1.53	1.39
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.-D:
(1) See Sentence 4.1.8.4.(6).

Table 4.1.8.4.-E
Values of F(2.0) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(2.0)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	0.58	0.58	0.58	0.58	0.58
B	0.63	0.63	0.63	0.63	0.63
C	1.00	1.00	1.00	1.00	1.00
D	1.57	1.44	1.36	1.31	1.27
E	2.90	2.24	1.92	1.72	1.58
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.-E:
(1) See Sentence 4.1.8.4.(6).

Table 4.1.8.4.-F
Values of F(5.0) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(5.0)				
	$PGA_{ref} \le 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \ge 0.5$
A	0.61	0.61	0.61	0.61	0.61
B	0.64	0.64	0.64	0.64	0.64
C	1.00	1.00	1.00	1.00	1.00
D	1.58	1.48	1.41	1.37	1.34
E	2.93	2.40	2.14	1.96	1.84
F	(1)	(1)	(1)	(1)	(1)

Table 4.1.8.4.-C
Design Spectral Acceleration
Forming Part of Sentence 4.1.8.4.(6)

Period, T, in s	Design Spectral Acceleration, S(T)
≤ 0.2	$S_a(0.2,X)$ or $S_a(0.5,X)$, whichever is greater
0.5	$S_a(0.5,X)$
1.0	$S_a(1.0,X)$
2.0	$S_a(2.0,X)$
5.0	$S_a(5.0,X)$
10.0	$S_a(10.0,X)$

7) Where required for the application of a standard referenced in this Subsection, the acceleration-based site coefficient, F_a, for site designation X shall be taken as $S(0.2)/S_a(0.2,X_{450})$ and the velocity-based site coefficient, F_v, for site designation X shall be taken as $S(1.0)/S_a(1.0,X_{450})$.

2015 NBC		2020 NBC	CHANGES MADE
Code Ref.	Part 4	Code Ref.	Part 4

Table 4.1.8.4.-G
Values of F(10.0) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(10.0)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.67	0.67	0.67	0.67	0.67
B	0.69	0.69	0.69	0.69	0.69
C	1.00	1.00	1.00	1.00	1.00
D	1.49	1.41	1.37	1.34	1.31
E	2.52	2.18	2.00	1.88	1.79
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.-G:
(1) See Sentence 4.1.8.4.(6).

Table 4.1.8.4.-H
Values of F(PGA) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(PGA)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.90	0.90	0.90	0.90	0.90
B	0.87	0.87	0.87	0.87	0.87
C	1.00	1.00	1.00	1.00	1.00
D	1.29	1.10	0.99	0.93	0.88
E	1.81	1.23	0.98	0.83	0.74
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.-H:
(1) See Sentence 4.1.8.4.(6).

Table 4.1.8.4.-I
Values of F(PGV) as a Function of Site Class and PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Site Class	Values of F(PGV)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.62	0.62	0.62	0.62	0.62
B	0.67	0.67	0.67	0.67	0.67
C	1.00	1.00	1.00	1.00	1.00
D	1.47	1.30	1.20	1.14	1.10
E	2.47	1.80	1.48	1.30	1.17
F	(1)	(1)	(1)	(1)	(1)

2015 NBC Code Ref.	Part 4	2020 NBC Code Ref.	Part 4 (Changes Made)
4.1.8.5	Importance Factor 1) The earthquake importance factor, I_E, shall be determined according to Table 4.1.8.5.	**4.1.8.5**	Importance Factor and Seismic Category 1) The earthquake importance factor, I_E, shall be determined according to Table 4.1.8.5.-A

2015 NBC		2020 NBC	**CHANGES MADE**
Code Ref.	Part 4	Code Ref.	Part 4
	New on 2020 New on 2020		2) *Buildings* shall be assigned a Seismic Category in accordance with Table 4.1.8.5.-B. **Table 4.1.8.5.-B** **Seismic Categories for Buildings** Forming Part of Sentence 4.1.8.5.(2) Seismic Category(1) / $I_E S(0.2)$ / $I_E S(1.0)$ SC1 / $I_E S(0.2) < 0.2$ / $I_E S(1.0) < 0.1$ SC2 / $0.2 \le I_E S(0.2) < 0.35$ / $0.1 \le I_E S(1.0) < 0.2$ SC3 / $0.35 \le I_E S(0.2) \le 0.75$ / $0.2 \le I_E S(1.0) \le 0.3$ SC4 / $I_E S(0.2) > 0.75$ / $I_E S(1.0) > 0.3$ Notes to Table 4.1.8.5.-B: (1) The Seismic Category of a *building* shall be taken as the more severe of the categories determined on the basis of $I_E S(0.2)$ and $I_E S(1.0)$, irrespective of the fundamental lateral period of the *building*, T_a.
4.1.8.6	3) Except as required by Article 4.1.8.10., in cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, structures designated as irregular must satisfy the provisions referenced in Table 4.1.8.6. **Table 4.1.8.6.** **Structural Irregularities(1)(2)** Forming Part of Sentence 4.1.8.6.(1) Type / Irregularity Type and Definition / Notes 1 / **Vertical Stiffness Irregularity** Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a *storey* is less than 70% of the stiffness of any adjacent *storey*, or less than 80% of the average stiffness of the three *storeys* above or below. / (3)(4)	**4.1.8.6**	3) Except as required by Article 4.1.8.10., where the Seismic Category is SC3 or SC4, structures designated as irregular must satisfy the provisions referenced in Table 4.1.8.6. **Table 4.1.8.6.** **Structural Irregularities(1)(2)** Forming Part of Sentences 4.1.8.6.(1) and (3), Clause 4.1.8.7.(1)(c) and Article 4.1.8.10. Type / Irregularity Type and Definition / Notes 1 / **Vertical Stiffness Irregularity** For concrete and masonry shear walls, vertical stiffness irregularity shall be considered to exist where the lateral stiffness of the SFRS in any *storey* is less than 70% of the stiffness in an adjacent *storey*, or less than 80% of the average stiffness in the three *storeys* above or below. For all other types of SFRS, vertical stiffness irregularity shall be considered to exist where the inter*storey* deflection under lateral earthquake forces divided by the inter*storey* height, h_s, of any *storey* is greater than 130% of that of an adjacent *storey*. / (3)(4)(5)

