Safety Codes Council

PART 4

CODE UPDATE INFORMATION

NBC 2019 AE Div B

November 2019

Safety Codes Council
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www.safetycodes.ab.ca

~~Red strikethrough~~ = deleted text

Blue underline = New text

Review this document in conjunction with the National Building Code-2019 Alberta Edition.

<table>
<tr><th>ABC 2014</th><th>NBC(AE) 2019</th><th>Comments</th></tr>
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4.1.3.2. Strength and Stability

1) 2) 3) 4) 5) 6)

7) The companion-load factor 0.5 for *live loads* L in Table 4.1.3.2.A. and LXC in Table 4.1.3.2.B. shall be increased to 1.0 for storage areas, and equipment areas and *service rooms* referred to in Table 4.1.5.3.

Table 4.1.3.2.A.

Load Combinations Without Crane Loads for Ultimate Limit States Forming Part of Sentences 4.1.3.2.(2) and (5) to (10)

<table>
<tr><th rowspan="2">Case</th><th colspan="2">Load Combination(1)</th></tr>
<tr><th>Principal Loads</th><th>Companion Loads</th></tr>
<tr><td>1</td><td>$1.4\mathbf{D}^{(2)}$</td><td>—</td></tr>
<tr><td>2</td><td>$(1.25\mathbf{D}^{(3)} \text{ or } 0.9\mathbf{D}^{(4)}) + 1.5\mathbf{L}^{(5)}$</td><td>$0.5\mathbf{S}^{(6)} \text{ or } 0.4\mathbf{W}$</td></tr>
<tr><td>3</td><td>$(1.25\mathbf{D}^{(3)} \text{ or } 0.9\mathbf{D}^{(4)}) + 1.5\mathbf{S}$</td><td>$0.5\ \text{L}^{(6)(7)} \text{ or } 0.4\mathbf{W}$</td></tr>
<tr><td>4</td><td>$(1.25\mathbf{D}^{(3)} \text{ or } 0.9\text{D}(4)) + 1.4\text{W}$</td><td>$0.5\mathbf{L}^{(7)} \text{ or } 0.5\mathbf{S}$</td></tr>
<tr><td>5</td><td>$1.0\mathbf{D}^{(4)} + 1.0\mathbf{E}^{(8)}$</td><td>$0.5\mathbf{L}^{(6)(7)} + 0.25\mathbf{S}^{(6)}$</td></tr>
</table>

Table 4.1.3.2.B.

Load Combinations With Crane Loads for Ultimate Limit States
Forming Part of Sentences 4.1.3.2.(2), (5) to (8), and (10)

<table>
<tr><th rowspan="2">Case</th><th colspan="2">Load Combination(1)</th></tr>
<tr><th>Principal Loads</th><th>Companion Loads</th></tr>
<tr><td>1</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + (1.5\mathbf{C} + 1.0\mathbf{L}_{XC})$</td><td>$1.0\mathbf{S}^{(4)} \text{ or } 0.4\mathbf{W}$</td></tr>
<tr><td>2</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + (1.5\mathbf{L}_{XC}^{(5)} + 1.0\mathbf{C})$</td><td>$0.5\text{-}\mathbf{S}^{(4)} \text{ or } 0.4\mathbf{W}$</td></tr>
<tr><td>3</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + 1.5\mathbf{S}$</td><td>$1.0\mathbf{C} + 0.5\text{-}\mathbf{L}_{XC}^{(4)(6)})$</td></tr>
<tr><td>4</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + 1.4\mathbf{W}$</td><td>$(1.0\mathbf{C}^{(7)} + 0.5\mathbf{L}_{XC}^{(4)(6)})$</td></tr>
<tr><td>5</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + \mathbf{C}_7$</td><td>—</td></tr>
<tr><td>6</td><td>$1.0\mathbf{D}^{(3)} + 1.0\mathbf{E}^{(8)}$</td><td>$1.0\mathbf{C}_d + 0.5\mathbf{L}_{XC}^{(4)(6)} + 0.25\mathbf{S}^{(4)}$</td></tr>
</table>

8) 9) 10) 11) 12)

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4.1.3.2. Strength and Stability

1) 2) 3) 4) 5) 6)

7) The companion-load factor ~~0.5~~ for *live loads* L in Table 4.1.3.2.-A. and LXC in Table 4.1.3.2.-B. shall be increased ~~to 1.0~~ by 0.5 for storage areas, and equipment areas and *service rooms* referred to in Table 4.1.5.3.

Table 4.1.3.2.-A~~.~~

Load Combinations Without Crane Loads for Ultimate Limit States Forming Part of Sentences 4.1.3.2.(2) and (5) to (10)

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Table 4.1.3.2.-B~~.~~

Load Combinations With Crane Loads for Ultimate Limit States Forming Part of Sentences 4.1.3.2.(2), (5) to (8), and (10)

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<tr><th rowspan="2">Case</th><th colspan="2">Load Combination(1)</th></tr>
<tr><th>Principal Loads</th><th>Companion Loads</th></tr>
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<tr><td>2</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + (1.5\mathbf{L}_{XC}^{(5)} + 1.0\mathbf{C})$</td><td>~~0.5~~ $1.0\ \mathbf{S}^{(4)} \text{ or } 0.4\mathbf{W}$</td></tr>
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<tr><td>4</td><td>$(1.25\mathbf{D}^{(2)} \text{ or } 0.9\mathbf{D}^{(3)}) + 1.4\mathbf{W}$</td><td>$(1.0\mathbf{C}^{(7)} + 0.5\mathbf{L}_{XC}^{(4)(6)})$</td></tr>
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4.1.5.13. Helicopter Landing Areas

1) Helicopter landing areas on roofs shall be constructed in conformance with the requirements for heliports contained in the “Canadian Aviation Regulations – Part III,” published by Transport Canada.

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4.1.5.13. Helicopter Landing Areas

1) Helicopter landing areas on roofs shall be constructed in conformance with the requirements for heliports contained in ~~the~~ TC SOR/96-433, “Canadian Aviation Regulations – Part III~~,” published by Transport Canada~~.”

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</tr>
</table>

ABC 2014	NBC(AE) 2019	Comments
4.1.5.14. Loads on Guards 1) The minimum specified horizontal load applied inward or 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 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, hichever governs for locations other than those described in Clauses (a) and (b). 2) Individual elements within the *guard*, including solid panels and pickets, shall be designed for a load of 0.5 kN applied 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. 3) The loads required in Sentence (2) need not be considered to act simultaneously with the loads provided for in Sentences (1) and (4). 4) 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). 5) For loads on handrails, refer to Sentence 3.4.6.5.(12).	**4.1.5.14. Loads on Guards and Handrails** (See ~~Appendix A~~ Note A-4.1.5.14. and 4.1.5.15.(1).) **1)** The minimum specified horizontal load applied ~~inward or~~ 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)** ~~2)~~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 direction of the *guard* so as to produce the most critical effect. **5)** ~~3)~~The loads required in Sentence (~~2~~3) need not be considered to act simultaneously with the loads provided for in Sentences (1), (2) and (~~4~~6). **6)** ~~4)~~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). ~~5)For loads on handrails, refer to Sentence 3.4.6.5.(12).~~ **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*.	Inserted new Sentences (2), (4), (7).
4.1.5.15. Loads on Vehicle Guardrails 1) Vehicle guardrails shall be designed for a concentrated load of 22 kN applied horizontally outward at any point 500 mm above the floor surface. (See Appendix A.)	**4.1.5.15. Loads on Vehicle Guardrails** **1)** Vehicle guardrails shall be designed for a concentrated load of 22 kN applied horizontally outward at any point 500 mm above the floor surface so as to produce the most critical effect. (See ~~Appendix A~~ Note A-4.1.5.14. and 4.1.5.15.(1).) **2)** The loads required in Sentence (1) need not be considered to act simultaneously with the loads provided for in Article 4.1.5.14.	Inserted new Sentence (2).

ABC 2014	NBC(AE) 2019	Comments
4.1.5.16. Loads on Walls Acting As Guards 1) Where the floor elevation on one side of a wall, including a wall around a shaft, is more than 600 mm higher than the elevation of the floor or ground on the other side, the wall shall be designed to resist the appropriate lateral design loads rescribed elsewhere in this Section or 0.5 kPa, whichever produces the more critical effect.	**4.1.5.16. Loads on Walls Acting As Guards** **1)** Where the floor elevation on one side of a wall, including a wall around a shaft, is more than 600 mm higher than the elevation of the floor or ground on the other side, the wall shall be designed to resist the appropriate outward lateral design loads prescribed elsewhere in ~~this Section~~ Subsection 4.1.5. or 0.5 kPa acting outward, whichever produces the more critical effect.	
4.1.5.18. Roof Suspended Platforms	~~**4.1.5.18.Roof Suspended Platforms**~~	Deleted entire Article.
4.1.6.2. Specified Snow Load **1)** The specified load, **S**, due to snow and associated rain accumulation on a roof or any other *building* surface subject to snow accumulation shall be calculated using the formula $S = I_s [S_s(C_bC_wC_sC_a) + S_r]$ where I_s = importance factor for snow load as provided in Table 4.1.6.2., S_s = 1-in-50-year ground snow load, in kPa, determined in accordance with Subsection 1.1.3., C_b = basic roof snow load factor in Sentence (2), C_w = wind exposure factor in Sentences (3) and (4), C_s = slope factor in Sentences (5), (6) and (7), C_a = shape factor in Sentence (8), and S_r = 1-in-50-year associated rain load, in kPa, determined in accordance with Subsection 1.1.3., but not greater than $S_s(C_bC_wC_sC_a)$. **2)** The basic roof snow load factor, C_b, shall be 0.8, except that for large roofs it shall be a) $1.0 - (30/l_c)_2$, for roofs with $C_w = 1.0$ and l_c greater than or equal to 70 m, or b) $1.3 - (140/l_c)_2$, for roofs with $C_w = 0.75$ or 0.5 and l_c greater than or equal to 200 m, where l_c = characteristic length of the upper or lower roof, defined as $2w-w_2/l$, in metres, w = smaller plan dimension of the roof, in metres, *l* = larger plan dimension of the roof, in metres.	**4.1.6.2. Specified Snow Load** **1)** The specified load, **S**, due to snow and associated rain accumulation on a roof or any other *building* surface subject to snow accumulation shall be calculated using the formula $S = I_s [S_s(C_bC_wC_sC_a) + S_r]$ where I_s = importance factor for snow load as provided in Table 4.1.6.2.-A, S_s = 1-in-50-year ground snow load, in kPa, determined in accordance with Subsection 1.1.3., C_b = basic roof snow load factor in Sentence (2), C_w = wind exposure factor in Sentences (3) and (4), C_s = slope factor in Sentences (5), (6) and (7), C_a = ~~shape~~ accumulation factor in Sentence (8), and S_r = 1-in-50-year associated rain load, in kPa, determined in accordance with Subsection 1.1.3., but not greater than $S_s(C_bC_wC_sC_a)$. **2)** The basic roof snow load factor, C_b, shall be ~~0.8, except that for large roofs it shall be~~ ~~a) 1.0 - (30/lc)2, for roofs with Cw = 1.0 and lc greater than or equal to 70 m, or~~ ~~b) 1.3 - (140/lc)2, for roofs with Cw = 0.75 or 0.5 and lc greater than or equal to 200 m, where~~ a) be determined as follows: i) $C_b = 0.8 \text{ for } l_c \leq \left(\frac{70}{C_w^2}\right)$, and ii) $C_b = \frac{1}{C_w}\left[1 - (1 - 0.8C_w)\exp\left(-\frac{l_cC_w^2 - 70}{100}\right)\right] \text{ for } l_c > \left(\frac{70}{C_w^2}\right)$ where l_c = characteristic length of the upper or lower roof, defined as $2w-w^2/l$, in ~~metres~~m, w = smaller plan dimension of the roof, in ~~metres~~m, and	Inserted new Table 4.1.6.2.-B. Inserted new Sentence (9). Inserted new Clause 4.1.6.2.(8)(c),(d) and (e)

