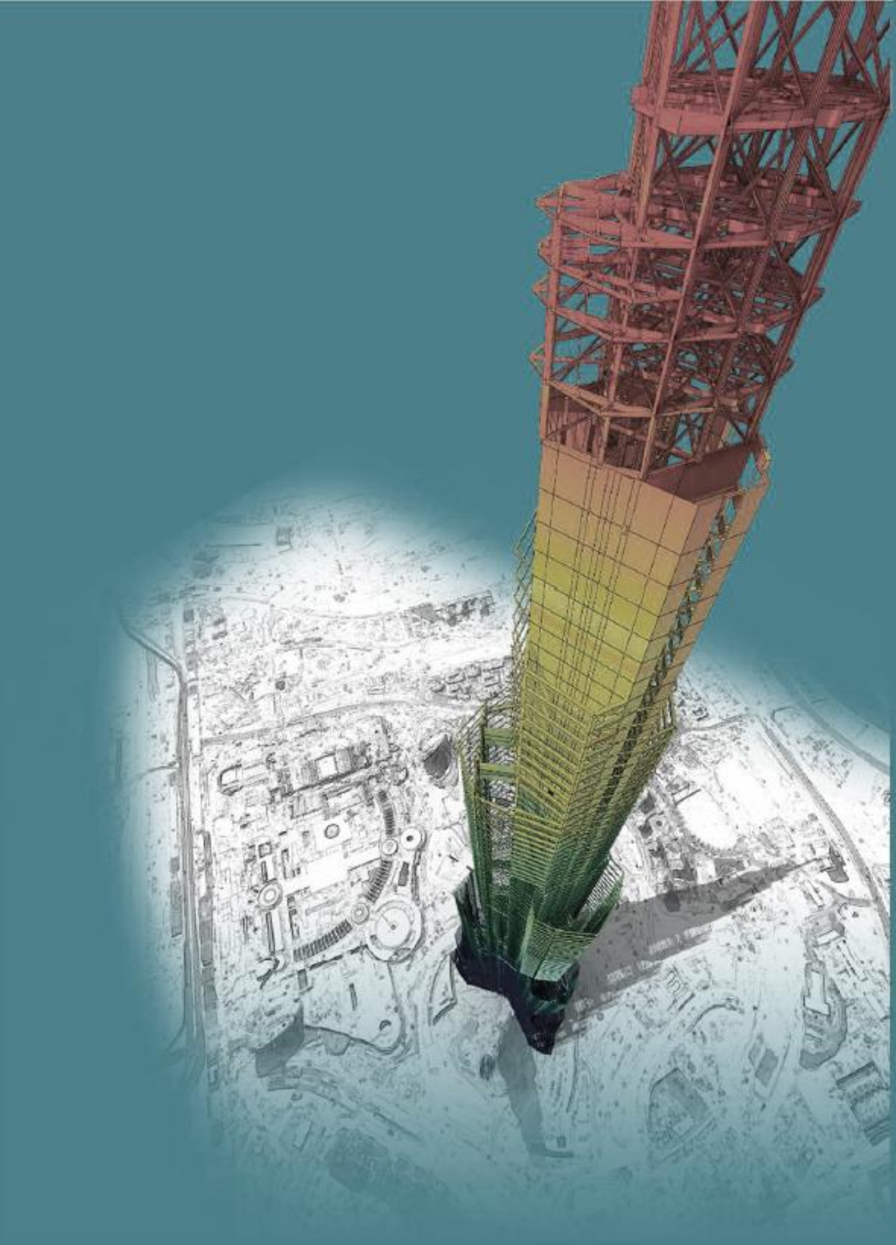
Release Note

Release Date: Jan. 2025

Product Version : GEN 2025 (v1.2)

DESIGN + 2025 (v1.1)



Design System of General Structures

Integrated Design System for Building and General Structures

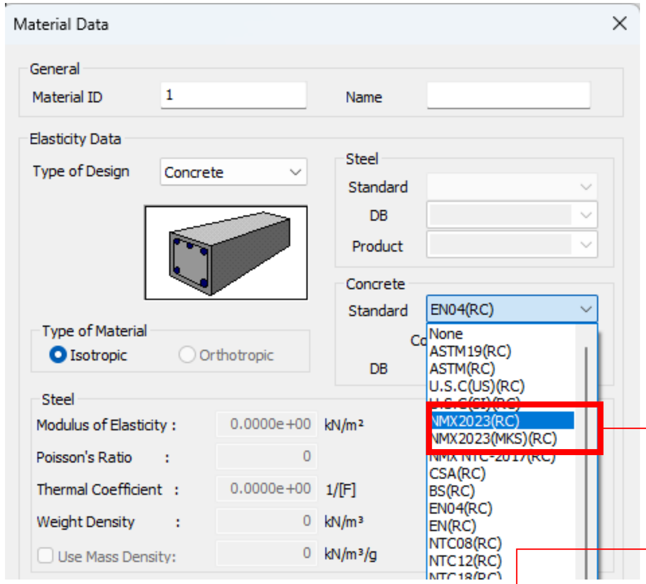
Index

GEN

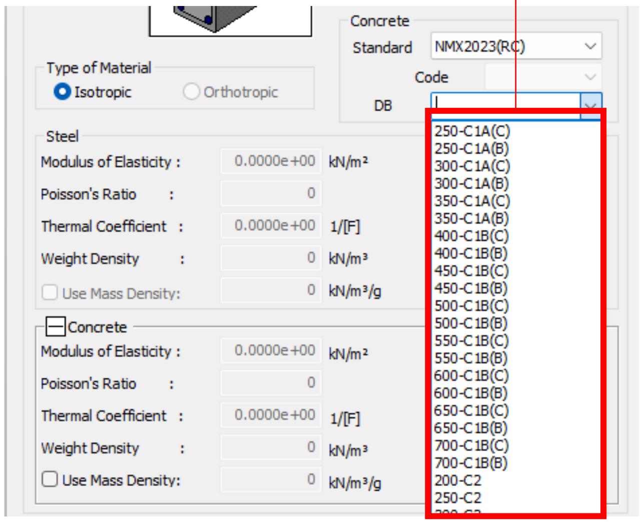
1. Added the rebar size & material DB of NMX 2023 (Mexican design code)