2015 NBC		2020 NBC	<u>CHANGES MADE</u>
Code Ref.	Part 4	Code Ref.	Part 4
	New on 2020 Notes to Table 4.1.8.6.: (1) One-*storey* penthouses with a weight of less than 10% of the level below need not be considered in the application of this Table. (2) See Note A-Table 4.1.8.6. (3) See Article 4.1.8.7. (4) See Article 4.1.8.10. (5) See Article 4.1.8.15. (6) See Sentences 4.1.8.11.(10), (11) and 4.1.8.12.(4). (7) See Article 4.1.8.8		10 | **Sloped Column Irregularity** Sloped column irregularity shall be considered to exist where a vertical member that is inclined more than 2° from the vertical supports a portion of the weight of the *building* in axial compression. | (4) Notes to Table 4.1.8.6.: (1) One-*storey* penthouses with a weight of less than 10% of the level below need not be considered in the application of this Table. (2) See Note A-Table 4.1.8.6. (3) See Article 4.1.8.7. (4) See Article 4.1.8.10. (5) Increased stiffness in *storeys* below *grade* need not be considered in the determination of vertical stiffness irregularity. (6) See Article 4.1.8.15. (7) See Sentences 4.1.8.11.(10) and (11), and 4.1.8.12.(4). (8) See Article 4.1.8.8.
4.1.8.7	1) Analysis for design earthquake actions shall be carried out in accordance with the Dynamic Analysis Procedure described in Article 4.1.8.12. (see Note A-4.1.8.7.(1)), except that the Equivalent Static Force Procedure described in Article 4.1.8.11. may be used for structures that meet any of the following criteria: a) in cases where $I_E F_a S_a(0.2)$ is less than 0.35,	**4.1.8.7**	1) Analysis for earthquake actions shall be carried out in accordance with the Dynamic Analysis Procedure described in Article 4.1.8.12. (see Note A-4.1.8.7.(1)), except that the Equivalent Static Force Procedure described in Article 4.1.8.11. may be used for structures that meet any of the following criteria: a) where the Seismic Category is SC1 or SC2,
4.1.8.8	b) where the components of the SFRS are not oriented along a set of orthogonal axes and $I_E F_a S_a(0.2)$ is less than 0.35, independent analyses about any two orthogonal axes is permitted, or c) where the components of the SFRS are not oriented along a set of orthogonal axes and $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, analysis of the structure independently in any two orthogonal directions for 100% of the prescribed earthquake loads applied in one direction plus 30% of the prescribed earthquake loads in the perpendicular direction, with the combination requiring the greater element strength being used in the design.	**4.1.8.8**	b) where the components of the SFRS are not oriented along a set of orthogonal axes and the Seismic Category is SC1 or SC2, independent analyses about any two orthogonal axes is permitted, or c) where the components of the SFRS are not oriented along a set of orthogonal axes and the Seismic Category is SC3 or SC4, analysis of the structure independently in any two orthogonal directions for 100% of the specified earthquake loads applied in one direction plus 30% of the specified earthquake loads in the perpendicular direction, with the combination requiring the greater element strength being used in the design.
4.1.8.9	1) Except as provided in Sentence 4.1.8.20.(7), the values of R_d and R_o and the corresponding system restrictions shall conform to Table 4.1.8.9. and the requirements of this Subsection.	**4.1.8.9**	1) Except as provided in Articles 4.1.8.20. and 4.1.8.22., the values of R_d and R_o and the corresponding system restrictions shall conform to Table 4.1.8.9. and the requirements of this Subsection.

2015 NBC		2020 NBC	CHANGES MADE
Code Ref.	Part 4	Code Ref.	Part 4

Table 4.1.8.9.
SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions[1]
Forming Part of Sentences 4.1.8.9.(1) and (5)

Type of SFRS	R_d	R_o	Restrictions[2]				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Steel Structures Designed and Detailed According to CSA S16[3][4]							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30	30

Table 4.1.8.9.
SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions[1]
Forming Part of Sentences 4.1.8.9.(1) and (5), 4.1.8.10.(5) and (6), 4.1.8.11.(12), 4.1.8.15.(9) and 4.1.8.20.(8)

Type of SFRS	R_d	R_o	Restrictions[2]			
			Seismic Category			
			SC1	SC2	SC3	SC4
Steel Structures Designed and Detailed According to CSA S16[3][4]						
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30

2015 NBC		2020 NBC	**CHANGES MADE**
Code Ref.	Part 4	Code Ref.	Part 4

Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Concrete Structures Designed and Detailed According to CSA A23.3							
Ductile moment-resisting frames	4.0	1.7	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	2.5	1.4	NL	NL	60	40	40
Ductile coupled walls	4.0	1.7	NL	NL	NL	NL	NL
Moderately ductile coupled walls	2.5	1.4	NL	NL	NL	60	60
Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL	NL
Moderately ductile partially coupled walls	2.0	1.4	NL	NL	NL	60	60
Ductile shear walls	3.5	1.6	NL	NL	NL	NL	NL
Moderately ductile shear walls	2.0	1.4	NL	NL	NL	60	60
Conventional construction							
Moment-resisting frames	1.5	1.3	NL	NL	20	15	10(5)
Shear walls	1.5	1.3	NL	NL	40	30	30
Two-way slabs without beams	1.3	1.3	20	15	NP	NP	NP
Tilt-up construction							
Moderately ductile walls and frames	2.0	1.3	30	25	25	25	25
Limited ductility walls and frames	1.5	1.3	30	25	20	20	20(6)
Conventional walls and frames	1.3	1.3	25	20	NP	NP	NP
Other concrete SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	NP
Timber Structures Designed and Detailed According to CSA O86							
Shear walls							
Nailed shear walls: wood-based panel	3.0	1.7	NL	NL	30	20	20
Shear walls: wood-based and gypsum panels in combination	2.0	1.7	NL	NL	20	20	20
Braced or moment-resisting frames with ductile connections							
Moderately ductile	2.0	1.5	NL	NL	20	20	20
Limited ductility	1.5	1.5	NL	NL	15	15	15
Other wood- or gypsum-based SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	NP
Masonry Structures Designed and Detailed According to CSA S304							
Ductile shear walls	3.0	1.5	NL	NL	60	40	40
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40	40
Conventional construction							
Shear walls	1.5	1.5	NL	60	30	15	15
Moment-resisting frames	1.5	1.5	NL	30	NP	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	NP

Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)			
			Seismic Category			
			SC1	SC2	SC3	SC4
Moderately ductile truss moment-resisting frames	3.5	1.6	NL	NL	50	30
Moderately ductile concentrically braced frames						
Tension-compression braces	3.0	1.3	NL	NL	40	40
Tension only braces	3.0	1.3	NL	NL	20	20
Limited ductility concentrically braced frames						
Tension-compression braces	2.0	1.3	NL	NL	60	60
Tension only braces	2.0	1.3	NL	NL	40	40
Ductile buckling-restrained braced frames	4.0	1.2	NL	NL	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL
Ductile plate walls	5.0	1.6	NL	NL	NL	NL
Moderately ductile plate walls	3.5	1.3	NL	NL	40	40
Limited ductility plate walls	2.0	1.3	NL	NL	60	60
Conventional construction of moment-resisting frames, braced frames or plate walls						
Assembly occupancies	1.5	1.3	NL	NL	15	15
Other *occupancies*	1.5	1.3	NL	NL	60	40
Other steel SFRSs not defined above	1.0	1.0	15	15	NP	NP
Concrete Structures Designed and Detailed According to CSA A23.3						
Ductile moment-resisting frames	4.0	1.7	NL	NL	NL	NL
Moderately ductile moment-resisting frames	2.5	1.4	NL	NL	60	40
Ductile coupled walls	4.0	1.7	NL	NL	NL	NL
Moderately ductile coupled walls	2.5	1.4	NL	NL	NL	60
Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL
Moderately ductile partially coupled walls	2.0	1.4	NL	NL	NL	60
Ductile shear walls	3.5	1.6	NL	NL	NL	NL
Moderately ductile shear walls	2.0	1.4	NL	NL	NL	60
Conventional construction						
Moment-resisting frames	1.5	1.3	NL	NL	20	10(5)(6)
Shear walls	1.5	1.3	NL	NL	40	30
Two-way slabs without beams	1.3	1.3	20	15	NP	NP
Tilt-up construction						
Moderately ductile walls and frames	2.0	1.3	30	25	25	25
Limited ductility walls and frames	1.5	1.3	30	25	20	20(7)
Conventional walls and frames	1.3	1.3	25	20	NP	NP
Other concrete SFRSs not listed above	1.0	1.0	15	15	NP	NP
Timber Structures Designed and Detailed According to CSA O86						
Shear walls						
Nailed shear walls: wood-based panel	3.0	1.7	NL	NL	30	20
Shear walls: wood-based and gypsum panels in combination	2.0	1.7	NL	NL	20	20
Moderately ductile cross-laminated timber shear walls: platform-type construction	2.0	1.5	30	30	30	20