ABC 2014	NBC(AE) 2019	Comments
	l = larger plan dimension of the roof, in ~~metres.~~m, or b) conform to Table 4.1.6.2.-B, using linear interpolation for intermediate values of $l_cC_w^2$. (See Note A-4.1.6.2.(2).) *****TABLE NOT SHOWN HERE – SEE NBC(AE) 2019***** **Table 4.1.6.2.-B**	
3) ….. **4)** For *buildings* in the Low and Normal Importance Categories as set out in Table 4.1.2.1., the wind exposure factor given in Sentence (3) may be reduced to 0.75, or to 0.5 in exposed areas north of the treeline, where **a)** the *building* is exposed on all sides to wind over open terrain as defined in Clause 4.1.7.1.(5)(a), and is expected to remain so during its life, **b)** the area of roof under consideration is exposed to the wind on all sides with no significant obstructions on the roof, such as parapet walls, within a distance of at least 10 times the difference between the height of the obstruction and CbCwSs/γ metres, where γ is the unit weight of snow on roofs (see Appendix A), and **c)** the loading does not involve the accumulation of snow due to drifting from adjacent surfaces.	**3) …..** **4)** For *buildings* in the Low and Normal Importance Categories as set out in Table 4.1.2.1., the wind exposure factor, C_w, given in Sentence (3) may be reduced to 0.75 for rural areas only, or to 0.5 ~~in~~ for exposed areas north of the treeline, where **a)** the *building* is exposed on all sides to wind over open terrain as defined in Clause ~~4.1.7.1.~~ 4.1.7.3.(5)(a), and is expected to remain so during its life, **b)** the area of roof under consideration is exposed to the wind on all sides with no significant obstructions on the roof, such as parapet walls, within a distance of at least 10 times the difference between the height of the obstruction and $C_bC_wS_s/\gamma$ metres, where γ is the ~~unit~~ specific weight of snow on roofs ~~(see Appendix A),~~ as specified in Article 4.1.6.13., and **c)** the loading does not involve the accumulation of snow due to drifting from adjacent surfaces.	
5) 6) ……	**5) 6)** *****TABLE NOT SHOWN HERE – SEE NBC(AE) 2019***** **Table 4.1.6.2.-B** **Basic Roof Snow Load Factor for** Forming Part of Sentence 4.1.6.2.(2)	
7) The slope factor, Cs, shall be 1.0 when used in conjunction with shape factors for increased snow loads as given in Clauses (8)(b) and (e). **8)** The shape factor, Ca, shall be 1.0, except that where appropriate for the shape of the roof, it shall be assigned other values that account for **a)** non-uniform snow loads on gable, arched or curved roofs and domes, **b)** increased snow loads in valleys, **c)** increased non-uniform snow loads due to snow drifting onto a roof that is at a level lower than other parts of the same building or at a level lower than another building within 5 m of it, **d)** increased non-uniform snow loads on areas adjacent to roof projections, such as penthouses, large chimneys and equipment, and **e)** increased snow or ice loads due to snow sliding or meltwater draining from adjacent roofs.	**7)** ~~The~~ Unless otherwise stated in this Subsection, the slope factor, Cs, shall be 1.0 when used in conjunction with ~~shape~~ accumulation factors for increased snow loads ~~as given in Clauses (8)(b) and (e)~~. **8)** The ~~shape~~ accumulation factor, Ca, shall be 1.0, which corresponds to the uniform snow load case, except that where appropriate for the shape of the roof, it shall be assigned other values that account for ~~a) non-uniform snow loads on gable, arched or curved roofs and domes,~~ ~~b) increased snow loads in valleys,~~ **a)** ~~c)~~ increased non-uniform snow loads due to snow drifting onto a roof that is at a level lower than other parts of the same *building* or at a level lower than another *building* within 5 m of it~~,~~ horizontally, as prescribed in Articles 4.1.6.5., 4.1.6.6. and 4.1.6.8.,	

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	b) ~~d)~~ increased non-uniform snow loads on areas adjacent to roof projections, such as penthouses, large *chimneys* and equipment, ~~and~~ as prescribed in Articles 4.1.6.7. and 4.1.6.8., c) non-uniform snow loads on gable, arch or curved roofs and domes, as prescribed in Articles 4.1.6.9. and 4.1.6.10., d) increased snow or ice loads due to snow sliding as prescribed in Article 4.1.6.11., e) increased snow loads in roof valleys, as prescribed in Article 4.1.6.12., and f) ~~e)~~increased snow or ice loads due to ~~snow sliding or~~ meltwater draining from adjacent ~~roofs~~ *building* elements and roof projections. 9) For shapes not addressed in Sentence (8), C_a corresponding to the non-uniform snow load case shall be established based on applicable field observations, special analyses including local climatic effects, appropriate model tests, or a combination of these methods.	
4.1.6.3. Full and Partial Loading **1)** A roof or other *building* surface and its structural members subject to loads due to snow accumulation shall be designed for the specified load given in Sentence 4.1.6.2.(1), distributed over the entire loaded area. **2)** In addition to the distribution mentioned in Sentence (1), flat roofs and shed roofs, gable roofs of 15° slope or less, and arched or curved roofs shall be designed for the specified uniform snow load indicated in Sentence 4.1.6.2.(1), which shall be calculated using Ca = 1.0, distributed on any one portion of the loaded area and half of this load on the remainder of the loaded area, in such a way as to produce the most critical effects on the member concerned. (See Appendix A.)	**4.1.6.3. Full and Partial Loading** **1)** A roof or other *building* surface and its structural members subject to loads due to snow accumulation shall be designed for the specified load given in Sentence 4.1.6.2.(1), distributed over the entire loaded area. **2)** In addition to the distribution mentioned in Sentence (1), flat roofs and shed roofs, gable roofs of 15° slope or less, and ~~arched~~ arch or curved roofs shall be designed for the specified uniform snow load indicated in Sentence 4.1.6.2.(1), which shall be calculated using the accumulation factor Ca = 1.0, distributed on any one portion of the loaded area and half of this load on the remainder of the loaded area, in such a way as to produce the most critical effects on the member concerned. (See ~~Appendix A~~ Note A-4.1.6.3.(2).)	
	4.1.6.5. Multi-level Roofs	Inserted new Article
	4.1.6.6. Horizontal Gap between a Roof and a Higher Roof	Inserted new Article
	4.1.6.7. Areas Adjacent to Roof Projections	Inserted new Article
	4.1.6.8. Snow Drift at Corners	Inserted new Article
	4.1.6.9. Gable Roofs	Inserted new Article
	4.1.6.10. Arch Roofs, Curved Roofs and Domes	Inserted new Article and new Table 4.1.6.10.
	4.1.6.11. Snow Loads Due to Sliding	Inserted new Article
	4.1.6.12. Valleys in Curved or Sloped Roofs	Inserted new Article
	4.1.6.13. Specific Weight of Snow	Inserted new Article
	4.1.6.14. Snow Removal	Inserted new Article
	4.1.6.15. Ice Loading of Structures	Inserted new Article
	4.1.7.1. Specified Wind Load	Inserted new Article
	4.1.7.2. Classification of Buildings	Inserted new Article
4.1.7.1. Specified Wind Load **1)** The specified external pressure or suction due to wind on part or all of a surface of a *building* shall be calculated using the formula $p = I_w q C_e C_g C_p$	**4.1.7.~~1~~3. ~~Specified Wind Load~~Static Procedure** **1)** The specified external pressure or suction due to wind on part or all of a surface of a *building* shall be calculated ~~using the formula~~ as follows: $p = I_w q C_e \underline{C_t} C_g C_p$	Article renumbered. Inserted new title and new sentences (6),(7) and (10).

ABC 2014	NBC(AE) 2019	Comments
where p = specified external pressure acting statically and in a direction normal to the surface, either as a pressure directed towards the surface or as a suction directed away from the surface, I_W = importance factor for wind load, as provided in Table 4.1.7.1., q =reference velocity pressure, as provided in Sentence (4), C_e = exposure factor, as provided in Sentence (5), C_g = gust effect factor, as provided in Sentence (6), and C_p = external pressure coefficient, averaged over the area of the surface considered. (See Appendix A.) ***TABLE NOT SHOWN HERE – SEE NBC(AE) 2014*** **Table 4.1.7.1.** **Importance Factor for Wind Load, I_W** Forming Part of Sentences 4.1.7.1.(1) and (3) **2)** The net wind load for the *building* as a whole shall be the algebraic difference of the loads on the windward and leeward surfaces, and in some cases, may be calculated as the sum of the products of the external pressures or suctions and the areas of the surfaces over which they are averaged as provided in Sentence (1). (See Appendix A.) **3)** The net specified pressure due to wind on part or all of a surface of a *building* shall be the algebraic difference of the external pressure or suction as provided in Sentence (1) and the specified internal pressure or suction due to wind calculated using the following formula: $P_i = I_w q C_e C_{gi} C_{pi}$ where P_i = specified internal pressure acting statically and in a direction normal to the surface, either as a pressure directed towards the surface or as a suction directed away from the surface, I_w = importance factor for wind load, as provided in Table 4.1.7.1., q = reference velocity pressure, as provided in Sentence (4), C_e = exposure factor, as provided in Sentence (5), C_{gi} = internal gust effect factor, as provided in Sentence (6), and C_{pi} = internal pressure coefficient. (See Appendix A.)	where p = specified external pressure acting statically and in a direction normal to the surface, ~~either as a~~ considered positive when the pressure ~~directed~~ acts towards the surface ~~or as a suction directed~~ and negative when it acts away from the surface, I_w = importance factor for wind load, as provided in Table ~~4.1.7.1.,~~ 4.1.7.3., q = reference velocity pressure, as provided in Sentence (4), C_e = exposure factor, as provided in ~~Sentence (5),~~ Sentences (5) and (7), C_t = topographic factor, as provided in Article 4.1.7.4., C_g = gust effect factor, as provided in Sentence (~~6~~ 8), and C_p = external pressure coefficient, ~~averaged over the area of the surface considered.~~ as provided in Articles 4.1.7.5. and 4.1.7.6. ~~(See Appendix A.)~~ ***TABLE NOT SHOWN HERE – SEE NBC(AE) 2019*** **Table ~~4.1.7.1.~~ 4.1.7.3.** **Importance Factor for Wind Load, I_w** Forming Part of Sentences 4.1.7.3.(1) and (3) **2)** The net wind load for the *building* as a whole shall be the algebraic difference of the loads on the windward and leeward surfaces, and in some cases, may be calculated as the sum of the products of the external pressures or suctions and the areas of the surfaces over which they are averaged as provided in Sentence (1). ~~(See Appendix A.)~~ **3)** The net specified pressure due to wind on part or all of a surface of a *building* shall be the algebraic difference, such as to produce the most critical effect, of the external pressure or suction ~~as provided~~ calculated in accordance with Sentence (1) and the specified internal pressure or suction due to wind calculated ~~using the following formula~~ as follows: $P_i = I_w q C_{ei} C_t C_{gi} C_{pi}$ where P_i = specified internal pressure acting statically and in a direction normal to the surface, either as a pressure directed towards the surface or as a suction directed away from the surface, I_w ~~= importance factor for wind load, as provided in Table 4.1.7.1., q = reference velocity,~~ q, C_t = as defined in Sentence (1), C_{ei} = exposure factor for internal pressure, as provided in Sentence (47), ~~Ce = exposure factor, as provided in Sentence (5),~~ C_{gi} = internal gust effect factor, as provided in Sentence (~~6~~10), and C_{pi} = internal pressure coefficient, as provided in Article 4.1.7.7. ~~(See Appendix A.)~~	