Tabla 2.2.1 – Clases y propiedades de los concretos estructurales convencionales

Requerimiento (inciso de referencia)	Método de ensayo	Concreto Clase 1: 1A	Concreto Clase 1: 1B	Concreto Clase 2
Resistencia a la compresión, f_c' (2.2.6)	NMX-C-083-ONNCCE-2020	$25 \le f_c' < 40$ MPa ($250 \le f_c' < 400$ kg/cm²)	$40 \le f_c' \le 70$ MPa ($400 < f_c' < 700$ kg/cm²)	$20 \le f_c' \le 35$ MPa ($200 < f_c' \le 350$ kg/cm²)
Resistencia media a la tensión, $\bar{f}_t$ (2.2.7)	NMX-C-163-ONNCCE-2019	$0.47\sqrt{f_c'}$, en MPa ($1.5\sqrt{f_c'}$, en kg/cm²)	• Concretos con agregado grueso calizo: $0.53\sqrt{f_c'}$, en MPa ($1.67\sqrt{f_c'}$, en kg/cm²) • Concretos con agregado grueso basáltico: $0.47\sqrt{f_c'}$, en MPa ($1.50\sqrt{f_c'}$, en kg/cm²)	$0.38\sqrt{f_c'}$, en MPa ($1.2\sqrt{f_c'}$, en kg/cm²)
Resistencia media a la tensión por flexión o módulo de rotura, $\bar{f}_f$ (2.2.7)	NMX-C-191-ONNCCE-2015	$0.63\sqrt{f_c'}$, en MPa ($2\sqrt{f_c'}$, en kg/cm²)	• Concretos con agregado grueso calizo: $0.85\sqrt{f_c'}$, en MPa ($2.70\sqrt{f_c'}$, en kg/cm²) • Concretos con agregado grueso basáltico: $0.80\sqrt{f_c'}$, en MPa ($2.54\sqrt{f_c'}$, en kg/cm²)	$0.44\sqrt{f_c'}$, en MPa ($1.4\sqrt{f_c'}$, en kg/cm²)
Peso volumétrico en estado fresco (2.2.2)	NMX-C-162-ONNCCE-2014	> 22 kN/m³ (> 2 200 kg/m³)		19 < peso vol. ≤ 22 kN/m³ (1 900 ≤ peso vol. ≤ 2 200 kg/m³)
Módulo de elasticidad, E_c (2.2.8)	NMX-C-128-ONNCCE-2013	• Concretos con agregado grueso calizo: $4\,400\sqrt{f_c'}$, en MPa ($14\,000\sqrt{f_c'}$, en kg/cm²) • Concretos con agregado grueso basáltico: $3\,500\sqrt{f_c'}$, en MPa ($11\,000\sqrt{f_c'}$, en kg/cm²)	• Concretos con agregado grueso calizo: $2\,700\sqrt{f_c'} + 11\,000$, en MPa ($8\,500\sqrt{f_c'} + 110\,000$, en kg/cm²) • Concretos con agregado grueso basáltico: $2\,700\sqrt{f_c'} + 5\,000$, en MPa ($8\,500\sqrt{f_c'} + 50\,000$, en kg/cm²)	$2\,500\sqrt{f_c'}$, en MPa ($8\,000\sqrt{f_c'}$, en kg/cm²)
Contracción por secado, ε_{cf} (2.2.9)	NMX C 173 ONNCCE-2010	≤ 0.001	≤ 0.0006	≤ 0.002
Coeficiente de flujo plástico, C_f (2.2.10)	ASTM C512/C512M-15	2		4
Aplicaciones		Debe utilizarse en: • Cimentaciones y estructuras del grupo A, B1 y B2. • Estructuras con requerimientos de durabilidad		Es aceptable el uso en estructuras del grupo B2 que cumplan con todo lo siguiente: • Claros no mayores que 4 m • Altura total de no más de 5 m en dos niveles, sobre nivel de banqueta y • Estructuras de no más de 120 m² de construcción.



- **Add Standards for SI & MKS unit**



- **Add DB for Class 1A, Class 1B, and Class 2**
  - "Class 2" Density : 22kN/m³
  - Change in tensile strength and elastic modulus formulas according to class and aggregate type.

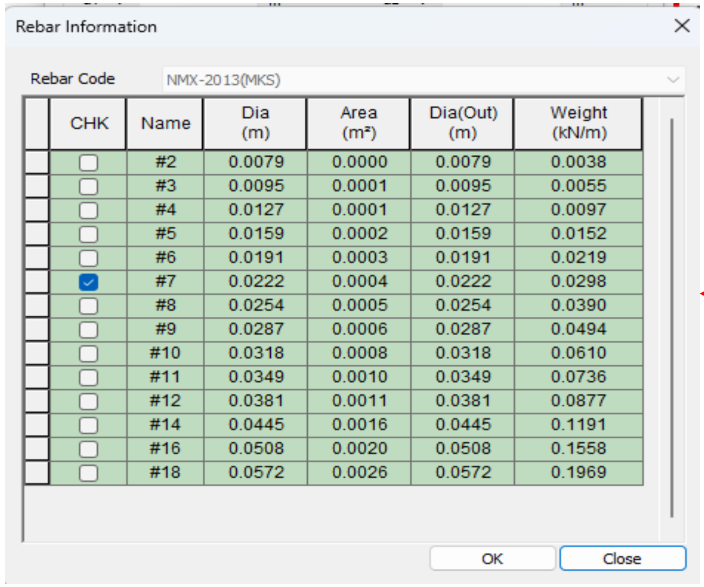
*** DB Name**

300 – C1A(C)

- Aggregate type: (C) : Calizo (B) : Basaltico
- Class Type
- Compression Strength

2. Added the rebar size & material DB of NMX 2023 (Mexican design code)

DB	Ec modulus of elasticity	W weight density	Diameter	Area	Weight
UNIT	stress = F/L^2	density = F/L^3	cm	cm2	kgf/cm
#2	2.00E+06	0.00792	0.79	0.49	0.004
#3	2.00E+06	0.00788	0.953	0.71	0.006
#4	2.00E+06	0.00783	1.27	1.27	0.010
#5	2.00E+06	0.00784	1.588	1.98	0.016
#6	2.00E+06	0.00784	1.905	2.85	0.022
#7	2.00E+06	0.00784	2.223	3.88	0.030
#8	2.00E+06	0.00784	2.54	5.07	0.040
#9	2.00E+06	0.00784	2.865	6.42	0.050
#10	2.00E+06	0.00784	3.18	7.94	0.062
#11	2.00E+06	0.00784	3.49	9.57	0.075
#12	2.00E+06	0.00784	3.81	11.4	0.089
#14	2.00E+06	0.00783	4.45	15.52	0.121
#16	2.00E+06	0.00784	5.08	20.26	0.159
#18	2.00E+06	0.00783	5.72	25.65	0.201



Rebar Information

Rebar Code: NMX-2013(MKS)