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Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Cold-Formed Steel Structures Designed and Detailed According to CSA S136							
Shear walls							
Screw-connected shear walls – wood-based panels	2.5	1.7	20	20	20	20	20
Screw-connected shear walls – wood-based and gypsum panels in combination	1.5	1.7	20	20	20	20	20
Diagonal strap concentrically braced walls							
Limited ductility	1.9	1.3	20	20	20	20	20
Conventional construction	1.2	1.3	15	15	NP	NP	NP
Other cold-formed SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

Notes to Table 4.1.8.9.:

(1) See Article 4.1.8.10.

(2) NP = system is not permitted.

NL = system is permitted and not limited in height as an SFRS.

Numbers in this Table are maximum height limits above *grade*, in m.

Height may be limited in other Parts of the Code.

The most stringent requirement governs.

(3) Higher design force levels are prescribed in CSA S16 for some heights of *buildings*.

(4) See Note A-Table 4.1.8.9.

(5) Frames limited to a maximum of 2 *storeys*.

(6) Frames limited to a maximum of 3 *storeys*.

Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)			
			Seismic Category			
			SC1	SC2	SC3	SC4
Limited ductility cross-laminated timber shear walls: platform-type construction	1.0	1.3	30	30	30	20
Braced or moment-resisting frames with ductile connections						
Moderately ductile	2.0	1.5	NL	NL	20	20
Limited ductility	1.5	1.5	NL	NL	15	15
Other wood- or gypsum-based SFRSs not listed above	1.0	1.0	15	15	NP	NP
Masonry Structures Designed and Detailed According to CSA S304						
Ductile shear walls	3.0	1.5	NL	NL	60	40
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40
Conventional construction						
Shear walls	1.5	1.5	NL	60	30	15
Moment-resisting frames	1.5	1.5	NL	30	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP
Other masonry SFRSs not listed above	1.0	1.0	15	NP	NP	NP
Cold-Formed Steel Structures Designed and Detailed According to CSA S136						
Shear walls						
Screw-connected shear walls – wood-based panels	2.5	1.7	20	20	20	20
Screw-connected shear walls – wood-based and gypsum panels in combination	1.5	1.7	20	20	20	20
Diagonal strap concentrically braced walls						
Limited ductility	1.9	1.3	20	20	20	20
Conventional construction	1.2	1.3	15	15	NP	NP
Other cold-formed SFRSs not defined above	1.0	1.0	15	15	NP	NP

Notes to Table 4.1.8.9.:

(1) See Article 4.1.8.10.

(2) NP = system is not permitted.

NL = system is permitted and not limited in height as an SFRS.

Numbers in this Table are maximum height limits above *grade*, in m.

Height may be limited in other Parts of the Code.

The most stringent requirement governs.

(3) Higher design force levels are prescribed in CSA S16 for some heights of *buildings*.

(4) See Note A-Table 4.1.8.9.

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			(5) Frames are limited to a maximum of 2 *storeys*. (6) The maximum height limit is permitted to be increased to 15 m where $I_E S(1.0) \leq 0.3$. (7) Frames are limited to a maximum of 3 *storeys*.
4.1.8.10	1) Except as required by Clause (2)(b), structures with a Type 6 irregularity, Discontinuity in Capacity - Weak Storey, as described in Table 4.1.8.6., are not permitted unless $I_E F_a S_a(0.2)$ is less than 0.2 and the forces used for design of the SFRS are multiplied by $R_d R_o$. 2) *Post-disaster buildings* shall a) not have any irregularities conforming to Types 1, 3, 4, 5, 7 and 9 as described in Table 4.1.8.6., in cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, b) not have a Type 6 irregularity as described in Table 4.1.8.6., c) have an SFRS with an R_d of 2.0 or greater, and d) have no *storey* with a lateral stiffness that is less than that of the *storey* above it. 3) For *buildings* having fundamental lateral periods, T_a, of 1.0 s or greater, and where $I_E F_v S_a(1.0)$ is greater than 0.25, shear walls that are other than wood-based and form part of the SFRS shall be continuous from their top to the *foundation* and shall not have irregularities of Type 4 or 5 as described in Table 4.1.8.6. 4) For *buildings* constructed with more than 4 *storeys* of continuous wood construction and where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, timber SFRS consisting of shear walls with wood-based panels or of braced or moment-resisting frames as defined in Table 4.1.8.9. within the continuous wood construction shall not have Type 4 or Type 5 irregularities as described in Table 4.1.8.6. (See Note A-4.1.8.10.(4).) 5) The ratio, α, for a Type 9 irregularity as described in Table 4.1.8.6. shall be determined independently for each orthogonal direction using the following equation: where Q_G = gravity-induced lateral demand on the SFRS at the critical level of the yielding system, and Q_y = the resistance of the yielding mechanism required to resist the minimum earthquake loads, which need not be taken as less than R_o multiplied by	**4.1.8.10**	Additional System Restrictions 1) Except as required by Clause (2)(b), structures with a Type 6 irregularity, Discontinuity in Capacity - Weak Storey, as described in Table 4.1.8.6., are not permitted unless the Seismic Category is SC1 and the forces used for design of the SFRS are multiplied by $R_d R_o$. 2) *Post-disaster buildings* shall a) not have Type 1, 3, 4, 5, 7, 9 or 10 irregularities as described in Table 4.1.8.6., where the Seismic Category is SC3 or SC4, b) not have a Type 6 irregularity as described in Table 4.1.8.6., c) have an SFRS with an R_d of 2.0 or greater, d) where they are constructed with concrete or masonry shear walls, have no *storey* with a lateral stiffness that is less than that of the *storey* above it, and e) where they are constructed with other types of SFRS, have no *storey* for which the inter*storey* deflection under lateral earthquake forces divided by the inter*storey* height, h_s, is greater than that of the *storey* above it. 3) High Importance Category *buildings* shall a) not have Type 1, 3, 4, 5, 7, 9 or 10 irregularities as described in Table 4.1.8.6., where the Seismic Category is SC4, b) not have a Type 6 irregularity as described in Table 4.1.8.6., c) have an SFRS with an R_d of at least i) 2.0 where the Seismic Category is SC4, and ii) 1.5 otherwise, d) where they are constructed with concrete or masonry shear walls, have no *storey* with a lateral stiffness that is less than that of the *storey* above it, and e) where they are constructed with other types of SFRS, have no *storey* for which the inter*storey* deflection under lateral earthquake forces divided by the inter*storey* height, h_s, is greater than that of the *storey* above it. 4) Where the fundamental lateral period, T_a, is greater than or equal to 1.0 s and $I_E S(1.0)$ is greater than 0.25, shear walls that are other than wood-based and form part