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4) The reference velocity pressure, q, shall be the appropriate value determined in conformance with Subsection 1.1.3., based on a probability of being exceeded in any one year of 1 in 50. **5)** The exposure factor, Ce, shall be **a)** $(h/10)^{0.2}$ but not less than 0.9 for open terrain, where open terrain is level terrain with only scattered *buildings*, trees or other obstructions, open water or shorelines thereof, h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A), **b)** $0.7(h/12)^{0.3}$ but not less than 0.7 for rough terrain, where rough terrain is suburban, urban or wooded terrain extending upwind from the *building* uninterrupted for at least 1 km or 20 times the height of the *building*, whichever is greater, h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A), **c)** an intermediate value between the two exposures defined in Clauses (a) and (b) in cases where the site is less than 1 km or 20 times the height of the *building* from a change in terrain conditions, whichever is greater, provided an appropriate interpolation method is used (see Appendix A), or **d)** if a dynamic approach to the action of wind gusts is used, an appropriate value depending on both height and shielding (see Appendix A). **6)** The gust effect factor, C_g, shall be one of the following values: **a)** for the *building* as a whole and main structural members, $C_g = 2.0$ (see Appendix A), **b)** for external pressures and suctions on small elements including cladding, $C_g = 2.5$, **c)** for internal pressures, $C_{gi} = 2.0$ or a value determined by detailed calculation that takes into account the sizes of the openings in the *building* envelope, the internal volume and the flexibility of the *building* envelope (see Appendix A), or **d)** if a dynamic approach to wind action is used, C_g is a value that is appropriate for the turbulence of the wind and the size and natural frequency of the structure (see Appendix A).	**4)** The reference velocity pressure, q, shall be the appropriate value determined in conformance with Subsection 1.1.3., based on a probability of being exceeded in any one year of 1 in 50. **5)** The exposure factor, C_e, shall be based on the reference height, h, determined in accordance with Sentence (6), for the surface or part of the surface under consideration and shall be **a)** $(h/10)^{0.2}$ but not less than 0.9 for open terrain, where open terrain is level terrain with only scattered *buildings*, trees or other obstructions, open water or shorelines thereof~~, h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A)~~, **b)** $0.7(h/12)^{0.3}$ but not less than 0.7 for rough terrain, where rough terrain is suburban, urban or wooded terrain extending upwind from the *building* uninterrupted for at least 1 km or 20 times the height of the *building*, whichever is greater, ~~h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A),~~or **c)** an intermediate value between the two exposures defined in Clauses (a) and (b) in cases where the site is less than 1 km or 20 times the height of the *building* from a change in terrain conditions, whichever is greater, provided an appropriate interpolation method is used (see Note A-4.1.7.3.(5)(c)). ~~Appendix A), or~~ ~~d)if a dynamic approach to the action of wind gusts is used, an appropriate value depending on both height and shielding (see Appendix A).~~ **6)** The reference height, h, shall be determined as follows: **a)** for *buildings* whose height is less than or equal to 20 m and less than the smaller plan dimension, h shall be the mid-height of the roof above *grade*, but not less than 6 m, **b)** for other *buildings*, h shall be i) the actual height above *grade* of the point on the windward wall for which external pressures are being calculated, ii) the mid-height of the roof for pressures on surfaces parallel to the wind direction, and iii) the mid-height of the *building* for pressures on the leeward wall, and **c)** for any structural element exposed to wind, h shall be the mid-height of the element above the ground. **7)** The exposure factor for internal pressures, C_{ei}, shall be determined as follows: **a)** for *buildings* whose height is greater than 20 m and that have a dominant opening, C_{ei} shall be equal to the exposure factor for external pressures, C_e, calculated at the mid-height of the dominant opening, and **b)** for other *buildings*, C_{ei} shall be the same as the exposure factor for external pressures, C_e, calculated for a reference height, h, equal to the mid-height of the *building* or 6 m, whichever is greater.	

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	8) ~~6)The~~ Except as provided in Sentences (9) and 4.1.7.6.(1), the gust effect factor, C_g, shall be one of the following values: **a)** 2.0 for the *building* as a whole and main structural members, ~~Cg = 2.0 (see Appendix A),~~ or **b)** 2.5 for external pressures and suctions on ~~small elements~~ secondary structural members, including cladding~~, C_g = 2.5,~~. ~~c) for internal pressures, C_{gi} = 2.0 or a value determined by detailed calculation that takes into account the sizes of the openings in the *building* envelope, the internal volume and the flexibility of the *building* envelope (see Appendix A), or~~ ~~d)if a dynamic approach to wind action is used, C_g is a value that is appropriate for the turbulence of the wind and the size and natural frequency of the structure (see Appendix A).~~ **9)** For cases where C_g and C_p are combined into a single product, C_pC_g, the values of C_p and C_g need not be independently specified. (See Article 4.1.7.6.) **10)** The internal gust effect factor, C_{gi}, shall be 2.0, except it is permitted to be calculated using the following equation for large structures enclosing a single large unpartitioned volume that does not have numerous overhead doors or openings: $C_{gi}=1+1/\sqrt{(1+V_0/6950A)}$ where V_0 = internal volume, in m^3, and A = total area of all exterior openings of the volume, in m^2. (See Note A-4.1.7.3.(10).)	
4.1.7.2. Dynamic Effects of Wind	~~4.1.7.2.**Dynamic Effects of Wind**~~	Deleted Article and it covered under new article 4.1.7.2. Classification of Buildings - SEE NBC (AE) 2019.
	4.1.7.4. Topographic Factor **1)** Except as provided in Sentence (2), the topographic factor, C_t, shall be taken as 1.0. **2)** For *buildings* on hills or escarpments with a slope, $H_h/(2L_h)$, greater than 0.1 (see Figure 4.1.7.4.), the topographic factor, C_t, shall be calculated as follows: $C_t=(1+\Delta S/C_g)(1+\Delta S)$ where $\Delta S=\Delta S_{max}(1-\lvert x\rvert/(kL_h))\exp(-\alpha z/L_h)$ where ΔS_{max} = applicable value from Table 4.1.7.4.,	Inserted new article.

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	x = horizontal distance from the peak of the hill or escarpment, L_h = horizontal distance upwind from the peak to the point where the ground surface lies at half the height of the hill or escarpment, or $2H_h$ (where H_h = height of hill or escarpment), whichever is greater, z = height above ground, and k and α = applicable constants from Table 4.1.7.4. based on shape of hill or escarpment. *****TABLE AND FIGURE NOT SHOWN HERE – SEE NBC(AE) 2019***** **Figure 4.1.7.4.** **Speed-up of mean velocity on a hill or escarpment** Forming Part of Sentence 4.1.7.4.(2) **Table 4.1.7.4.** **Parameters for Maximum Speed-up Over Hills and Escarpments** Forming Part of Sentence 4.1.7.4.(2)	
	4.1.7.5. External Pressure Coefficients	Inserted new Article.
	4.1.7.6. External Pressure Coefficients for Low Buildings	Inserted new Article.
	4.1.7.7. Internal Pressure Coefficient	Inserted new Article.
	4.1.7.8. Dynamic Procedure	Inserted new Article.
4.1.7.3. Full and Partial Loading **1)** *Buildings* and structural members shall be capable of withstanding the effects of **a)** the full wind loads acting along each of the 2 principal horizontal axes considered separately, **b)** the wind loads as described in Clause (a) but with 100% of the load removed from any portion of the area, **c)** the wind loads as described in Clause (a) but considered simultaneously at 75% of their full value, and **d)** the wind loads as described in Clause (c) but with 50% of these loads removed from any portion of the area. (See Appendix A.)	**4.1.7.9. ~~4.1.7.3.~~Full and Partial Wind Loading** **1)** ~~*Buildings*~~ 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 ~~as~~ described in Clause (a) but with 100% of the load removed from any one portion of the area, **c)** the wind loads ~~as~~ described in Clause (a) but with both axes considered simultaneously at 75% of their full value, and **d)** the wind loads ~~as~~ described in Clause (c) but with 50% of these loads removed from any portion of the area. (See ~~Appendix~~ Note A-4.1.7.9.(1).)	Renumbered Article.
4.1.7.4. Interior Walls and Partitions	**4.1.7.10. ~~4.1.7.4.~~ Interior Walls and Partitions**	Renumbered Article.
	4.1.7.11. Exterior Ornamentations, Equipment and Appendages	Inserted new Article.
	4.1.7.12. Wind Tunnel Procedure	Inserted new Article.
4.1.8.1. Analysis **1)** The deflections and specified loading due to earthquake motions shall be determined according to the requirements in this Subsection, except that the	**4.1.8.1. Analysis** **1)** ~~*The deflections and specified loading due to earthquake motions shall be determined according to the requirements in this Subsection, except that the*~~	Inserted new sentences (1) to (16).