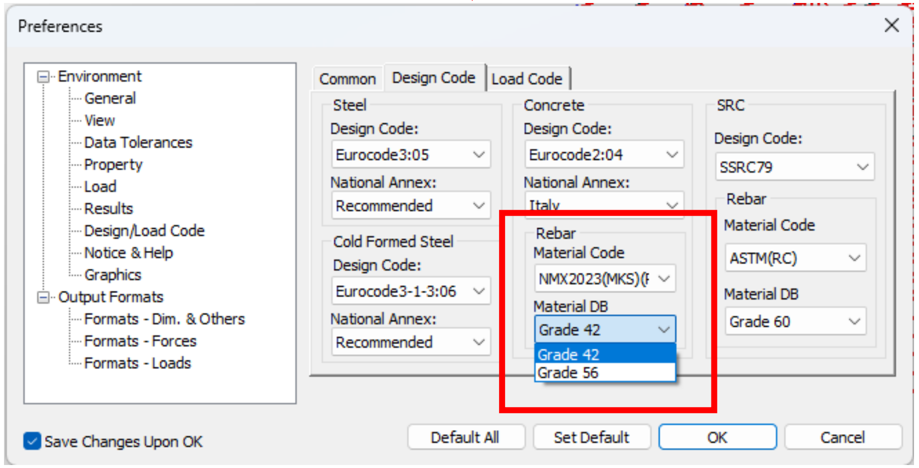
CHK	Name	Dia (m)	Area (m²)	Dia(Out) (m)	Weight (kN/m)
☐	#2	0.0079	0.0000	0.0079	0.0038
☐	#3	0.0095	0.0001	0.0095	0.0055
☐	#4	0.0127	0.0001	0.0127	0.0097
☐	#5	0.0159	0.0002	0.0159	0.0152
☐	#6	0.0191	0.0003	0.0191	0.0219
☑	#7	0.0222	0.0004	0.0222	0.0298
☐	#8	0.0254	0.0005	0.0254	0.0390
☐	#9	0.0287	0.0006	0.0287	0.0494
☐	#10	0.0318	0.0008	0.0318	0.0610
☐	#11	0.0349	0.0010	0.0349	0.0736
☐	#12	0.0381	0.0011	0.0381	0.0877
☐	#14	0.0445	0.0016	0.0445	0.1191
☐	#16	0.0508	0.0020	0.0508	0.1558
☐	#18	0.0572	0.0026	0.0572	0.1969

OK Close

◀ *Rebar Size DB as per NMX 2023*

Tabla 2.4.2.1.2.b – Requisitos de tensión para refuerzo NMX-B-457-CANACERO-2019

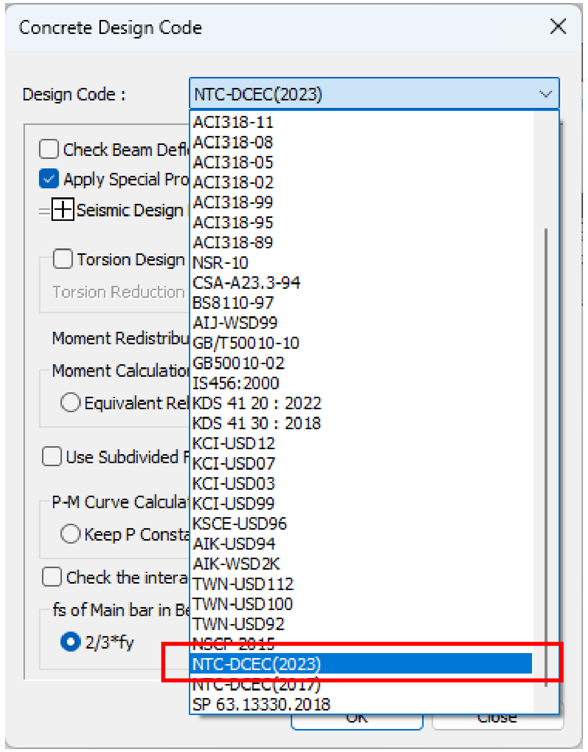
Requisitos	Grado 42	Grado 56
Resistencia mínima a la tensión, MPa (kg/cm²)	550 (5 600)	690 (7 030)
Esfuerzo de fluencia, mínimo, MPa (kg/cm²)	412 (4 200)	550 (5 600)
Esfuerzo de fluencia, máximo, MPa (kg/cm²)	540 (5 500)	675 (6 880)
Relación mínima entre la resistencia a la tensión real y el esfuerzo de fluencia real	1.25	1.25
Alargamiento a la fractura en 200 mm, mínimo, %		
Designación 3, 4, 5, 6	14	12
Designación 7, 8, 9, 10, 11, 12	12	12
Designación 14, 16, 18	10	10



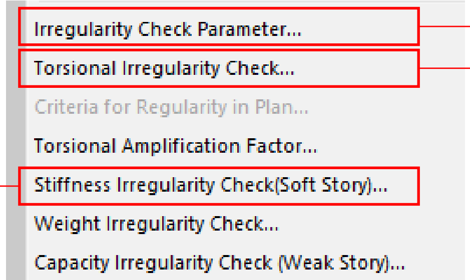
▲ *Rebar Material DB as per NMX 2023*

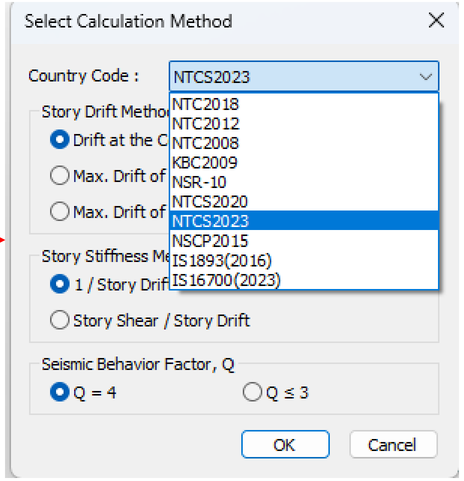
3. Added Design Code for NMX 2023 (Mexican design code)

- **Design Code Setting for NTC-DCEC 2023**



- **Irregularity Check for NTC-DCEC 2023**





◀ *Irregularity Check Parameter (Added NTCS-2023)*

Load Case	Story	Level (mm)	Story Height (mm)	Average Value of Extreme Points				Maximum Value		Remark
				Story Drift (mm)	1.15*Story Drift	1.3*Story Drift	1.4*Story Drift	Node	Story Drift (mm)	
Ex	5F	16000.00	4000.00	0.0696	0.0801	0.0905	0.0975	18	0.0711	Regular
Ex	4F	12000.00	4000.00	0.1739	0.1999	0.2260	0.2434	14	0.1818	Regular
Ex	3F	8000.00	4000.00	1.3834	1.5909	1.7984	1.9367	10	1.3897	Regular
Ex	2F	4000.00	4000.00	0.3723	0.4282	0.4840	0.5212	2	0.3852	Regular
Ex	1F	0.00	4000.00	6.0428	6.9492	7.8557	8.4599	1	6.0506	Regular