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	the minimum lateral earthquake force as determined in Article 4.1.8.11. or 4.1.8.12., as appropriate. (See Note A-4.1.8.10.(5).) 6) For *buildings* with a Type 9 irregularity as described in Table 4.1.8.6. and where $I_EF_aS_a(0.2)$ is equal to or greater than 0.5, deflections determined in accordance with Article 4.1.8.13. shall be multiplied by 1.2. 7) Structures where the value of α, as determined in accordance with Sentence (5), exceeds twice the limits specified in Table 4.1.8.6. for a Type 9 irregularity, and where $I_EF_aS_a(0.2)$ is equal to or greater than 0.5 are not permitted unless determined to be acceptable based on non-linear dynamic analysis studies. (See Note A-4.1.8.10.(7).) New on 2020		of the SFRS shall be continuous from their top to the *foundation* and shall not have Type 4 or 5 irregularities as described in Table 4.1.8.6. 5) For *buildings* in Seismic Category SC3 or SC4 that are constructed with more than 4 *storeys* of continuous wood construction, timber SFRSs consisting of shear walls with wood-based panels or of braced or moment-resisting frames as defined in Table 4.1.8.9. within the continuous wood construction shall not have Type 4 or 5 irregularities as described in Table 4.1.8.6. (See Note A-4.1.8.10.(5) and (6).) 6) For *buildings* in Seismic Category SC3 or SC4 that are constructed with more than 4 *storeys* of continuous wood construction, timber SFRSs consisting of moderately ductile or limited ductility cross-laminated timber shear walls, platform-type construction, as defined in Table 4.1.8.9. within the continuous wood construction shall not have Type 4, 5, 6, 8, 9 or 10 irregularities as described in Table 4.1.8.6. (See Note A-4.1.8.10.(5) and (6).) 7) The ratio α for a Type 9 irregularity as described in Table 4.1.8.6. shall be determined independently for each orthogonal direction using the following equation: 9) For *buildings* where the value of α, as determined in accordance with Sentence (7), exceeds twice the appropriate limit specified in Table 4.1.8.6. for a Type 9 irregularity and where $I_ES(0.2)$ is equal to or greater than 0.5, a Non-linear Dynamic Analysis of the structure shall be carried out in accordance with Article 4.1.8.12. and the following criteria: a) the analysis shall account for the effects of the vertical response of the *building* mass, b) the analysis shall account for the effects of the vertical response of *building* components that undergo a vertical displacement when displaced laterally, c) the analysis shall use vertical ground motion time histories that are compatible with horizontal ground motion time histories scaled to the target response spectrum and that are applied concurrently with the horizontal ground motion time histories, d) the largest inter*storey* deflection at any level of the *building* as determined from the analysis shall not be greater than 60% of the appropriate limit stated in Sentence 4.1.8.13.(3), and

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			e) the results of an analysis using the ground motion time histories in Clause (c) multiplied by 1.5 shall satisfy the non-linear acceptance criteria. (See Note A-4.1.8.10.(9).) 10) The design of *buildings* in Seismic Category SC3 or SC4 with a Type 10 irregularity as described in Table 4.1.8.6. shall satisfy the following requirements: a) the structure shall be designed to resist the additional earthquake forces due to the vertical accelerations of the mass supported by inclined vertical members (see Note A-4.1.8.10.(10)(a)), and b) the effects of the horizontal and vertical movements of inclined vertical members, while undergoing earthquake-induced deformations, on the floor systems they support shall be considered in the design of the *building* and accounted for in the application of Sentence 4.1.8.3.(5).
4.1.8.11	c) for *buildings* located on a site other than Class F and having an SFRS with an R_d equal to or greater than 1.5, V need not be greater than the larger of	**4.1.8.11**	c) for *buildings* located on a site designated as other than X_F and having an SFRS with an R_d equal to or greater than 1.5, V need not be greater than the larger of

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Table 4.1.8.11.
Higher Mode Factor, M_v, and Base Overturning Moment Reduction Factor, J[1][2][3][4]
Forming Part of Sentence 4.1.8.11.(6)

S(0.2)/S(5.0)	M_v for $T_a \le 0.5$	M_v for $T_a = 1.0$	M_v for $T_a = 2.0$	M_v for $T_a \ge 5.0$	J for $T_a \le 0.5$	J for $T_a = 1.0$	J for $T_a = 2.0$	J for $T_a \ge 5.0$
Moment-Resisting Frames								
5	1	1	1	[5]	1	0.97	0.92	[5]
20	1	1	1	[5]	1	0.93	0.85	[5]
40	1	1	1	[5]	1	0.87	0.78	[5]
65	1	1	1.03	[5]	1	0.80	0.70	[5]
Coupled Walls[6]								
5	1	1	1	1[7]	1	0.97	0.92	0.80[8]
20	1	1	1	1.08[7]	1	0.93	0.85	0.65[8]
40	1	1	1	1.30[7]	1	0.87	0.78	0.53[8]
65	1	1	1.03	1.49[7]	1	0.80	0.70	0.46[8]
Braced Frames								
5	1	1	1	[5]	1	0.95	0.89	[5]
20	1	1	1	[5]	1	0.85	0.78	[5]
40	1	1	1	[5]	1	0.79	0.70	[5]
65	1	1.04	1.07	[5]	1	0.71	0.66	[5]
Walls, Wall Frame Systems								
5	1	1	1	1.25[7]	1	0.97	0.85	0.55[8]
20	1	1	1.18	2.30[7]	1	0.80	0.60	0.35[8]
40	1	1.19	1.75	3.70[7]	1	0.63	0.46	0.28[8]
65	1	1.55	2.25	4.65[7]	1	0.51	0.39	0.23[8]
Other Systems								
5	1	1	1	[5]	1	0.97	0.85	[5]
20	1	1	1.18	[5]	1	0.80	0.60	[5]
40	1	1.19	1.75	[5]	1	0.63	0.46	[5]
65	1	1.55	2.25	[5]	1	0.51	0.39	[5]

Notes to Table 4.1.8.11.:

(1) For intermediate values of the spectral ratio S(0.2)/S(5.0), M_v and J shall be obtained by linear interpolation.

(2) For intermediate values of the fundamental lateral period, T_a, $S(T_a)M_v$ shall be obtained by linear interpolation using the values of M_v obtained in accordance with Note (1).

(3) For intermediate values of the fundamental lateral period, T_a, J shall be obtained by linear interpolation using the values of J obtained in accordance with Note (1).