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requirements in this Subsection need not be considered in design if S(0.2), as defined in Sentence 4.1.8.4.(7), is less than or equal to 0.12.	~~requirements in this Subsection need not be considered in design if S(0.2), as defined in Sentence 4.1.8.4.(7), is less than or equal to 0.12.~~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) 3) 4) 5) 6) 7) 8) 9) 10) 11) 12) 13) 14) 15) 16)	
4.1.8.2. Notation **1)** In this Subsection A_r = response amplification factor to account for type of attachment of mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1), A_x = amplification factor at level x to account for variation of response of mechanical/electrical equipment with elevation within the *building*, as defined in Sentence 4.1.8.18.(1), B_x = ratio at level x used to determine torsional sensitivity, as defined in Sentence 4.1.8.11.(9), B = maximum value of Bx, as defined in Sentence 4.1.8.11.(9), C_p = seismic coefficient for mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1), D_{nx} = plan dimension of the *building* at level x perpendicular to the direction of seismic loading being considered, e_x = distance measured perpendicular to the direction of earthquake loading between centre of mass and centre of rigidity at the level being considered (see Appendix A), F_a = acceleration-based site coefficient, as defined in Sentence 4.1.8.4.(4), F_t = portion of V to be concentrated at the top of the structure, as defined in Sentence 4.1.8.11.(6), F_v = velocity-based site coefficient, as defined in Sentence 4.1.8.4.(4), F_x = lateral force applied to level x, as defined in Sentence 4.1.8.11.(6), h_i, h_n, h_x = the height above the base (i = 0) to level i, n, or x respectively, where the base of the structure is the level at which horizontal earthquake motions are considered to be imparted to the structure, h_s = interstorey height (hi - hi-1), I_E = earthquake importance factor of the structure, as described in Sentence 4.1.8.5.(1), J = numerical reduction coefficient for base overturning moment, as defined in Sentence 4.1.8.11.(5), J_x = numerical reduction coefficient for overturning moment at level x, as defined in Sentence 4.1.8.11.(7), Level i = any level in the *building*, i = 1 for first level above the base, Level n = level that is uppermost in the main portion of the structure, Level x = level that is under design consideration, M_v = factor to account for higher mode effect on base shear, as defined in Sentence 4.1.8.11.(5),	**4.1.8.2. Notation** **1)** In this Subsection A_r = response amplification factor to account for type of attachment of mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1), A_x = amplification factor at level x to account for variation of response of mechanical/electrical equipment with elevation within the *building*, as defined in Sentence 4.1.8.18.(1), B_x = ratio at level x used to determine torsional sensitivity, as defined in Sentence 4.1.8.11.(~~9~~ 10), B = maximum value of B_x, as defined in Sentence 4.1.8.11.(~~9~~ 10), C_p = seismic coefficient for mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1), D_{nx} = plan dimension of the *building* at level x perpendicular to the direction of seismic loading being considered, e_x = distance measured perpendicular to the direction of earthquake loading between centre of mass and centre of rigidity at the level being considered (see ~~Appendix A~~ Note A-4.1.8.2.(1)), F_a= ~~acceleration-based 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.(~~4~~5), F_t = portion of V to be concentrated at the top of the structure, as defined in Sentence 4.1.8.11.(~~6~~7), F_v = ~~velocity-based~~ site coefficient, for application in Subsection 4.1.8., as defined in Sentence 4.1.8.4.(~~4~~7), F_x = lateral force applied to level x, as defined in Sentence 4.1.8.11.(~~6~~7), h_i, h_n, h_x = the height above the base (i = 0) to level i, n, or x respectively, where the base of the structure is the level at which horizontal earthquake motions areconsidered to be imparted to the structure, h_s = interstorey height (h_i - h_{i-1}), I_E = earthquake importance factor of the structure, as described in Sentence 4.1.8.5.(1),	

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M_x = overturning moment at level x, as defined in Sentence 4.1.8.11.(7), N =totalnumberof *storeys* above exterior *grade* to level n, $\bar{N}_{60}$ = Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum, PGA = Peak Ground Acceleration expressed as a ratio to gravitational acceleration, as defined in Sentence 4.1.8.4.(1), PI = plasticity index for clays, R_d = ductility-related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour, as given in Article 4.1.8.9., R_o = overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed according to these provisions, as defined in Article 4.1.8.9., S_p = horizontal force factor for part or portion of a *building* and its anchorage, as given in Sentence 4.1.8.18.(1), S(T) = design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(7), $S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(1), SFRS = Seismic Force Resisting System(s) is that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects defined in Subsection 4.1.8., s_u = average undrained shear strength in the top 30 m of *soil*, T = periodin seconds, T_a = fundamental lateral period of vibration of the *building* or structure in seconds in the direction under consideration, as defined in Sentence 4.1.8.11.(3), T_x = floor torque at level x, as defined in Sentence 4.1.8.11.(10), V = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.11., V_d = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.12., V_e = lateral earthquake elastic force at the base of the structure, as determined by Article 4.1.8.12., V_{ed} = lateral earthquake design elastic force at the base of the structure, as determined by Article 4.1.8.12., V_p = lateral force on a part of the structure, as determined by Article 4.1.8.18., $\bar{V}_s$ = average shear wave velocity in the top 30m of *soil* or *rock,*	J = numerical reduction coefficient for base overturning moment, as defined in Sentence 4.1.8.11.(~~5~~6), J_x = numerical reduction coefficient for overturning moment at level x, as defined in Sentence 4.1.8.11.(~~7~~8), Level i = any level in the *building*, i = 1 for first level above the base, Level n = level that is uppermost in the main portion of the structure, Level x = level that is under design consideration, M_v = factor to account for higher mode effect on base shear, as defined in Sentence 4.1.8.11.(~~5~~6), M_x = overturning moment at level x, as defined in Sentence 4.1.8.11.(~~7~~8), N = total number of *storeys* above exterior *grade* to level n, $\bar{N}_{60}$ = Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum, PGA = Peak Ground Acceleration expressed as a ratio to gravitational acceleration, as defined in Sentence 4.1.8.4.(1), PGA_{ref} = reference PGA for determining F(T), F(PGA) and F(PGV), as defined in Sentence 4.1.8.4.(4), PGV = Peak Ground Velocity, in m/s, as defined in Sentence 4.1.8.4.(1), PI = plasticity index for clays, R_d = ductility-related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour, as given in Article 4.1.8.9., R_o = overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed according to these provisions, as defined in Article 4.1.8.9., R_s = combined overstrength and ductility-related modification factor, as defined in Sentence 4.1.8.1.(7), for application in Article 4.1.8.1., S_p = horizontal force factor for part or portion of a *building* and its anchorage, as given in Sentence 4.1.8.18.(1), S(T) = design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(79), $S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(1), SFRS = Seismic Force Resisting System(s) is that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects defined in Subsection 4.1.8., s_u = average undrained shear strength in the top 30 m of *soil*, T = period in seconds, T_a = fundamental lateral period of vibration of the *building* or structure, in ~~seconds~~s, in the direction under consideration, as defined in Sentence 4.1.8.11.(3),	

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W =*dead load*, as defined in Article 4.1.4.1., except that the minimum *partition* load as defined in Sentence 4.1.4.1.(3) need not exceed 0.5 kPa, plus 25% of the design snow load specified in Subsection 4.1.6., plus 60% of the storage load for areas used for storage, except that *storage garages* need not be considered storage areas, and the full contents of any tanks (see Appendix A), W_i, W_x = portion of W that is located at or is assigned to level i or x respectively, W_p = weight of a part or portion of a structure, e.g., cladding, *partitions* and appendages, δ_{ave} = average displacement of the structure at level x, as defined in Sentence 4.1.8.11.(9), and δ_{max} = maximum displacement of the structure at level x, as defined in Sentence 4.1.8.11.(9).	T_s = fundamental lateral period of vibration of the *building* or structure, in s, in the direction under consideration, as defined in Sentence 4.1.8.1.(7), T_x = floor torque at level x, as defined in Sentence 4.1.8.11.(~~10~~11), TDD = Total Design Displacement of any point in a seismically isolated structure,within or above the isolation system, obtained by calculating the mean + (I_E× the standard deviation) of the peak horizontal displacements from all sets of ground motion histories analyzed, but not less than $\sqrt{I_E}$ × the mean, where the peak horizontal displacement is based on the vector sum of the two orthogonal horizontal displacements considered for each time step, V = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.11., V_d = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.12., V_e = lateral earthquake elastic force at the base of the structure, as determined by Article 4.1.8.12., V_{ed} = lateral earthquake design elastic force at the base of the structure, as determined by Article 4.1.8.12., V_p = lateral force on a part of the structure, as determined by Article 4.1.8.18., V_s = lateral earthquake design force at the base of the structure, as determined by Sentence 4.1.8.1.(7), for application in Article 4.1.8.1., $\overline{V}_{s30}$ = average shear wave velocity in the top 30 m of *soil* or *rock,* W = *dead load*, as defined in Article 4.1.4.1., except that the minimum *partition* load as defined in Sentence 4.1.4.1.(3) need not exceed 0.5 kPa, plus 25% of the design snow load specified in Subsection 4.1.6., plus 60% of the storage load for areas used for storage, except that *storage garages* need not be considered storage areas, and the full contents of any tanks (see ~~Appendix~~ Note A-4.1.8.2.(1)), W_i, W_x = portion of W that is located at or is assigned to level i or x respectively, W_p = weight of a part or portion of a structure, e.g., cladding, *partitions* and appendages, W_t = sum of W_i over the height of the *building*, for application in Sentence 4.1.8.1.(7), δ_{ave} = average displacement of the structure at level x, as defined in Sentence 4.1.8.11.(~~9~~10), and δ_{max} = maximum displacement of the structure at level x, as defined in Sentence 4.1.8.11.(~~9~~10).	
4.1.8.4. Site Properties **1)** The peak ground acceleration (PGA) and the 5% damped spectral response acceleration values, Sa(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, and 2.0 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)** 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, ~~and~~ 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.	Inserted new Tables: Table 4.1.8.4.-D Table 4.1.8.4.-E Table 4.1.8.4.-F Table 4.1.8.4.-G Table 4.1.8.4.-H

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TABLE NOT SHOWN HERE – SEE ABC 2014

Table 4.1.8.4.