▲ *Torsional Irregular Check Table as per NTCS-2023*

Load Case	Story	Level (mm)	Story Height (mm)	Story Drift (mm)	Story Shear Force (kN)	Story Stiffness	Upper Story Stiffness				Remark	Average(Ki+1, Ki-1)		Remark
							1.3K (Upper)	0.85K (Upper)	0.5K (Upper)	0.4K (Upper)		0.5K (Average)	0.4K (Average)	
Ex	5F	16000.00	4000.00	0.0696	3.61	57436.14	-	-	-	-		-	-	
Ex	4F	12000.00	4000.00	0.1739	7.40	23005.77	74666.98	48820.72	28718.07	22974.45	Irregular	15081.90	12065.52	Regular
Ex	3F	8000.00	4000.00	1.3834	10.13	2891.46	29907.50	19554.90	11502.89	9202.31	Irregular	8437.35	6749.88	Strongly Irreg
Ex	2F	4000.00	4000.00	0.3723	11.95	10743.61	3758.90	2457.74	1445.73	1156.58	Irregular	888.35	710.68	Regular
Ex	1F	0.00	4000.00	6.0428	12.86	661.94	13966.69	9132.07	5371.80	4297.44	Irregular	-	-	

▲ *Stiffness Irregular Check Table as per NTCS-2023*

4. Added Eccentricity as per NTCS 2023

- The analysis eccentricity is additionally reflected in the accidental eccentricity.
- The accidental eccentricity ratio can be reflected to increase linearly.

Perform Analysis

Calculate Accidental Ecc. (e_a) by NTCS 2023

$$\left[0.05+\frac{0.05(i-1)}{(n-1)}\right]b_i$$

Ecc_total= Max.[Scale factor [1] x e_s ± e_a]

Input Ecc. Distance in table (Post-Mode)

☑ Consider Eccentricity below G.L

Story	Cross (m)	Along (m)
Roof	0.3000	0.3000
5F	0.3000	0.3000
4F	0.3000	0.3000
3F	0.3000	0.3000
2F	0.3000	0.3000
1F	0.3000	0.3000

- Get Analysis Ecc. (e_s) from Story Eccentricity Table

Start Page | MIDAS/Gen | Result-[Story Eccentricity]

Story	Level (m)	Weight Center		Stiffness Center		Ecc. Dist.	
		X (m)	Y (m)	X (m)	Y (m)	X (m)	Y (m)
Roof	20.00	0.00	0.00	0.00	0.00	0.00	0.00
5F	16.00	0.00	0.00	0.00	0.00	0.00	0.00
4F	12.00	0.00	0.00	0.00	0.00	0.00	0.00
3F	8.00	0.00	0.00	0.00	0.00	0.00	0.00
2F	4.00	0.00	0.00	0.00	0.00	0.00	0.00
1F	0.00	0.00	0.00	0.00	0.00	0.00	0.00

2.3 Efectos de torsión

La excentricidad torsional, e_s, calculada en cada entrepiso, debe tomarse como la distancia entre el centro de torsión del nivel correspondiente y la línea de acción de la fuerza lateral que actúa en él. Para fines de diseño, el momento torsionante debe tomarse, por lo menos, igual a la fuerza lateral que actúa en el nivel multiplicada por la excentricidad que para cada elemento vertical sismo-resistente resulte más desfavorable de las siguientes:

[Example] Scale factor = (1.5 – 1) = 0.5

$$1.5e_s + e_a \quad (2.3.1.a)$$

[Example] Scale factor = (1.0 – 1) = 0

$$e_s - e_a \quad (2.3.1.b)$$

donde e_a es la excentricidad accidental en la dirección de análisis, medida perpendicularmente a la acción sísmica.

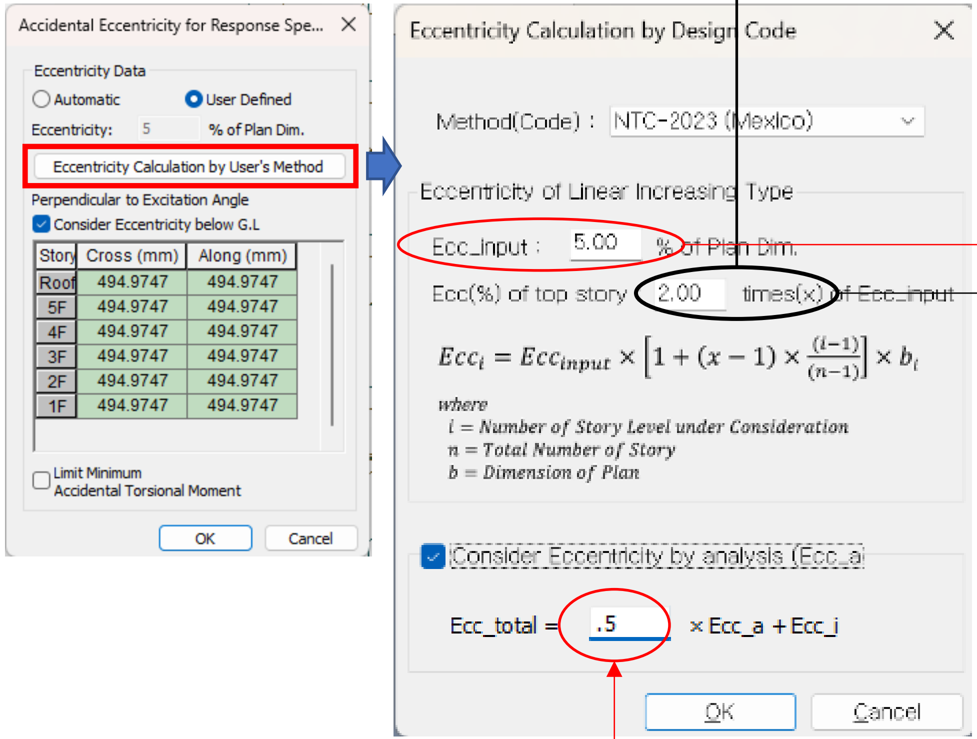
La excentricidad accidental, e_{ai}, en la dirección perpendicular a la de análisis en el *i-ésimo* entrepiso debe calcularse como sigue:

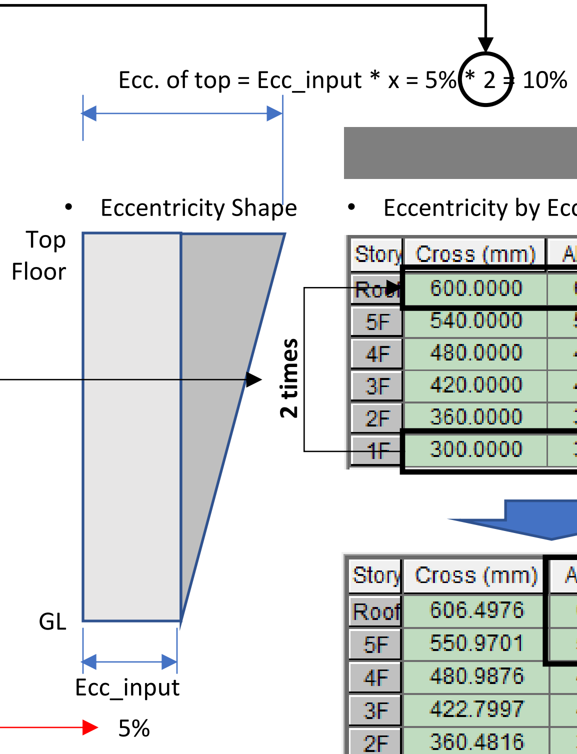
$$\left[0.05+\frac{0.05(i-1)}{(n-1)}\right]b_i \quad (2.3.2)$$

donde b_i es la dimensión del *i-ésimo* piso en la dirección perpendicular a la dirección de análisis; y n, el número de pisos del sistema estructural. Cuando las fuerzas sísmicas se aplican de manera concurrente en 2 direcciones ortogonales, la excentricidad accidental no necesita ser considerada de manera simultánea en ambas direcciones, sino que debe ser aplicada en la dirección que produce el mayor efecto.

[1] e_s (analysis Ecc.) is automatically reflected during an analysis, so it is used only when applying a factor exceeding 1.0.

4. Added Eccentricity as per NTCS 2023 : Eccentricity in Response Spectrum





Example

- Eccentricity by Ecc_i

Story	Cross (mm)	Along (mm)
Roof	600.0000	600.0000
5F	540.0000	540.0000
4F	480.0000	480.0000
3F	420.0000	420.0000
2F	360.0000	360.0000
1F	300.0000	300.0000

- Eccentricity by Ecc_a (Analysis Ecc.)

Story	Level (mm)	Weight Center X (mm)	Weight Center Y (mm)	Stiffness Center X (mm)	Stiffness Center Y (mm)	Ecc. Dist. X (mm)	Ecc. Dist. Y (mm)
Roof	20000.00	3000.00	3000.00	3013.00	2989.53	13.00	10.47
5F	16000.00	3000.00	3000.00	3021.94	3012.38	21.94	12.38
4F	12000.00	3000.00	3000.00	3001.98	2983.70	1.98	16.30
3F	8000.00	3000.00	3000.00	3005.60	2998.62	5.60	1.38
2F	4000.00	3000.00	2999.43	2999.04	2993.14	0.96	6.29
1F	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Story	Cross (mm)	Along (mm)
Roof	606.4976	605.2345
5F	550.9701	546.1913
4F	480.9876	488.1518
3F	422.7997	420.6920
2F	360.4816	363.1472
1F	300.0000	300.0000

Roof & Y-Dir. : 0.5*10.47 + 600 = 605.235

5F & Y-Dir. : 0.5*12.38 + 540 = 546.190

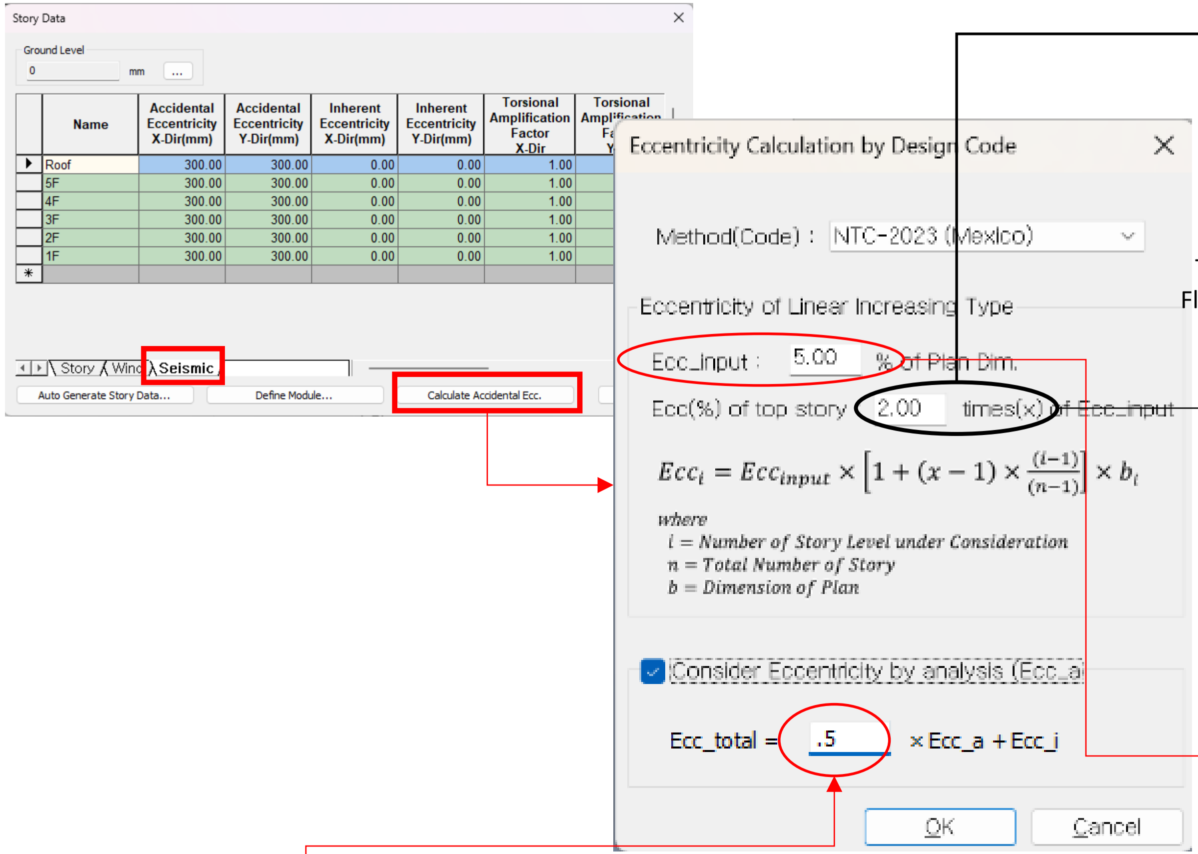
$1.5e_s + e_a$: Input “0.5” in Ecc_total Equation

→ Because $1.0e_s$ (analysis Ecc.) is automatically reflected during an analysis, consider just “0.5” (=1.5 - 1.0)

$e_s - e_a$: Check off “Consider Eccentricity by~~”

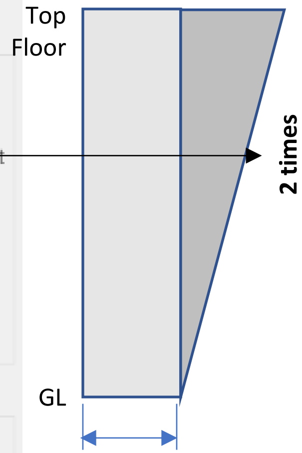
→ Because $1.0e_s$ (analysis Ecc.) is automatically reflected during an analysis, Ecc_a (Analysis Ecc.) should not be considered

4. Added Eccentricity as per NTCS 2023 : Eccentricity for Static Seismic



Ecc. of top = Ecc_input * x = 5% * 2 = 10%

- Eccentricity Shape



Example

- Eccentricity by Ecc_i

Accidental Eccentricity X-Dir(mm)	Accidental Eccentricity Y-Dir(mm)
600.00	600.00
540.00	540.00
480.00	480.00
420.00	420.00
360.00	360.00
300.00	300.00

- Eccentricity by Ecc_a (Analysis Ecc.)