11) Torsional effects shall be accounted for as follows:

a) for a *building* with $B \le 1.7$ or where $I_E F_a S_a(0.2)$ is less than 0.35, by applying

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Table 4.1.8.11.
Higher Mode Factor, M_v, and Base Overturning Moment Reduction Factor, J[1][2][3][4]
Forming Part of Sentence 4.1.8.11.(6)

S(0.2)/S(5.0)	M_v for $T_a \le 0.5$	M_v for $T_a = 1.0$	M_v for $T_a = 2.0$	M_v for $T_a \ge 5.0$	J for $T_a \le 0.5$	J for $T_a = 1.0$	J for $T_a = 2.0$	J for $T_a \ge 5.0$
Moment-Resisting Frames								
5	1	1	1	[5]	1	1	0.95	[5]
20	1	1	1	[5]	1	0.97	0.88	[5]
40	1	1	1	[5]	1	0.90	0.79	[5]
70	1	1	1	[5]	0.98	0.88	0.70	[5]
Coupled Walls[6]								
5	1	1	1	1[7]	1	1	0.95	0.80[8]
20	1	1	1	1.09[7]	1	0.97	0.88	0.66[8]
40	1	1	1	1.33[7]	1	0.90	0.79	0.52[8]
70	1	1	1	1.90[7]	0.98	0.88	0.70	0.40[8]
Braced Frames								
5	1	1	1	[5]	1	0.98	0.93	[5]
20	1	1	1	[5]	1	0.91	0.80	[5]
40	1	1	1	[5]	0.91	0.82	0.72	[5]
70	1	1	1.19	[5]	0.91	0.77	0.61	[5]
Walls, Wall-Frame Systems								
5	1	1	1	1.30[7]	1	1	0.85	0.59[8]
20	1	1	1.18	2.50[7]	1	0.80	0.60	0.35[8]
40	1	1.25	1.85	4.10[7]	0.80	0.59	0.42	0.23[8]
70	1	1.25	2.30	6.40[7]	0.80	0.56	0.30	0.18[8]
Other Systems								
5	1	1	1	[5]	1	1	0.85	[5]
20	1	1	1.18	[5]	1	0.80	0.60	[5]
40	1	1.25	1.85	[5]	0.80	0.59	0.44	[5]
70	1	1.37	2.30	[5]	0.80	0.56	0.30	[5]

Notes to Table 4.1.8.11.:

(1) For intermediate values of the spectral ratio S(0.2)/S(5.0), M_v and J shall be obtained by linear interpolation. For spectral ratios less than 5, M_v and J shall be obtained by linear interpolation with their values at a spectral ratio of 0 taken as equal to 1. For spectral ratios greater than 70, M_v and J shall be taken as equal to their values at a spectral ratio of 70.

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	torsional moments about a vertical axis at each level throughout the *building*, derived for each of the following load cases considered separately: i) $T_x = F_x(e_x + 0.10\ D_{nx})$, and ii) $T_x = F_x(e_x - 0.10\ D_{nx})$ where F_x is the lateral force at each level determined according to Sentence (7) and where each element of the *building* is designed for the most severe effect of the above load cases, or b) for a *building* with B > 1.7, in cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, by a Dynamic Analysis Procedure as specified in Article 4.1.8.12. 6) For structures located on sites other than Class F that have an SFRS with R_d equal to or greater than 1.5, the elastic base shear obtained from a Linear Dynamic Analysis may be multiplied by the larger of the following factors to obtain the design elastic base shear, V_{ed}:		(2) For intermediate values of the fundamental lateral period, T_a, in cases where $S(T_a)$ is obtained by log–log interpolation, M_v shall be obtained by linear interpolation using the values of M_v obtained in accordance with Note (1). In cases where $S(T_a)$ is obtained by linear interpolation, the product $S(T_a)M_v$ shall be obtained by linear interpolation using the values of M_v obtained in accordance with Note (1). (3) For intermediate values of the fundamental lateral period, T_a, J shall be obtained by linear interpolation using the values of J obtained in accordance with Note (1). 11) Torsional effects shall be accounted for as follows: a) for a *building* with B ≤ 1.7 or in Seismic Category SC1 or SC2, by applying torsional moments about a vertical axis at each level throughout the *building*, derived for each of the following load cases considered separately: i) $T_x = F_x(e_x + 0.10D_{nx})$, and ii) $T_x = F_x(e_x - 0.10D_{nx})$ where F_x is determined in accordance with Sentence (7) and where each element of the *building* is designed for the most severe effect of the above load cases, or b) for a *building* with B > 1.7 in Seismic Category SC3 or SC4, by a Dynamic Analysis Procedure as specified in Article 4.1.8.12. 6) For *buildings* located on a site designated as other than X_F that have an SFRS with R_d equal to or greater than 1.5, the elastic base shear, V_e, obtained from a Linear Dynamic Analysismay bemultiplied by the larger of the following factors to obtain V_{ed}:
4.1.8.15	5) In cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, the elements supporting any discontinuous wall, column or braced frame shall be designed for the lateral load capacity of the components of the SFRS they support. (See Note A-4.1.8.15.(5).)	**4.1.8.15**	5) Where the Seismic Category is SC3 or SC4, the elements supporting any discontinuous wall, column or braced frame shall be designed for the lateral load capacity of the components of the SFRS they support. (See Note A-4.1.8.15.(5).)
4.1.8.16	6) In cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, the following requirements shall be satisfied: a) *piles* or *pile* caps, drilled piers, and *caissons* shall be interconnected by continuous ties in not less than two directions (see Note A-4.1.8.16.(6)(a)), b) *piles*, drilled piers, and *caissons* shall be embedded a minimum of 100 mm		6) Where the Seismic Category is SC3 or SC4, the following requirements shall be satisfied: a) *piles* or *pile* caps, drilled piers, and *caissons* shall be interconnected by continuous ties in not less than two directions (see Note A-4.1.8.16.(6)(a)), b) *piles*, drilled piers, and *caissons* shall be embedded a minimum of 100 mm

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	into the *pile* cap or structure, and c) *piles*, drilled piers, and *caissons*, other than wood *piles*, shall be connected to the *pile* cap or structure for a minimum tension force equal to 0.15 times the factored compression load on the *pile* 7) At sites where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, *basement* walls shall be designed to resist earthquake lateral pressures from backfill or natural ground. (See Note A-4.1.8.16.(7).) 8) At sites where $I_E F_a S_a(0.2)$ is greater than 0.75, the following requirements shall be satisfied: a) *piles*, drilled piers, or *caissons* shall be designed and detailed to accommodate cyclic inelastic behaviour when the design moment in the element due to earthquake effects is greater than 75% of its moment capacity (see Note A-4.1.8.16.(8)(a)), and b) spread footings founded on *soil* defined as Site Class E or F shall be interconnected by continuous ties in not less than two directions.		into the *pile* cap or structure, and c) *piles*, drilled piers, and *caissons*, other than wood *piles*, shall be connected to the *pile* cap or structure for a minimum tension force equal to 0.15 times the factored compression load on the *pile*. 7) Where the Seismic Category is SC3 or SC4, *basement* walls shall be designed to resist earthquake lateral pressures from backfill or natural ground. (See Note A-4.1.8.16.(7).) tisfied: a) *piles*, drilled piers, or *caissons* shall be designed and detailed to accommodate cyclic inelastic behaviour when the design moment in the element due to earthquake effects is greater than 75% of its moment capacity (see Note A-4.1.8.16.(8)(a)), and b) spread footings founded on *soil* designated as X_V, where V_{s30} is less than or equal to 180 m/s, X_E or X_F shall be interconnected by continuous ties in not less than two directions.
4.1.8.18	1) Except as provided in Sentences (2), (7) and (16), elements and components of *buildings* described in Table 4.1.8.18. and their connections to the structure shall be designed to accommodate the *building* deflections calculated in accordance with Article 4.1.8.13. and the element or component deflections calculated in accordance with Sentence (9), and shall be designed for a lateral force, V_p, distributed according to the distribution of mass: $V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$ where F_a = as defined in Sentence 4.1.8.4.(7), $S_a(0.2)$ = spectral response acceleration value at 0.2 s, as defined in Sentence 4.1.8.4.(1), I_E = importance factor for the *building*, as defined in Article 4.1.8.5., $S_p = C_p A_r A_x / R_p$ (the maximum value of S_p shall be taken as 4.0 and the minimum value of S_p shall be taken as 0.7), where C_p = element or component factor from Table 4.1.8.18., A_r = element or component force amplification factor from Table 4.1.8.18., A_x = height factor $(1 + 2\, h_x / h_n)$, R_p = element or component response modification factor from Table 4.1.8.18., and W_p = weight of the component or element.	**4.1.8.18**	1) Except as provided in Sentences (2), (7) and (16), elements and components of *buildings* described in Table 4.1.8.18. and their connections to the structure shall be designed to accommodate the *building* deflections calculated in accordance with Article 4.1.8.13. and the element or component deflections calculated in accordance with Sentence (9), and shall be designed for a specified lateral earthquake force, V_p, distributed according to the distribution of mass: $V_p = 0.3 S(0.2) I_E S_p W_p$ where $S(0.2)$ = design spectral acceleration value at a period of 0.2 s, as defined in Sentence 4.1.8.4.(6), I_E = earthquake importance factor for the *building*, as defined in Article 4.1.8.5., $S_p = C_p A_r A_x / R_p$ (the maximum value of S_p shall be taken as 4.0 and the minimum value of S_p shall be taken as 0.7), where C_p = element or component factor from Table 4.1.8.18., A_r = element or component force amplification factor from Table 4.1.8.18., A_x = height factor $(1 + 2h_x/h_n)$, R_p = element or component response modification factor from Table 4.1.8.18., and W_p = weight of the component or element.