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 Appendix A).
(2) If $\overline{V}_s$ has been measured in-situ, the F_a and F_v values derived from Tables 4.1.8.4.B. and 4.1.8.4.C. may be multiplied by (1500/ $\overline{V}_s$).
(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,

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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)

<table>
<tr><th rowspan="2">Site Class</th><th rowspan="2">Ground Profile Name</th><th colspan="3">Average Properties in Top 30 m, as per ~~Appendix~~Note A -4.1.8.4.(3) and Table 4.1.8.4.-A</th></tr>
<tr><th>Average Shear Wave Velocity, $\overline{V}_{s30}$, m/s</th><th>Average Standard Penetration Resistance, $\overline{N}_{60}$</th><th>*Soil* Undrained Shear Strength, s_u</th></tr>
<tr><td>A</td><td>Hard *rock*(1)(2)</td><td>$\overline{V}_{s30} > 1500$</td><td>n/a</td><td>n/a</td></tr>
<tr><td>B</td><td>*Rock*(1)</td><td>$760 < \overline{V}_{s30} \leq 1500$</td><td>n/a</td><td>n/a</td></tr>
<tr><td>C</td><td>Very dense *soil* and soft *rock*</td><td>$360 < \overline{V}_{s30} < 760$</td><td>$\overline{N}_{60} > 50$</td><td>$s_u > 100$ kPa</td></tr>
<tr><td>D</td><td>Stiff *soil*</td><td>$180 < \overline{V}_{s30} < 360$</td><td>$15 \leq \overline{N}_{60} \leq 50$</td><td>50 kPa $< s_u \leq$ 100 kPa</td></tr>
<tr><td rowspan="2">E</td><td rowspan="2">Soft *soil*</td><td>$\overline{V}_{s30} < 180$</td><td>$\overline{N}_{60} < 15$</td><td>$s_u < 50$ kPa</td></tr>
<tr><td colspan="3">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</td></tr>
<tr><td>F</td><td>Other *soils*(3)</td><td colspan="3">Site-specific evaluation required</td></tr>
</table>

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 ~~Appendix~~Note A-4.1.8.4.(3) and Table 4.1.8.4.-A).
(2) ~~If~~Where $\overline{V}_{s30}$ has been measured in-situ, the F~~$_a$~~(T) ~~and F_v-~~values for Site Class A derived from Tables 4.1.8.4.- B~~.~~ ~~and~~to 4.1.8.4. ~~C.~~G ~~may~~ are permitted to be multiplied by the factor 0.04 + $(1500/\overline{V}_{s30})^{½}$.
(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,

</td>
<td>Table 4.1.8.4.-I</td>
</tr>
</table>

ABC 2014	NBC(AE) 2019	Comments
(c) highly plastic clays (PI > 75) more than 8 m thick, and (d) soft to medium stiff clays more than 30 m thick.	(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.	
2) Site classifications for ground shall conform to Table 4.1.8.4.A. and shall be determined using $\overline{V}_s$ except as provided in Sentence (3).	**2)** Site classifications for ground shall conform to Table 4.1.8.4. -A~~.~~ and shall be determined using $\overline{V}_{s30}$ ~~except as provided in~~, or where $\overline{V}_{s30}$ is not known, using Sentence (3).	
3) If average shear wave velocity, $\overline{V}_s$, is not known, Site Class shall be determined from energy-corrected Average Standard Penetration Resistance, $\overline{N}_{60}$, or from *soil* average undrained shear strength, S_u, as noted in Table 4.1.8.4.A., $\overline{N}_{60}$ and Su being calculated based on rational analysis. (See Appendix A.)	**3)** If average shear wave velocity, $\overline{V}_{s30}$, is not known, Site Class shall be determined from energy-corrected Average Standard Penetration Resistance, $\overline{N}_{60}$, or from *soil* average undrained shear strength, s_u, as noted in Table 4.1.8.4. -A~~.~~, $\overline{N}_{60}$ and s_u being calculated based on rational analysis. (See ~~Appendix~~Note A-4.1.8.4.(3) and Table 4.1.8.4.-A.) **4)** For the purpose of determining the values of F(T) to be used in the calculation of design spectral acceleration, S(T), in Sentence (9), and the values of F(PGA) and F(PGV), the value of PGAref to be used with Tables 4.1.8.4.-B to 4.1.8.4.-I shall be taken as **a)** 0.8 PGA, where the ratio Sa(0.2)/PGA < 2.0, and **b)** PGA, otherwise.	
4) Acceleration- and velocity-based site coefficients, F_a and F_v, shall conform to Tables 4.1.8.4.B. and 4.1.8.4.C. using linear interpolation for intermediate values of $S_a(0.2)$ and Sa(1.0). ***TABLEs NOT SHOWN HERE – SEE ABC 2014*** Table 4.1.8.4.B. Table 4.1.8.4.C.	~~**4)** Acceleration- and velocity-based site coefficients, F_a and F_v, shall conform to Tables 4.1.8.4.B. and 4.1.8.4.C. using linear interpolation for intermediate values of $S_a(0.2)$ and $S_a(1.0)$.~~ **5)** The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.-B to 4.1.8.4.-I using linear interpolation for intermediate values of PGA_{ref}.	
5) Site-specific evaluation is required to determine F_a and F_v for Site Class F. (See A-4.1.8.4.(3) and Table 4.1.8.4.A. in Appendix A.)	**~~5~~6)** Site-specific evaluation is required to determine F~~A~~(T), F~~v~~ (PGA) and F(PGV) for Site Class F. (See Note A-4.1.8.4.(3) and Table 4.1.8.4.-A.) **7)** For all applications in Subsection 4.1.8., Fa = F(0.2) and Fv = F(1.0).	
6) For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, Site Class and the corresponding values of F_a and F_v may be determined as described in Tables 4.1.8.4.A., 4.1.8.4.B., and 4.1.8.4.C. by assuming that the soils are not liquefiable. (See A-4.1.8.4.(3) and Table 4.1.8.4.A. in Appendix A.) 7) The design spectral acceleration values of S(T) shall be determined as follows, using linear interpolation for intermediate values of T: $S(T) = F_aS_a(0.2)$ for $T \leq 0.2$ s $= F_vS_a(0.5)$ or FaSa(0.2), whichever is smaller for $T = 0.5$ s $= F_vS_a(1.0)$ for $T = 1.0$ s $= F_vS_a(2.0)$ for $T = 2.0$ s $= F_vS_a(2.0)/2$ for $T \geq 4.0$ s	**~~6~~8)** For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, Site Class and the corresponding values of ~~F_a and~~ F~~v~~(T) may be determined as described in Tables 4.1.8.4.-A~~.~~, 4.1.8.4.-B~~.~~, and 4.1.8.4.-C~~.~~ by assuming that the soils are not liquefiable. (See Note A-4.1.8.4.(3) and Table 4.1.8.4.-A. ~~in Appendix A.~~) ~~7)~~9)The design spectral acceleration values of S(T) shall be determined as follows, using linear interpolation for intermediate values of T: S(T) = F~~a~~(0.2)$S_a(0.2)$ or F(0.5)Sa(0.5), whichever is larger, for $T \leq 0.2$ s = F~~v~~(0.5)$S_a(0.5)$ ~~or $F_aS_a(0.2)$, whichever is smaller~~ for $T = 0.5$ s = F~~v~~(1.0)$S_a(1.0)$ for $T = 1.0$ s = F~~v~~(2.0)$S_a(2.0)$ for $T = 2.0$ s ~~= FvSa(2.0)/2 for T ≥ 4.0 s~~ = F(5.0)Sa(5.0)for T = 5.0 s = F(10.0)Sa(10.0) for T ≥ 10.0 s *****TABLES NOT SHOWN HERE – SEE NBC(AE) 2019*****	

<table>
<tr><th>ABC 2014</th><th>NBC(AE) 2019</th><th>Comments</th></tr>
<tr><td></td><td>**Table 4.1.8.4.-B~~.~~**
Values of F~~$_a$~~(0.2) as a Function of Site Class and ~~S_a(0.2)~~ PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)
Table 4.1.8.4.-C~~.~~
Values of F~~$_v$~~(0.5) as a Function of Site Class and ~~S_a(1.0)~~ PGA_{ref}
Forming Part of Sentences 4.1.8.4.(4) and (5)

Table 4.1.8.4.-D
Table 4.1.8.4.-E
Table 4.1.8.4.-F
Table 4.1.8.4.-G
Table 4.1.8.4.-H
Table 4.1.8.4.-I</td><td></td></tr>
<tr><td>**4.1.8.6. Structural Configuration**

TABLEs NOT SHOWN HERE – SEE ABC 2014</td><td>**4.1.8.6. Structural Configuration**

Full TABLE NOT SHOWN HERE – SEE NBC(AE) 2019

Table 4.1.8.6.
Structural Irregularities(1)
Forming Part of Sentence 4.1.8.6.(1)
<table>
<tr><th>Type</th><th>Irregularity Type and Definition</th><th>Notes</th></tr>
<tr><td>…</td><td>…</td><td>…</td></tr>
<tr><td>6</td><td>…</td><td>(2)(3)</td></tr>
<tr><td>7</td><td>**Torsional Sensitivity (to be considered when diaphragms are not flexible)**
Torsional sensitivity shall be considered to exist when the ratio B calculated according to Sentence 4.1.8.11.(~~9~~10) exceeds 1.7.</td><td>(2)(3)(4)(6)</td></tr>
<tr><td>8</td><td>…</td><td>(2)(4)(7)</td></tr>
<tr><td>9</td><td>Gravity-Induced Lateral Demand Irregularity Gravity-induced lateral demand irregularity on the SFRS shall be considered to exist where the ratio, α, calculated in accordance with Sentence 4.1.8.10.(5), exceeds 0.1 for an SFRS with self-centering characteristics and 0.03 for other systems.</td><td>(2)(3)(4)(7)</td></tr>
</table>
</td><td>Inserted new update to Table 4.1.8.6.</td></tr>
<tr><td>**4.1.8.9. SFRS Force Reduction Factors, System Overstrength Factors, and General Restrictions**</td><td>**4.1.8.9. SFRS Force Reduction Factors, System Overstrength Factors, and General Restrictions**</td><td></td></tr>
</table>

ABC 2014

1) The values of Rd and Ro and the corresponding system restrictions shall conform to Table 4.1.8.9. and the requirements of this Subsection.

2) 3) …

4) For vertical variations of RdRo, excluding rooftop structures not exceeding two *storeys* in height whose weight is less than the greater of 10% of W and 30% of Wi of the level below, the value of RdRo used in the design of any *storey* shall be less than or equal to the lowest value of RdRo used in the given direction for the *storeys* above, and the requirements of Sentence 4.1.8.15.(5) must be satisfied. (See Appendix A.)

5) If it can be demonstrated through testing, research and analysis that the seismic performance of a structural system is at least equivalent to one of the types of SFRS mentioned in Table 4.1.8.9., then such a structural system will qualify for values of Rd and Ro corresponding to the equivalent type in that Table. (See Appendix A.)

TABLEs NOT SHOWN HERE – SEE ABC 2014

NBC(AE) 2019

1) ~~The~~ 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.

2) 3) …

4) For vertical variations of R_dR_o, excluding rooftop structures not exceeding two *storeys* in height whose weight is less than the greater of 10% of W and 30% of W_i of the level below, the value of R_dR_o used in the design of any *storey* shall be less than or equal to the lowest value of R_dR_o used in the given direction for the *storeys* above, and the requirements of Sentence 4.1.8.15.(~~5~~6) must be satisfied. (See ~~Appendix~~ Note A-4.1.8.9.(4).)

5) If it can be demonstrated through testing, research and analysis that the seismic performance of a structural system is at least equivalent to one of the types of SFRS mentioned in Table 4.1.8.9., then such a structural system will qualify for values of R_d and R_o corresponding to the equivalent type in that Table. (See ~~Appendix~~ Note A-4.1.8.9.(5).)