Story	Level (mm)	Weight Center X (mm)	Weight Center Y (mm)	Stiffness Center X (mm)	Stiffness Center Y (mm)	Ecc. Dist. X (mm)	Ecc. Dist. Y (mm)
Roof	20000.00	3000.00	3000.00	3013.00	2989.53	13.00	10.47
5F	16000.00	3000.00	3000.00	3021.94	3012.38	21.94	12.38
4F	12000.00	3000.00	3000.00	3001.98	2983.70	1.98	16.30
3F	8000.00	3000.00	3000.00	3005.60	2998.62	5.60	1.38
2F	4000.00	3000.00	2999.43	2999.04	2993.14	0.96	6.29
1F	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Accidental Eccentricity X-Dir(mm)	Accidental Eccentricity Y-Dir(mm)
606.50	605.23
550.97	546.19
480.99	488.15
422.80	420.69
360.48	363.15
300.00	300.00

Roof & Y-Dir. : 0.5*10.47 + 600 = 605.23

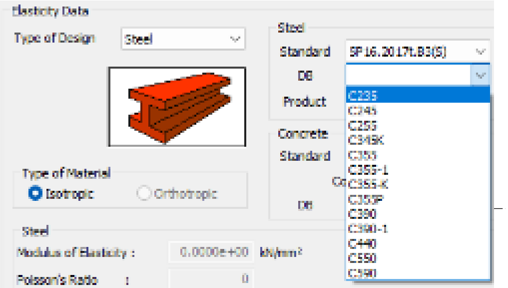
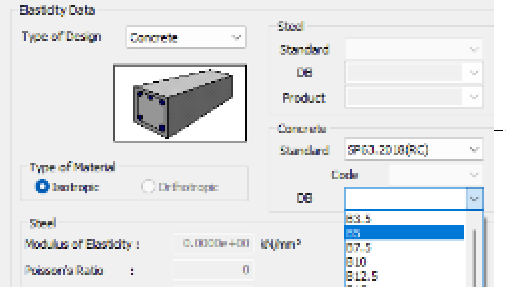
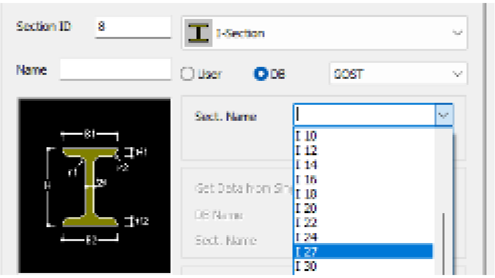
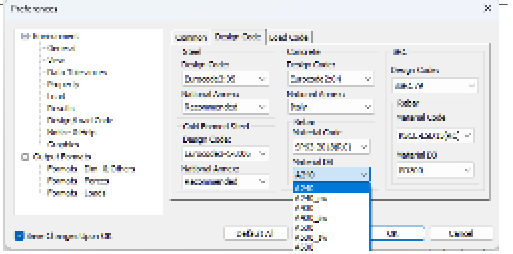
5F & Y-Dir. : 0.5*12.38 + 540 = 546.190

$1.5e_s + e_a$: Input "0.5" in Ecc_total Equation.

→ Because 1.0e_s(analysis Ecc.) is automatically reflected during an analysis, consider just "0.5" (=1.5 - 1.0)

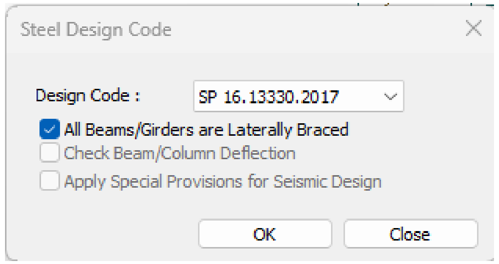
$e_s - e_a$: Check off "Consider Eccentricity by~~"

→ Because 1.0e_s(analysis Ecc.) is automatically reflected during an analysis, Ecc_a (Analysis Ecc.) should not be considered

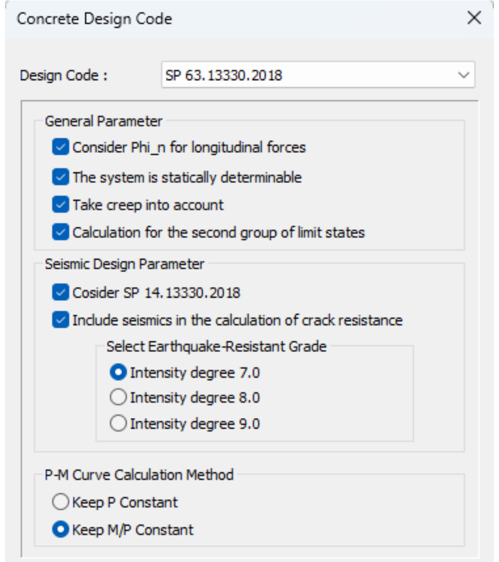
Items	Detail	
Steel Material	• SP 16_2017 (t.B3) : C235 / C245 / C255 / C345K / C355 / C355-1; / C355-K / C355П / C390; / C390-1 / C440 / C550 / C590 / C69 • SP 16_2017 (t.B4) : C255Б / C255Б-1 / C345Б / C345Б-1 / C355Б / C355Б-1 / C390Б / C440Б • SP 16_2017 (t.B5) : C245 / C255 / C345 / C345K / C355 / C355-1 / C390 / C	
RC material	• SP 63-2018 → Heavyweight, Fine grained-gr A type, Fine grained-gr B type (표기 : B + Number + Type) → Lightweight D800, D1000, D1200, D1400, D1600, D1800, D2000 (표기 : LB + Number) → Cellular D500, D600, D700, D800, D900, D1000, D1100, D1200 (표기 : CB + Number)	
Steel Section DB	• I-Shape(GOST 8239 – 89, GOST 26020 – 83, STO ASCHM 20 – 93, GOST 19425-74, GOST P 57837-2017) • Channel Bar (GOST 8240 - 97) • Cold Formed Channel (GOST 8278 - 83) • T-Shape (GOST 8239 – 72, TU 14 - 2 - 24 – 72) • Angle (GOST 8509 – 93, GOST 8510 - 72) • Box (GOST 30245 – 2003, GOST 8639 – 82, GOST P 54157 – 2010) • Pipe (GOST 8732 – 78, GOST 10704-91, GOST 54929-2012) • Solid Circle / Square (GOST 2590-2006, GOST 2591-2006) • Z-Shape (TU 100-180)	
Rebar Material / DB	• Material : SP 63-2018 → A240 / A400 / A500 / A600 / A800 / A1000 / B500 / BP500 / BP1200 / BP1300 / Bp1400 / BP1500 / Bp1600 / K1400 / K1450 / K1500 / K1550 / K1650 / K1750 / K1850 / K1900 • Size : GOST → #4 / #5 / #6 / #7 / #8 / #9 / #10 / #11 / #12 / #13 / #14 / #15 / #16 / #17 / #18 / #19 / #20 / #22 / #25 / #28 / #32 / #36 / #40	