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Table 4.1.8.18.
Elements of Structures and Non-structural Components and Equipment[1]
Forming Part of Sentences 4.1.8.18.(1), (2), (3), (6) and (7)

Category	Part or Portion of *Building*	C_p	A_r	R_p
1	All exterior and interior walls except those in Category 2 or 3	1.00	1.00	2.50
2	Cantilever parapet and other cantilever walls except retaining walls	1.00	2.50	2.50
3	Exterior and interior ornamentations and appendages	1.00	2.50	2.50
4	Floors and roofs acting as diaphragms[2]	-	-	-
5	Towers, *chimneys*, smokestacks and penthouses when connected to or forming part of a *building*	1.00	2.50	2.50
6	Horizontally cantilevered floors, balconies, beams, etc.	1.00	1.00	2.50
7	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	1.00	1.00	2.50
8	Masonry veneer connections	1.00	1.00	1.50
9	Access floors	1.00	1.00	2.50
10	Masonry or concrete fences more than 1.8 m tall	1.00	1.00	2.50
11	Machinery, fixtures, equipment and tanks (including contents)			
	that are rigid and rigidly connected	1.00	1.00	1.25
	that are flexible or flexibly connected	1.00	2.50	2.50
12	Machinery, fixtures, equipment and tanks (including contents) containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids			
	that are rigid and rigidly connected	1.50	1.00	1.25
	that are flexible or flexibly connected	1.50	2.50	2.50
13	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building*	0.70	1.00	2.50
14	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building* containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids	1.00	1.00	2.50
15	Pipes, ducts (including contents)	1.00	1.00	3.00

Table 4.1.8.18. (Continued)

Category	Part or Portion of *Building*	C_p	A_r	R_p
16	Pipes, ducts (including contents) containing toxic or explosive materials	1.50	1.00	3.00
17	Electrical cable trays, bus ducts, conduits	1.00	2.50	5.00
18	Rigid components with ductile material and connections	1.00	1.00	2.50
19	Rigid components with non-ductile material or connections	1.00	1.00	1.00
20	Flexible components with ductile material and connections	1.00	2.50	2.50
21	Flexible components with non-ductile material or connections	1.00	2.50	1.00
22	Elevators and escalators[3]			
	machinery and equipment	as per category 11		
	elevator rails	1.00	1.00	2.50
23	Floor-mounted steel pallet storage racks[4]	1.00	2.50	2.50
24	Floor-mounted steel pallet storage racks on which are stored toxic or explosive materials or materials having a flash point below 38°C[4].	1.50	2.50	2.50

2) For *buildings* other than *post-disaster buildings*, seismically isolated *buildings*, and *buildings* with supplemental energy dissipation systems, where $I_EF_aS_a(0.2)$ is less than 0.35, the requirements of Sentence (1) need not apply to Categories 6 through 22 of

Table 4.1.8.18.
Elements of Structures and Non-structural Components and Equipment[1]
Forming Part of Sentences 4.1.8.18.(1) to (3), (6), (7) and (16), and Clauses 4.1.8.23.(2)(c) and (3)(c)

Category	Part or Portion of *Building*	C_p	A_r	R_p
Architectural and Structural Components				
1	All exterior and interior walls, and cladding panels, except those in Category 2 or 3	1.00	1.00	2.50
2	Cantilever parapet and other cantilever walls, including cantilever cladding panels, except retaining walls	1.00	2.50	2.50
3	Exterior and interior ornamentations and appendages	1.00	2.50	2.50
4	Floors and roofs acting as diaphragms[2]	–	–	–
5	Towers, *chimneys*, smokestacks and penthouses when connected to or forming part of a *building*	1.00	2.50	2.50
6	Horizontally cantilevered floors, balconies, beams, etc.	1.00	1.00	2.50
7	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	1.00	1.00	2.50
8	Masonry veneer connections	1.00	1.00	1.50
9	Access floors	1.00	1.00	2.50
10	Masonry or concrete fences more than 1.8 m tall	1.00	1.00	2.50
Mechanical and Electrical Components				
11	Machinery, fixtures, equipment and tanks (including contents)			
	that are rigid and rigidly connected	1.00	1.00	1.25
	that are flexible or flexibly connected	1.00	2.50	2.50

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Table 4.1.8.18.

7) Connections to the structure of elements and components listed in Table 4.1.8.18. shall be designed to support the component or element for gravity loads, shall conform to the requirements of Sentence (1), and shall also satisfy these additional requirements:

a) friction due to gravity loads shall not be considered to provide resistance to seismic forces,

b) R_p for non-ductile connections, such as adhesives or power-actuated fasteners, shall be taken as 1.0,

c) R_p for anchorage using shallow expansion, chemical, epoxy or cast-in-place anchors shall be 1.5, where shallow anchors are those with a ratio of embedment length to diameter of less than 8,

d) power-actuated fasteners and drop-in anchors shall not be used for tension loads,

e) connections for non-structural elements or components of Category 1, 2 or 3 of Table 4.1.8.18. attached to the side of a *building* and above the first level above *grade* shall satisfy the following requirements:

i) for connections where the body of the connection is ductile, the body shall be designed for values of C_p, A_r and R_p given in Table 4.1.8.18., and all of the other parts of the connection, such as anchors, welds, bolts and inserts, shall be capable of developing 2.0 times the nominal yield resistance of the body of the connection, and

ii) connections where the body of the connection is not ductile shall be designed for values of $C_p = 2.0$, $R_p = 1.0$ and A_r given in Table 4.1.8.18., and

f) a ductile connection is one where the body of the connection is capable of dissipating energy through cyclic inelastic behaviour.