Full TABLE NOT SHOWN HERE – SEE NBC(AE) 2019

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 Sentence 4.1.8.9.(1)

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)							
…	…	…	…	…	…	…	…
Concrete Structures Designed and Detailed According to ~~CAN/~~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

Comments

<table>
<tr><th>ABC 2014</th><th>NBC(AE) 2019</th><th>Comments</th></tr>
<tr>
<td>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; height may be limited in other Parts of the Code.
Numbers in this Table are maximum height limits in m.
The most stringent requirement governs.
(3) Higher design force levels are prescribed in CSA S16 for some heights of *buildings*.</td>
<td>
<table>
<tr><td>Moderately ductile partially coupled walls</td><td>2.0</td><td>1.4</td><td>NL</td><td>NL</td><td>NL</td><td>60</td><td>60</td></tr>
<tr><td>Ductile shear walls</td><td>3.5</td><td>1.6</td><td>NL</td><td>NL</td><td>NL</td><td>NL</td><td>NL</td></tr>
<tr><td>Moderately ductile shear walls</td><td>2.0</td><td>1.4</td><td>NL</td><td>NL</td><td>NL</td><td>60</td><td>60</td></tr>
<tr><td>Conventional construction</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Moment-resisting frames</td><td>1.5</td><td>1.3</td><td>NL</td><td>NL</td><td>~~15~~20</td><td>~~NP~~15</td><td>~~NP~~10(5)</td></tr>
<tr><td>Shear walls</td><td>1.5</td><td>1.3</td><td>NL</td><td>NL</td><td>40</td><td>30</td><td>30</td></tr>
<tr><td>Two-way slabs without beams</td><td>1.3</td><td>1.3</td><td>20</td><td>15</td><td>NP</td><td>NP</td><td>NP</td></tr>
<tr><td>Tilt-up construction</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Moderately ductile walls and frames</td><td>2.0</td><td>1.3</td><td>30</td><td>25</td><td>25</td><td>25</td><td>25</td></tr>
<tr><td>Limited ductility walls and frames</td><td>1.5</td><td>1.3</td><td>30</td><td>25</td><td>20</td><td>20</td><td>20(6)</td></tr>
<tr><td>Conventional walls and frames</td><td>1.3</td><td>1.3</td><td>25</td><td>20</td><td>NP</td><td>NP</td><td>NP</td></tr>
<tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr>
<tr><td colspan="8">Masonry Structures Designed and Detailed According to CSA S304~~.1~~</td></tr>
<tr><td>Ductile shear walls</td><td>3.0</td><td>1.5</td><td>NL</td><td>NL</td><td>60</td><td>40</td><td>40</td></tr>
<tr><td>Moderately ductile shear walls</td><td>2.0</td><td>1.5</td><td>NL</td><td>NL</td><td>60</td><td>40</td><td>40</td></tr>
<tr><td>~~Limited ductility shear walls~~</td><td>~~1.5~~</td><td>~~1.5~~</td><td>~~NL~~</td><td>~~NL~~</td><td>~~40~~</td><td>~~30~~</td><td>~~30~~</td></tr>
<tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr>
<tr><td colspan="8">Cold-Formed Steel Structures Designed and Detailed According to ~~CAN/~~CSA~~-~~ S136</td></tr>
<tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr>
</table>
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~~; height may be limited in other Parts of the Code.~~ Numbers in this Table are maximum height limits 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.
</td>
<td></td>
</tr>
<tr>
<td>**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 IEFaSa(0.2) is less than 0.2 and the forces used for design of the SFRS are multiplied by RdRo.
2) *Post-disaster buildings* shall
a) not have any irregularities conforming to Types 1, 3, 4, 5 and 7 as described in Table 4.1.8.6., in cases where IEFaSa(0.2) is equal to or greater than 0.35,</td>
<td>**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 IEFaSa(0.2) is less than 0.2 and the forces used for design of the SFRS are multiplied by RdRo.
2) *Post-disaster buildings* shall
a) not have any irregularities conforming to Types 1, 3, 4, 5, 7and ~~7~~9 as described in Table 4.1.8.6., in cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35,</td>
<td>Inserted new sentences (5), (6) and (7).</td>
</tr>
</table>

ABC 2014	NBC(AE) 2019	Comments
b) not have a Type 6 irregularity as described in Table 4.1.8.6., c) have an SFRS with an Rd 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, Ta, of 1.0 s or greater, and where IEFvSa(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 IEFaSa(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 Appendix A.)	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 $I_EF_vS_a(1.0)$ is greater than 0.25, shear walls that are other than wood-based and form~~ing~~ 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 IEFaSa(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 ~~Appendix A~~ 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: $\alpha = Q_G/Q_y$ 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 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).)	
4.1.8.11. Equivalent Static Force Procedure for Structures Satisfying the Conditions of Article 4.1.8.7. **1)** .. **2)** Except as provided in Sentence (11), the minimum lateral earthquake force, V, shall be calculated using the following formula: a) b) … c) for *buildings* located on a site other than Class F and having an SFRS with an Rd equal to or greater than 1.5, V need not be greater than $\frac{2}{3}S(0.2)I_EW/(R_dR_o)$	**4.1.8.11. Equivalent Static Force Procedure for Structures Satisfying the Conditions of Article 4.1.8.7.** **1)** …. **2)** Except as provided in Sentence (~~11~~12), the minimum lateral earthquake force, V, shall be calculated using the following formula: a) b) …… 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 $\frac{2}{3}S(0.2)I_EW/(R_dR_o)$ and	Inserted new sentences (4) and (12).

ABC 2014	NBC(AE) 2019	Comments
3) The fundamental lateral period, T_a, in the direction under consideration in Sentence (2), shall be determined as: a) b) C) …. ***TABLE NOT SHOWN HERE – SEE ABC 2014*** **Table 4.1.8.11.** **Higher Mode Factor, Mv, and Base Overturning Reduction Factor, J(1)(2)** **Forming Part of Sentence 4.1.8.11.(5)** **5) 6) 7) 8) 9) 10) 11) ..**	$S(0.5)I_EW/(R_dR_o)$ **3)** Except as provided in Sentence (4), ~~T~~the fundamental lateral period, T_a, in the direction under consideration in Sentence (2), shall be determined as: a) b) c) d) …. (See ~~Appendix~~ Note A-4.1.8.11.(3).) **4)** For single-*storey buildings* with steel deck or wood roof diaphragms, the fundamental lateral period, T_a, in the direction under consideration is permitted to be taken as a) $0.05\ (h_n)^{3/4} + 0.004\ L$ for shear walls, b) $0.035\ h_n + 0.004\ L$ for steel moment frames and steel braced frames, or c) the value obtained from methods of mechanics using a structural model that complies with the requirements of Sentence 4.1.8.3.(8), except that T_a shall not be greater than 1.5 times the value determined in Clause (a) or (b), as applicable, where L is the shortest length of the diaphragm, in m, between adjacent vertical elements of the SFRS in the direction perpendicular to the direction under consideration. **~~4~~5)…** **~~5~~6)…** **~~6~~7)…** ***TABLES NOT SHOWN HERE – SEE NBC(AE) 2019*** **Table 4.1.8.11.** **Higher Mode Factor, Mv, and Base Overturning Moment Reduction Factor, J(1)(2)(3)(4)** Forming Part of Sentence 4.1.8.11.~~(5)~~(6) **Notes to Table 4.1.8.11.:** (1) For intermediate values of ~~M_v between fundamental lateral periods, Ta, of 1.0 s and 2.0 s and between 2.0 s and 4.0 s, the product $S(T_a)$•~~ the spectral ratio $S(0.2)/S(5.0)$, M_v and J shall be obtained by linear interpolation. (2) ~~Values of J between~~ For intermediate values of the fundamental lateral ~~periods, T_a, of 0.5 s and 2.0 s and between 2.0 s and 4.0 s~~ 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).	

ABC 2014	NBC(AE) 2019	Comments
	(4) For a combination of different seismic force resisting systems (SFRS) not given in Table 4.1.8.11. that are in the same direction under consideration, use the highest Mv factor of all the SFRS and the corresponding value of J. ~~(3)~~(5) For fundamental lateral periods, T_a , greater than 2.0 s, use the ~~values for Ta = 2.0.~~ 2.0 s values obtained in accordance with Note (1). See Clause 4.1.8.11.(2)(b). ~~(4)~~ (6) A “coupled” wall is a wall system with coupling beams, where at least 66% of the base overturning moment resisted by the wall system is carried by the axial tension and compression forces resulting from shear in the coupling beams. ~~(5) For hybrid systems, values corresponding to walls must be used or a dynamic analysis must be carried out as per Article 4.1.8.12.~~ (7) For fundamental lateral periods, Ta, greater than 4.0 s, use the 4.0 s values of $S(T_a)M_v$ obtained by interpolation between 2.0 s and 5.0 s using the value of M_v obtained in accordance with Note (1). See Clause 4.1.8.11.(2)(a). (8) For fundamental lateral periods, T_a, greater than 4.0 s, use the 4.0 s values of J obtained by interpolation between 2.0 s and 5.0 s using the value of J obtained in accordance with Note (1). See Clause 4.1.8.11.(2)(a). **~~7~~8)** The structure shall be designed to resist overturning effects caused by the earthquake forces determined in Sentence (~~6~~7) and the overturning moment at level x, M_x, shall be determined using the following equation: … **~~8~~9)** Torsional effects that are concurrent with the effects of the forces mentioned in Sentence (~~6~~7) and are caused by the simultaneous actions of the following torsional moments shall be considered in the design of the structure according to Sentence (~~10~~11): **~~9~~10)**… **~~10~~11)** Torsional effects shall be accounted for as follows: a) for a *building* with B ≤ 1.7 or where $I_EF_aS_a(0.2)$ is less than 0.35, 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) $T_x = F_x(e_x + 0.10 D_{nx})$, 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 (~~6~~7) and where each element of the *building* is designed for the most severe effect of the above load cases, or b)….	

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	12) Where the fundamental lateral period, T_a, is determined in accordance with Clause (3)(d) and the building is constructed with more than 4 storeys of continuous wood construction and has a 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., the lateral earthquake force, V, as determined in accordance with Sentence (2) shall be multiplied by 1.2 but need not exceed the value determined by using Clause (2)(c). (See Note A-4.1.8.10.(4).)	
4.1.8.12. Dynamic Analysis Procedure **1)** The Dynamic Analysis Procedure shall be in accordance with one of the following methods: a) Linear Dynamic Analysis by either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method using a structural model that complies with the requirements of Sentence 4.1.8.3.(8) (see Appendix A), or b) Non-linear Dynamic Analysis, in which case a special study shall be performed (see Appendix A). **2) 3) 4) 5)** **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 following factor to obtain the design elastic base shear, V_{ed}: $\frac{2S(0.2)}{3S(T_a)} \leq 1.0$ **7) 8) 9) 10) 11)** **12)** For *buildings* constructed with more than 4 *storeys* of continuous wood construction, having a 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., and whose fundamental lateral period, T_a, is determined in accordance with Clause 4.1.8.11.(3)(d), the design base shear, V_d, shall be taken as the larger of the design base shear obtained in accordance with Sentence (7) and 100% of the lateral earthquake force, V, obtained in accordance with Article 4.1.8.11.	**4.1.8.12. Dynamic Analysis Procedure** **1)** ~~The~~ Except as provided in Articles 4.1.8.19. and 4.1.8.21., the Dynamic Analysis Procedure shall be in accordance with one of the following methods: **a)** Linear Dynamic Analysis by either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method using a structural model that complies with the requirements of Sentence 4.1.8.3.(8) (see ~~Appendix A~~ Note A-4.1.8.12.(1)(a)), or **b)** Non-linear Dynamic Analysis, in which case a special study shall be performed (see ~~Appendix A~~ Note A-4.1.8.12.(1)(b)). **2)** The spectral acceleration values used in the Modal Response Spectrum Method shall be the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(~~7~~9). **3)** The ground motion histories used in the Numerical Integration Linear Time History Method shall be compatible with a response spectrum constructed from the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(~~7~~9). (See ~~Appendix~~ Note A-4.1.8.12.(3).) **4)** The effects of accidental torsional moments acting concurrently with the lateral earthquake forces that cause them shall be accounted for by the following methods: a) the static effects of torsional moments due to (± 0.10 D_{nx})F_x at each level x, where F_x is either determined from the elastic dynamic analysis or determined from Sentence 4.1.8.11.(~~6~~7) multiplied by R_dR_o/I_E, shall be combined with the effects determined by dynamic analysis (see ~~Appendix A~~ Note A-4.1.8.12.(4)(a)), or b) if B, as defined in Sentence 4.1.8.11.(~~9~~10), is less than 1.7, it is permitted to use a three-dimensional dynamic analysis with the centres of mass shifted by a distance of –0.05 D_{nx} and + 0.05 D_{nx}. **5)** Except as provided in Sentence (6), the design elastic base shear, V_{ed}, shall be equal to the elastic base shear, V_e, obtained from a Linear Dynamic Analysis. **6)** For structures located on sites other than Class F that have an SFRS with Rd 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}:	