<table>
<tr><th>Items</th><th>Detail</th><th></th></tr>
<tr><td>Steel Design</td><td>

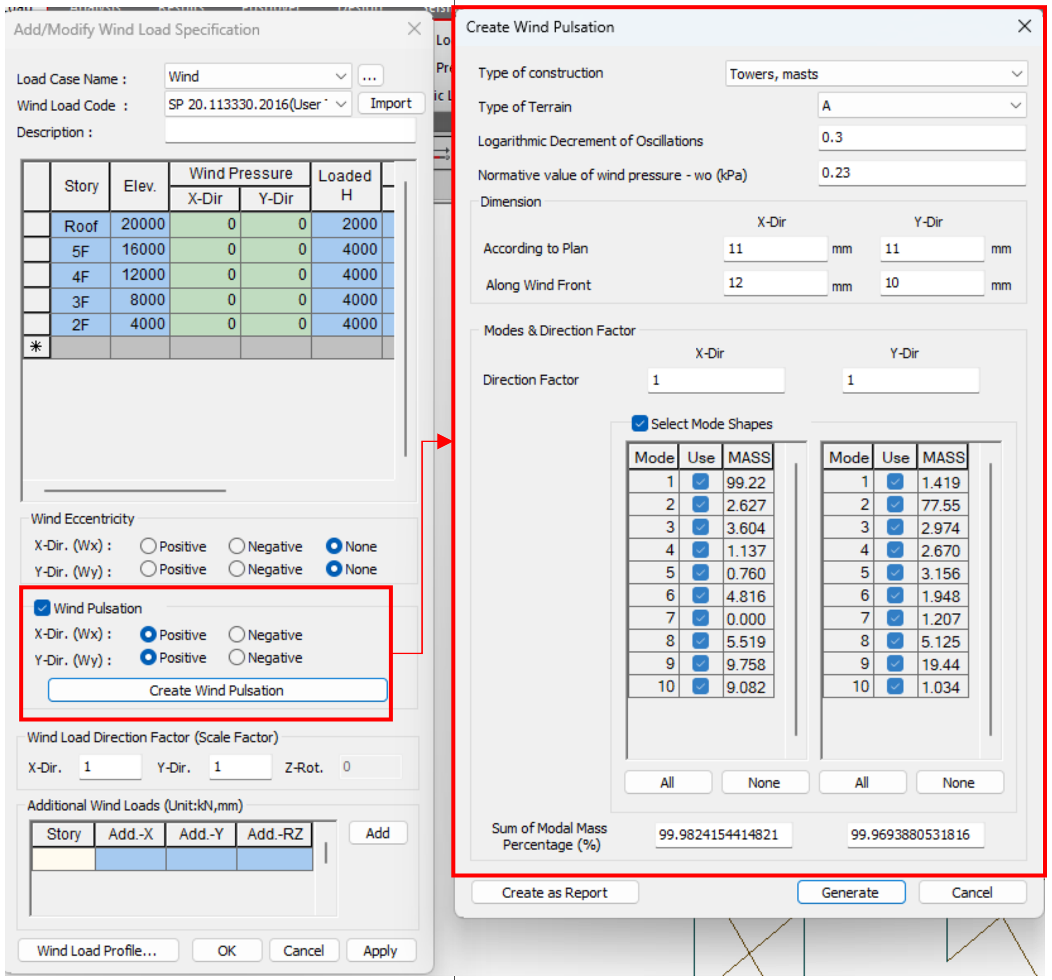
- Design Code : SP 16.13330.2017
- Added Design parameter features
  - Type of stress state
  - Cross Rib Length
  - Local stress
  - Pure bending zone
  - Lateral buckling
- Supported Table design result, Graphic result, Detail result

</td><td>
</td></tr>
<tr><td>RC Design</td><td>

- Design Code : SP 63.13330.2018
- Supported Member Design type : Beam, Column, Truss, Wall, Meshed Slab, Meshed Wall
- Supported Table design result, Graphic result, Detail result

</td><td>
</td></tr>
</table>

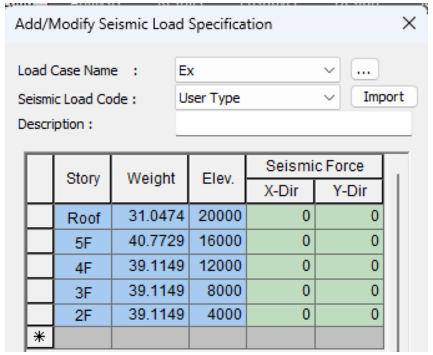
Items	Detail
Wind Load	• Code : SP 20.113330.2016 (User Type) → Input by user (Not auto-generation) • Added “Pulsation Wind Load” → It is available after an eigenvalue analysis.