15) Glass need not comply with Sentence (14), provided at least one of the following conditions is met:

a) $I_E F_a S_a(0.2) < 0.35$,

b) the glass has sufficient clearance from its frame such that $D_{clear} \geq 1.25\ D_p$ calculated as follows:

Table 4.1.8.18. (Continued)

Category	Part or Portion of *Building*	C_p	A_r	R_p
12	Machinery, fixtures, equipment and tanks (including contents) containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids			
	that are rigid and rigidly connected	1.50	1.00	1.25
	that are flexible or flexibly connected	1.50	2.50	2.50
13	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building*	0.70	1.00	2.50
14	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building* containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids	1.00	1.00	2.50
15	Pipes, ducts (including contents)	1.00	1.00	3.00
16	Pipes, ducts (including contents) containing toxic or explosive materials	1.50	1.00	3.00
17	Electrical cable trays, bus ducts, conduits	1.00	2.50	5.00
Other System Components				
18	Rigid components with ductile material and connections	1.00	1.00	2.50
19	Rigid components with non-ductile material or connections	1.00	1.00	1.00
20	Flexible components with ductile material and connections	1.00	2.50	2.50
21	Flexible components with non-ductile material or connections	1.00	2.50	1.00
22	Elevators and escalators(3)			
	machinery and equipment	as per Category 11		
	elevator rails	1.00	1.00	2.50
23	Floor-mounted steel pallet storage racks(4)	1.00	2.50	2.50
24	Floor-mounted steel pallet storage racks on which are stored toxic or explosive materials or materials having a flash point below 38°C(4)	1.50	2.50	2.50

2) For *buildings* in Seismic Category SC1 or SC2, other than *post-disaster buildings*, seismically isolated *buildings*, and *buildings* with supplemental energy dissipation systems, the requirements of Sentence (1) need not apply to Categories 6 through 22 of Table 4.1.8.18.

7) Connections to the structure of elements and components listed in Table 4.1.8.18. shall be designed to support the component or element for gravity loads, shall conform to the requirements of Sentence (1), and shall also satisfy these additional requirements:

a) except as provided in Sentence (17), friction due to gravity loads shall not be considered to provide resistance to earthquake forces,

b) R_p for non-ductile connections, such as adhesives or power-actuated fasteners, shall be taken as 1.0,

c) R_p for shallow post-installed mechanical, post-installed adhesive, and cast-in-place anchors in concrete shall be 1.5, where shallow anchors are

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	16) For structures with supplemental energy dissipation, the following criteria shall apply: a) the value of $S_a(0.2)$ used in Sentence (1) shall be determined from the mean 5% damped floor spectral acceleration values at 0.2 s by averaging the individual 5% damped floor spectra at the base of the structure determined using Non-Linear Dynamic Analysis, and b) the value of F_a used in Sentence (1) shall be 1.		those with a ratio of embedment length to diameter of less than 8, d) post-installed mechanical, drop-in and adhesive anchors in concrete shall be pre-qualified for seismic applications by cyclic load testing in accordance with i) CSA A23.3, "Design of concrete structures," and ii) ACI 355.2, "Qualification of Post-Installed Mechanical Anchors in Concrete (ACI 355.2-19) and Commentary," or ACI 355.4, "Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4-19) and Commentary," as applicable, e) post-installed mechanical and adhesive anchors in masonry and post-installed mechanical anchors in structural steel shall be pre-qualified for seismic applications by cyclic tension load testing (see Note A-4.1.8.18.(7)(e)), f) power-actuated fasteners shall not be used for cyclic tension loads, g) connections for non-structural elements or components of Category 1, 2 or 3 of Table 4.1.8.18. attached to the side of a *building* and above the first level above *grade* shall satisfy the following requirements: i) for connections where the body of the connection is ductile, the body shall be designed for values of C_p, A_r and R_p given in Table 4.1.8.18., and all of the other parts of the connection, such as anchors, welds, bolts and inserts, shall be capable of developing 2.0 times the nominal yield resistance of the body of the connection, and ii) connections where the body of the connection is not ductile shall be designed for values of $C_p = 2.0$, $R_p = 1.0$ and A_r given in Table 4.1.8.18., and h) a ductile connection is one where the body of the connection is capable of dissipating energy through cyclic inelastic behaviour. 15) Glass need not comply with Sentence (14), provided at least one of the following conditions is met: a) the Seismic Category is SC1 or SC2, b) the glass has sufficient clearance from its frame such that $D_{clear} \geq 1.25D_p$ calculated as follows:

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			16) For structures with supplemental energy dissipation, elements and components of *buildings* described in Table 4.1.8.18. and their connections to the structure shall be designed for a specified lateral earthquake force, V_p, determined at each floor level as follows: where S_{sed} = peak spectral acceleration, $S_a(T,X)$, in the period range of T = 0 s to T = 0.5 s determined from the mean 5%-damped floor spectral acceleration values by averaging the individual 5%-damped floor response spectra at the centroid of the floor area at that floor level determined using Non-linear Dynamic Analysis, and I_E, C_p, A_r, R_p,W_p = as defined in Sentence (1). (See Note A-4.1.8.18.(16).) 17) For a ballasted array of interconnected solar panels mounted on a roof, where $I_ES(0.2)$ is less than or equal to 1.0, friction due to gravity loads is permitted to be considered to provide resistance to seismic forces, provided a) the roof is not normally occupied, b) the roof is surrounded by a parapet extending from the roof surface to not less than the greater of i) 150 mm above the centre of mass of the array, and ii) 400 mm above the roof surface, c) the height of the centre of mass of the array above the roof surface is less than the lesser of i) 900 mm, and ii) one half of the smallest plan dimension of the supporting base of the array, d) the roof slope at the location of the array is less than or equal to 3°, e) the factored friction resistance calculated using the kinetic friction coefficient determined in accordance with Sentence (18) and a resistance factor of 0.7 is greater than or equal to the specified lateral earthquake force, V_p, on the array determined in accordance with Sentence (1) using values of A_r = 1.0, A_x = 3.0, C_p = 1.0, and R_p = 1.25, f) the minimum clearance between the array and other arrays or fixed objects is the greater of i) 225 mm, and ii) 1 500$(I_ES(0.2) - 0.4)^2$, in mm, and

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			g) the minimum clearance between the array and the roof parapet is the greater of i) 450 mm, and ii) 3 000($I_E S(0.2) - 0.4)^2$, in mm. 18) For the purpose of Clause (17)(e), the kinetic friction coefficient shall be determined in accordance with ASTM G115, "Standard Guide for Measuring and Reporting Friction Coefficients," through experimental testing that a) is carried out by an accredited laboratory on a full-scale array or a prototype of the array, b) models the interface between the supporting base of the array and the roof surface, and c) accounts for the adverse effects of anticipated climatic conditions on the friction resistance. (See Note A-4.1.8.18.(18).)
4.1.8.19	4) The ground motion histories used in Sentence (3) shall be a) appropriately selected and scaled following good engineering practice, b) compatible with i) a response spectrum derived from the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(9) for ground conditions of Site Classes A, B and C, and ii) a 5% damped response spectrum based on a site-specific evaluation for ground conditions of Site Classes D, E and F, and c) amplitude-scaled in an appropriate manner over the period range of 0.2 T_1 to 1.5 T_1, where T_1 is the period of the isolated structure determined using the post-yield stiffness of the isolation system in the horizontal direction under consideration, or the period specified in Sentence 4.1.8.20.(1) if the post-yield stiffness of the isolation system is not well defined. (See Note A-4.1.8.19.(4) and 4.1.8.21.(5).)	**4.1.8.19**	4) The ground motion time histories used in Sentence (3) shall be a) appropriately selected and scaled following good engineering practice, b) compatible with i) a response spectrum derived from the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(6) for site designations X_V, where V_{s30} is greater than 360 m/s, X_A, X_B and X_C, and ii) a 5%-damped response spectrum based on a site-specific evaluation for site designations X_V, where V_{s30} is less than or equal to 360 m/s, X_D, X_E and X_F, and c) amplitude-scaled in an appropriate manner over the period range of $0.2T_1$ to $1.5T_1$, where T_1 is the period of the isolated structure determined using the post-yield stiffness of the isolation system in the horizontal direction under consideration, or the period specified in Sentence 4.1.8.20.(1) if the post-yield stiffness of the isolation system is not well defined. (See Note A-4.1.8.19.(4) and 4.1.8.21.(5).)
4.1.8.23	New in 2020	**4.1.8.23**	Additional Performance Requirements for Post-disaster Buildings, High Importance Category Buildings, and a Subset of Normal Importance Category Buildings 1) *Buildings* designed in accordance with Articles 4.1.8.19. to 4.1.8.22. need not