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	$\frac{2S\,(0.2)}{3S\,(T_a)} \leq 1.0$ and $S\,(0.5)/S(T_a) \leq 1.0$ **7) 8) 9) 10) 11)…** **12)** For *buildings* constructed with more than 4 *storeys* of continuous wood construction, having a 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., and whose fundamental lateral period, T_a, is determined in accordance with Clause 4.1.8.11.(3)(d), the design base shear, V_d, shall be taken as the larger ~~of the design base shear obtained~~ value of V_d determined in accordance with Sentence (7) and 100% of ~~the lateral earthquake force, V, obtained in accordance with Article 4.1.8.11.~~V. (See Note A-4.1.8.10.(4).)	
4.1.8.13. Deflections and Drift Limits 1) Lateral deflections of a structure shall be calculated in accordance with the loads and requirements defined in this Subsection. 2) Lateral deflections obtained from a linear elastic analysis using the methods given in Articles 4.1.8.11. and 4.1.8.12. and incorporating the effects of torsion, including accidental torsional moments, shall be multiplied by R_dR_o/I_E to give realistic values of anticipated deflections. 3) Based on the lateral deflections calculated in Sentence (2), the largest inter*storey* deflection at any level shall be limited to 0.01 hs for *post-disaster buildings*, 0.02 hs for High Importance Category *buildings*, and 0.025 hs for all other *buildings*. 4) The deflections calculated in Sentence (2) shall be used to account for sway effects as required by Sentence 4.1.3.2.(12). (See Appendix A.)	**4.1.8.13. Deflections and Drift Limits** **1)** Except as provided in Sentences (5) and (6), ~~L~~lateral deflections of a structure shall be calculated in accordance with the loads and requirements defined in this Subsection. **2)** Lateral deflections obtained from a linear elastic analysis using the methods given in Articles 4.1.8.11. and 4.1.8.12. and incorporating the effects of torsion, including accidental torsional moments, shall be multiplied by R_dR_o/I_E and increased as required in Sentences 4.1.8.10.(6) and 4.1.8.16.(1) to give realistic values of anticipated deflections. **3)** Based on the lateral deflections calculated in Sentences (2), (5) and (6),the largest inter*storey* deflection at any level shall be limited to 0.01 h_s for *post-disaster buildings*, 0.02 h_s for High Importance Category *buildings*, and 0.025 hs for all other *buildings*. **4)** The deflections calculated in Sentence (2) shall be used to account for sway effects as required by Sentence 4.1.3.2.(12). (See ~~Appendix~~ Note A -4.1.8.13.(4).) **5)** The lateral deflections of a seismically isolated structure shall be calculated in accordance with Article 4.1.8.20. **6)** The lateral deflections of a structure with supplemental energy dissipation shall be calculated in accordance with Article 4.1.8.22.	Inserted new sentence (5) and (6).
4.1.8.14. Structural Separation 1) Adjacent structures shall either be separated by the square root of the sum of the squares of their individual deflections calculated in Sentence 4.1.8.13.(2), or shall be connected to each other. **2) 3) 4)**	**4.1.8.14. Structural Separation** **1)** Adjacent structures shall ~~either~~ be a) separated by a distance equal to at least the square root of the sum of the squares of their individual deflections calculated in Sentence 4.1.8.13.(2), or ~~shall be~~ b) connected to each other. **2) 3) 4) …**	
4.1.8.15. Design Provisions **1) 2) 3) 4) 5) 6)** **7)** Except as provided in Sentence (8), the design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with	**4.1.8.15. Design Provisions** **1) 2) 3)** **4)** For single-*storey buildings* with steel deck or wood roof diaphragms designed with a value of R_d greater than 1.5 and where the calculated maximum relative deflection, Δ_D, of	Inserted new sentences (4) and (10). Remaining Sentences renumbered.

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Sentence 4.1.8.7.(1) with R_dR_o taken as 1.0, unless otherwise provided by the applicable referenced design standards for elements, in which case the design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with Sentence 4.1.8.7.(1) with R_dR_o taken as 1.3. (See ppendix A.) **8)** If *foundation* rocking is accounted for, the design forces for the SFRS need not exceed the maximum values associated with *foundation* rocking, provided that R_d and R_o for the type of SFRS used conform to Table 4.1.8.9. and that the *foundation* is designed in accordance with Sentence 4.1.8.16.(1).	the diaphragm under lateral loads exceeds 50% of the average *storey* drift, Δ_B, of the adjoining vertical elements of the SFRS, dynamic magnification of the inelastic response due to the in-plane diaphragm deformations shall be accounted for in the design as follows: **a)** the vertical elements of the SFRS shall be designed and detailed to any one of the following: i) to accommodate the anticipated magnified lateral deformations taken as $R_oR_d (\Delta_B + \Delta_D) - R_o\Delta_D$, ii) to resist the forces magnified by $R_d(1 + \Delta_D/\Delta_B)/(R_d + \Delta_D/\Delta_B)$, or iii) by a special study, and **b)** the roof diaphragm and chords shall be designed for in-plane shears and moments determined while taking into consideration the inelastic higher mode response of the structure. (See Note A-4.1.8.15.(4).) **5) 6) 7)** **8)** ~~7)Except as provided in Sentence (8), the~~The design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with Sentence 4.1.8.7.(1) with R_dR_o taken as 1.0, unless otherwise provided by the applicable referenced design standards for elements, in which case the design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with Sentence 4.1.8.7.(1) with R_dR_o taken as less than or equal to 1.3. (See ~~Appendix A~~ Note A-4.1.8.15.(8).) **9)** ~~8)If *foundation* rocking is accounted for, the design forces for the SFRS need not exceed the maximum values associated with *foundation* rocking, provided that~~*Foundations* need not be designed to resist the lateral load overturning capacity of the SFRS, provided the design and the R_d and R_o for the type of SFRS used conform to Table 4.1.8.9. and that the *foundation* is designed in accordance with Sentence 4.1.8.16.(4). **10)** *Foundation* displacements and rotations shall be considered as required by Sentence 4.1.8.16.(1).	
4.1.8.16. Foundation Provisions **1)** *Foundations* shall be designed to resist the lateral load capacity of the SFRS, except that when the *foundations* are allowed to rock, the design forces for the *foundation* need not exceed those determined in Sentence 4.1.8.7.(1) using an R_dR_o equal to 2.0. (See Appendix A.) **2)** The design of *foundations* shall be such that they are capable of transferring earthquake loads and effects between the *building* and the ground without exceeding the capacities of the *soil* and *rock*. **3)** In cases where $I_EF_aS_a(0.2)$ 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 Appendix A), b) *piles*, drilled piers, and *caissons* shall be embedded a minimum of 100 mm into the *pile* cap or structure, and	**4.1.8.16. Foundation Provisions** **1)** ~~*Foundations* shall be designed to resist the lateral load capacity of the SFRS, except that when the *foundations* are allowed to rock, the design forces for the *foundation* need not exceed those determined in Sentence 4.1.8.7.(1) using an R_dR_o equal to 2.0. (See Appendix A.)~~The increased displacements of the structure resulting from *foundation* movement shall be shown to be within acceptable limits for both the SFRS and the structural framing elements not considered to be part of the SFRS. (See Note A-4.1.8.16.(1).) **2)** Except as provided in Sentences (3) and (4), *foundations* shall be designed to have factored shear and overturning resistances greater than the lateral load capacity of the SFRS. (See Note A-4.1.8.16.(2).) **3)** The shear and overturning resistances of the *foundation* determined using a bearing stress equal to 1.5 times the factored bearing strength of the *soil* or *rock* and all other resistances equal to 1.3 times the factored resistances need not exceed the design forces determined in Sentence 4.1.8.7.(1) using R_dR_o = 1.0, except that the factor of 1.3 shall not	Deleted sentence (1) and Inserted new sentences (1),(2),(3) and (4).

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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*. **4)** At sites where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, *basement* walls shall be designed to resist earthquake lateral pressures from backfill or natural ground. (See Appendix A.) **5)** At sites where $I_EF_aS_a(0.2)$ 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 Appendix 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. **6)** Each segment of a tie between elements that is required by Clauses (3)(a) or (5)(b) shall be designed to carry by tension or compression a horizontal force at least equal to the greatest factored *pile* cap or column vertical load in the elements it connects, multiplied by a factor of 0.10 $I_EF_aS_a(0.2)$, unless it can be demonstrated that equivalent restraints can be provided by other means. (See Appendix A.) **7)** The potential for liquefaction of the *soil* and its consequences, such as significant ground displacement and loss of *soil* strength and stiffness, shall be evaluated based on the ground motion parameters referenced in Subsection 1.1.3. and shall be taken into account in the design of the structure and its *foundations*. (See Appendix A.)	apply to the portion of the resistance to uplift or overturning resulting from gravity loads. **4)** A *foundation* is permitted to have a factored overturning resistance less than the lateral load overturning capacity of the supported SFRS, provided the following requirements are met: **a)** neither the *foundation* nor the supported SFRS are constrained against rotation, and **b)** the design overturning moment of the *foundation* is **i)** not less than 75% of the overturning capacity of the supported SFRS, and **ii)** not less than that determined in Sentence 4.1.8.7.(1) using $R_dR_o = 2.0$. (See Note A-4.1.8.16.(4).) **5)** ~~2)~~The design of *foundations* shall be such that they are capable of transferring earthquake loads and effects between the *building* and the ground without exceeding the capacities of the *soil* and *rock*. **6)** ~~3)~~In cases where $I_EF_aS_a(0.2)$ 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 ~~Appendix A~~ Note A-4.1.8.16.(6)(a)), **b)** *piles*, drilled piers, and *caissons* shall be embedded a minimum of 100 mm 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)** ~~4)~~At sites where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, *basement* walls shall be designed to resist earthquake lateral pressures from backfill or natural ground. (See ~~Appendix A.~~ Note A-4.1.8.16.(7).) **8)** ~~5)~~At sites where $I_EF_aS_a(0.2)$ 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 ~~Appendix A~~ 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. **9)** ~~6)~~Each segment of a tie between elements that is required by Clauses (36)(a) or (~~5~~8)(b) shall be designed to carry by tension or compression a horizontal force at least equal to the greatest factored *pile* cap or column vertical load in the elements it connects, multiplied by a factor of 0.10 $I_EF_aS_a(0.2)$, unless it can be demonstrated that equivalent restraints can be provided by other means. (See ~~Appendix A~~ Note A-.1.8.16.(9).) **10)** ~~7)~~The potential for liquefaction of the *soil* and its consequences, such as significant ground displacement and loss of *soil* strength and stiffness, shall be evaluated based on the ground motion parameters referenced in Subsection 1.1.3.1.1.3., as modified by Article 4.1.8.4., and shall be taken into account in the design of the structure and its *foundations*. (See ~~Appendix A~~Note A-4.1.8.16.(10).)	
4.1.8.17. Site Stability 1) The potential for slope instability and its consequences, such as slope	**4.1.8.17. Site Stability** **1)** The potential for slope instability and its consequences, such as slope displacement, shall be evaluated based on site-specific material properties and ground motion	

<table>
<tr><th>ABC 2014</th><th>NBC(AE) 2019</th><th>Comments</th></tr>
<tr>
<td>displacement, shall be evaluated based on site-specific material properties and ground motion parameters referenced in Subsection 1.1.3. and shall be taken into account in the design of the structure and its <i>foundations</i>. (See Appendix A.)</td>
<td>parameters referenced in Subsection ~~1.1.3.~~1.1.3., as modified by Article 4.1.8.4., and shall be taken into account in the design of the structure and its <i>foundations</i>. (See ~~Appendix A~~ Note A-4.1.8.17.(1).)</td>
<td></td>
</tr>
<tr>
<td>4.1.8.18. Elements of Structures, Non-structural Components and Equipment
(See Appendix A.)
1) Except as provided in Sentences (2) and (8), elements and components of <i>buildings</i> described in Table 4.1.8.18. and their connections to the structure shall be designed to accommodate the <i>building</i> deflections calculated in accordance with Article 4.1.8.13. and the element or component deflections calculated in accordance with Sentence (10), and shall be designed for a lateral force, V_p, distributed according to the distribution of mass:
…etc.

2) For <i>buildings</i> other than <i>post-disaster buildings</i>, where $I_EF_aS_a(0.2)$ is less than 0.35, the requirements of Sentence (1) need not apply to Categories 6 through 21 of Table 4.1.8.18.
3) 4) 5) 6) 7)
8) 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 <i>building</i> and above the first level above <i>grade</i> 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) for the purpose of applying Clause (e), a ductile connection is one where the body of the connection is capable of dissipating energy through cyclic inelastic behaviour.</td>
<td>4.1.8.18. Elements of Structures, Non-structural Components and Equipment
(See ~~Appendix A.~~Note A-4.1.8.18.)
1) Except as provided in Sentences (2), (7) and (~~8~~16), elements and components of <i>buildings</i> described in Table 4.1.8.18. and their connections to the structure shall be designed to accommodate the <i>building</i> deflections calculated in accordance with Article 4.1.8.13. and the element or component deflections calculated in accordance with Sentence (~~10~~9), and shall be designed for a lateral force, V_p, distributed according to the distribution of mass:

$$V_p = 0.3F_aS_a(0.2)I_ES_pW_p$$

where
Fa = as defined in ~~Table~~ Sentence 4.1.8.4.~~B.~~(7),
……
Full TABLE NOT SHOWN HERE – SEE NBC(AE) 2019
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)
<table>
<tr><th>Category</th><th>Part or Portion of <i>Building</i></th><th>C_p</th><th>A_r</th><th>R_p</th></tr>
<tr><td>1</td><td>All exterior and interior walls except those in Category 2 or 3~~(1)~~</td><td>1.00</td><td>1.00</td><td>2.50</td></tr>
<tr><td>2</td><td>Cantilever parapet and other cantilever walls except retaining walls~~(1)~~</td><td>1.00</td><td>2.50</td><td>2.50</td></tr>
<tr><td>3</td><td>Exterior and interior ornamentations and appendages~~(1)~~</td><td>1.00</td><td>2.50</td><td>2.50</td></tr>
<tr><td>…</td><td>…</td><td>…</td><td>…</td><td>…</td></tr>
<tr><td>11</td><td>Machinery, fixtures, equipment~~,~~ ~~ducts~~ and tanks (including contents)</td><td></td><td></td><td></td></tr>
<tr><td></td><td>that are rigid and rigidly connected~~(3)~~</td><td>1.00</td><td>1.00</td><td>1.25</td></tr>
<tr><td></td><td>that are flexible or flexibly connected~~(3)~~</td><td>1.00</td><td>2.50</td><td>2.50</td></tr>
<tr><td>12</td><td>Machinery, fixtures, equipment, ducts and tanks (including contents) containing toxic or explosive materials, materials having a <i>flash point</i> below 38°C or firefighting fluids</td><td></td><td></td><td></td></tr>
<tr><td></td><td>that are rigid and rigidly connected~~(3)~~</td><td>1.50</td><td>1.00</td><td>1.25</td></tr>
</table>
</td>
<td>Inserted new sentences (13),(14),(15) and (16).
Remaining Sentences renumbered.</td>
</tr>
</table>

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TABLE NOT SHOWN HERE – SEE ABC 2014

Table 4.1.8.18.

9) 10) 11) 12) 13)

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	that are flexible or flexibly connected~~(3)~~	1.50	2.50	2.50
…	…	…	…	…
15	Pipes, ducts~~, cable trays~~ (including contents)	1.00	1.00	3.00
…	…	…	…	…
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

Notes to Table 4.1.8.18.:

(1) See ~~Sentence~~ Note A-Table 4.1.8.18.~~(8).~~

(2) See Sentence ~~4.1.8.18.~~(~~9~~8).

(3) See ~~Sentence 4.1.8.18.(4).~~ also ASME A17.1/CSA B44, “Safety Code for Elevators and Escalators.”

(4) See Sentence (13) and Note A-Table 4.1.8.18.

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.

~~3) The values of Cp in Sentence (1) shall conform to Table 4.1.8.18.~~

3) 4) 5) 6)

7) ~~8)~~ 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)** Rp for non-ductile connections, such as adhesives or power-actuated fasteners, shall be taken as 1.0,
- **c)** Rp 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:

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	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)** ~~for the purpose of applying Clause (e),~~ a ductile connection is one where the body of the connection is capable of dissipating energy through cyclic inelastic behaviour. **8) 9) 10) 11) 12)** **13)** Free-standing steel pallet storage racks are permitted to be designed to resist earthquake effects using rational analysis, provided the design achieves the minimum performance level required by Subsection 4.1.8. (See Note A-4.1.8.18.(13).) **14)** Except as provided in Sentence (15), the relative displacement of glass in glazing systems, Dfallout, shall be equal to the greater of a) $D_{fallout} \geq 1.25 I_E D_p$, where $D_{fallout}$ .... = relative displacement at which glass fallout occurs, and D_p .... = relative earthquake displacement that the component must be designed to accommodate, calculated in accordance with Article 4.1.8.13. and applied over the height of the glass component, or b) 13 mm. (See Note A-4.1.8.18.(14) and (15).) **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: $D_{clear} = 2C_1 (1 + h_p C_2/(b_p C_1))$ where D_{clear} = relative horizontal displacement measured over the height of the glass panel, which causes initial glass-to-frame contact, C_1 = average of the clearances on both sides between the vertical glass edges and the frame, h_p = height of the rectangular glass panel,	

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	C_2 = averages of the top and bottom clearances between the horizontal glass edges and the frame, and b_p = width of the rectangular glass panel, c) the glass is fully tempered, monolithic, installed in a non-post-disaster *building*, and no part of the glass is located more than 3 m above a walking surface, or d) the glass is annealed or heat-strengthened laminated glass in a single thickness with an interlayer no less than 0.76 mm and captured mechanically in a wall system glazing pocket with the perimeter secured to the frame by a wet, glazed, gunable, curing, elastomeric sealant perimeter bead of 13 mm minimum glass contact width. (See Note A-4.1.8.18.(14) and (15).) **16)** For structures with supplemental energy dissipation, the following criteria shall apply: **a)** the value of Sa(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 Fa used in Sentence (1) shall be 1.	
	4.1.8.19. Seismic Isolation	Inserted new Article.
	4.1.8.20. Seismic Isolation Design Provisions	Inserted new Article.
	4.1.8.21. Supplemental Energy Dissipation	Inserted new Article.
	4.1.8.22. Supplemental Energy Dissipation Design Considerations	Inserted new Article.
4.2.4.1. Design Basis **1) 2) 3) 4) 5)**	**4.2.4.1. Design Basis** **1) 2) 3) 4) 5)** **6)** Communication, interaction and coordination between the *designer* and the *registered engineering professional* responsible for the geotechnical aspects of the *project* shall take place to a degree commensurate with the complexity and requirements of the *project*.	Inserted new sentence (6).
4.3.4.2. Design Basis for Cold-Formed Steel **1)** *Buildings* and their structural members made of cold-formed steel shall conform to CAN/CSA-S136, "North American Specification for the Design of Cold-Formed Steel Structural Members." (See Appendix A.)	**4.3.4.2. Design Basis for Cold-Formed Steel** **1)** *Buildings* and their structural members made of cold-formed steel shall conform to ~~CAN/~~CSA- S136, "North American Specification for the Design of Cold-Formed Steel Structural Members~~."~~ (~~See~~ using the Appendix ~~A~~ B provisions applicable to Canada)." (See Note A-4.3.4.2.(1).)	
4.3.6.1. Design Basis for Glass **1)** Glass used in *buildings* shall be designed in conformance with CAN/CGSB-12.20-M, "Structural Design of Glass for Buildings."	**4.3.6.1. Design Basis for Glass** **1)** Glass used in *buildings* shall be designed in conformance with a) CAN/CGSB-12.20-M, "Structural Design of Glass for Buildings~~."~~," using an adjustment factor on the wind load, W, of not less than 0.75, or b) ASTM E 1300, "Determining Load Resistance of Glass in Buildings," using an adjustment factor on the wind load, W, of not less than 1.0. (See Note A-4.3.6.1.(1).)	
4.4.2.1. Design Basis for Parking Structures **1)** Parking structures shall be designed in conformance with CSA S413, "Parking	**4.4.2.1. Design Basis for Parking Structures and Repair Garages** **1)** Parking structures and *repair garages* shall be designed in conformance with	

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Structures.”	CSA S413, “Parking Structures.” (See Note A-4.4.2.1.(1).)	