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			comply with this Article. 2) The design of *post-disaster buildings* in Seismic Category SC2, SC3 or SC4 shall be verified using 5%-damped spectral acceleration values based on a 5% probability of exceedance in 50 years and shall satisfy the following requirements: a) the *building* shall be shown to behave elastically for a specified lateral earthquake force, V, determined in accordance with Sentence 4.1.8.11.(2) using $I_E = 1.0$ and $R_dR_o = 1.3$, b) the largest inter*storey* deflection at any level of the *building*, as determined in accordance with Sentence 4.1.8.13.(2) using $I_E = 1.0$ and $R_dR_o = 1.0$, shall not exceed $0.005h_s$, and c) the connections of elements and components of the *building* described in Table 4.1.8.18. with $R_p > 1.5$ shall be shown to behave elastically for a specified lateral earthquake force, V_p, determined in accordance with Sentence 4.1.8.18.(1) using $R_p = 1.5$. 3) The design ofHigh Importance Category *buildings* in Seismic Category SC3 or SC4 shall be verified using 5%-damped spectral acceleration values based on a 10% probability of exceedance in 50 years and shall satisfy the following requirements: a) the *building* shall be shown to behave elastically for a specified lateral earthquake force, V, determined in accordance with Sentence 4.1.8.11.(2) using $I_E = 1.0$ and $R_dR_o = 1.3$, b) the largest inter*storey* deflection at any level of the *building*, as determined in accordance with Sentence 4.1.8.13.(2) using $I_E = 1.0$ and $R_dR_o = 1.0$, shall not exceed $0.005h_s$, and c) the connections of elements and components of the *building* described in Table 4.1.8.18. with $R_p > 1.3$ shall be shown to behave elastically for a specified lateral earthquake force, V_p, determined in accordance with Sentence 4.1.8.18.(1) using $R_p = 1.3$. 4) For Normal Importance Category *buildings* in Seismic Category SC4 with a height above *grade* of more than 30 m, the structural framing elements not considered to be part of the SFRS shall be designed to behave elastically for a specified lateral earthquake force, V, determined in accordance with Sentence 4.1.8.11.(2) using spectral acceleration values based on a 10% probability of exceedance in 50 years and $R_dR_o = 1.3$. 5) For the purposes of applying Sentences (2) to (4), torsional moments due to accidental eccentricities need not be considered if B, as determined in accordance with Sentence 4.1.8.11.(10), does not exceed 1.7.

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			6) For the purposes of applying Sentences (2) to (4), elements of the SFRS and structural framing elements not considered to be part of the SFRS, when included in the analysis, shall be modeled in accordance with Sentence 4.1.8.3.(8) using elastic properties. 7) All other requirements of Articles 4.1.8.2. to 4.1.8.18. shall be satisfied in meeting the additional requirements of this Article.
4.2.4.2	Drawings 1) Drawings associated with *foundations* and *excavations* shall conform to the appropriate requirements of Section 2.2. of Division C. (See Article 2.2.4.6. of Division C.)	**4.2.3.2**	Preservation Treatment of Wood 1) Wood exposed to *soil*, *rock* or air above the lowest anticipated *groundwater* table shall be treated with preservative in conformance with CAN/CSA-O80 Series, "Wood preservation," and the requirements of the appropriate standard as follows: a) CAN/CSA-O80.1, "Specification of treated wood," b) CAN/CSA-O80.2, "Processing and treatment," or c) CAN/CSA-O80.3, "Preservative formulations." 2) Wood treated as required in Sentence (1) shall be cared for as provided in Clause 4 of CAN/CSA-O80.0, "General requirements for wood preservation."
4.4.1	Air-Supported Structures	**4.4.1**	Air-, Cable- and Frame-Supported Membrane Structures
4.4.1.1	Design Basis for Air-Supported Structures 1) The structural design of *air-supported structures* shall conform to CSA S367, "Air-, Cable-, and Frame-Supported Membrane Structures," using the loads stipulated in Section 4.1., in accordance with limit states design in Subsection 4.1.3.	**4.4.1.1**	Design Basis for Air-, Cable- and Frame-Supported Membrane Structures 1) The structural design of air-, cable- and frame-supported membrane structures shall conform to CSA S367, "Air-, cable-, and frame-supported membrane structures," using the loads stipulated in Section 4.1., in accordance with limit states design in Subsection 4.1.3.
4.4.2.1	Design Basis for Parking Structures and Repair Garages 1) Parking structures and *repair garages* shall be designed in conformance with CSA S413, "Parking Structures." (See Note A-4.4.2.1.(1).)	**4.4.2.1**	Design Basis for Storage Garages and Repair Garages 1) *Storage garages* and *repair garages*, including associated ramps and pedestrian areas, shall be designed in conformance with the performance requirements of CSA S413, "Parking structures." (See Note A-4.4.2.1.(1).)
4.4.3	New in 2020	**4.4.3**	Storage Racks

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Code Ref.	Part 4	Code Ref.	Part 4
4.4.3.1	New in 2020	**4.4.3.1**	Design Basis for Storage Racks 1) Storage racks, including anchorage of racks, shall be designed for loads in accordance with this Part. (See Note A-4.1.8.18.(13) and 4.4.3.1.(1).)
4.4.8.5	**4.1.8.5. Importance Factor**		**4.1.8.5. Importance Factor and Seismic Category**
4.4.1.1	**4.4.1.1. Design Basis for Air-Supported Structures** (1) [F20-OS2.1] [F80-OS2.3] [F20-OP2.1] [F22-OP2.4] [F80-OP2.3] [F22-OH4]	**4.4.1.1**	**4.4.1.1. Design Basis for Air-, Cable- and Frame-Supported Membrane Structures** (1) [F20-OS2.1] [F80-OS2.3] [F20-OP2.1] [F22-OP2.4] [F80-OP2.3] [F22-OH4]
4.4.2.1	**4.4.2.1. Design Basis for Parking Structures and Repair Garages** (1) [F21,F61,F80-OS2.3] [F21,F61,F80-OP2.3,OP2.4] [F21,F61,F80-OH4]	**4.4.2.1**	**4.4.2.1. Design Basis for Storage Garages and Repair Garages** (1) [F21,F61,F80-OS2.3] [F21,F61,F80-OP2.3,OP2.4] [F21,F61,F80-OH4]
Notes to Part 4 Structural design?		**Notes to Part 4 Structural design?**	