Items	Detail	

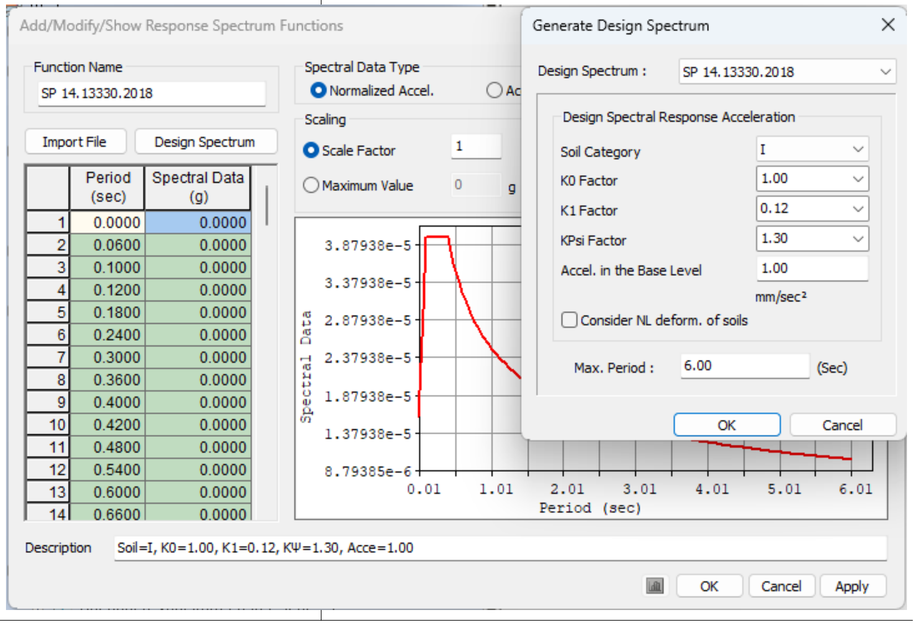
Seismic Load

- User Type → <u>Input by user (Not auto-generation)</u>



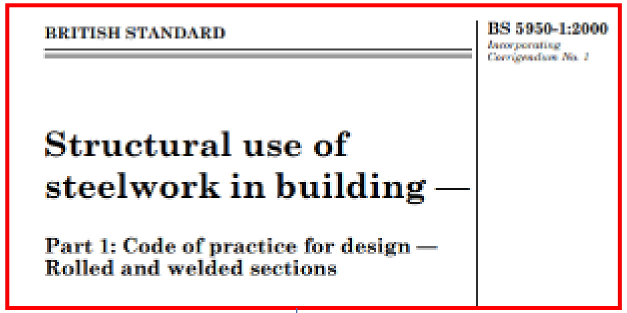
RS Load

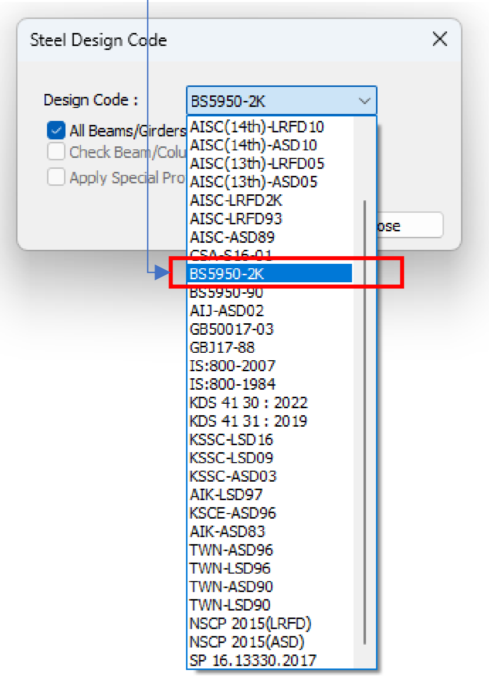
- Code : SP 14.13330.20018



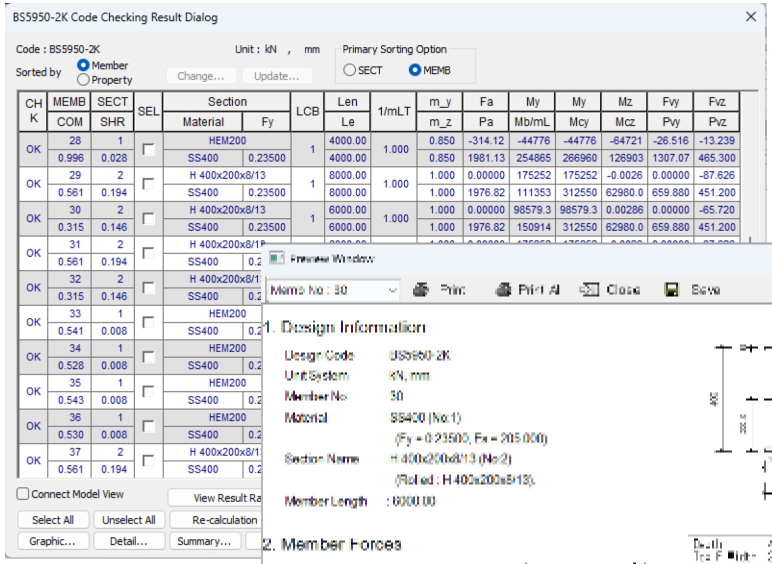
1. Added Steel Design as per BS 5950-2000

- **Design Code Setting for NTC-DCEC 2023**

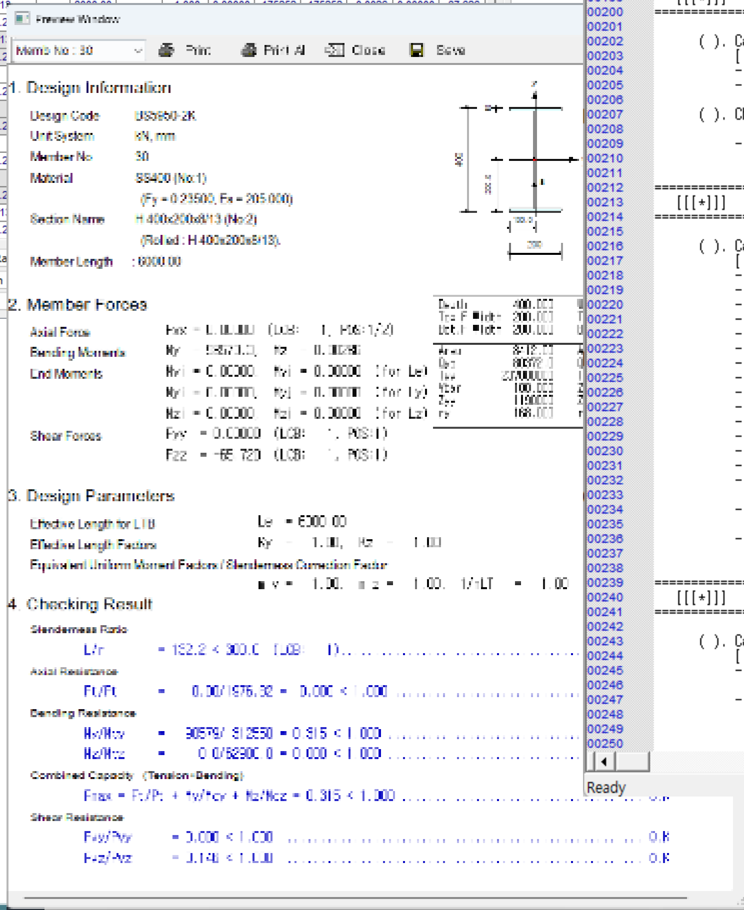




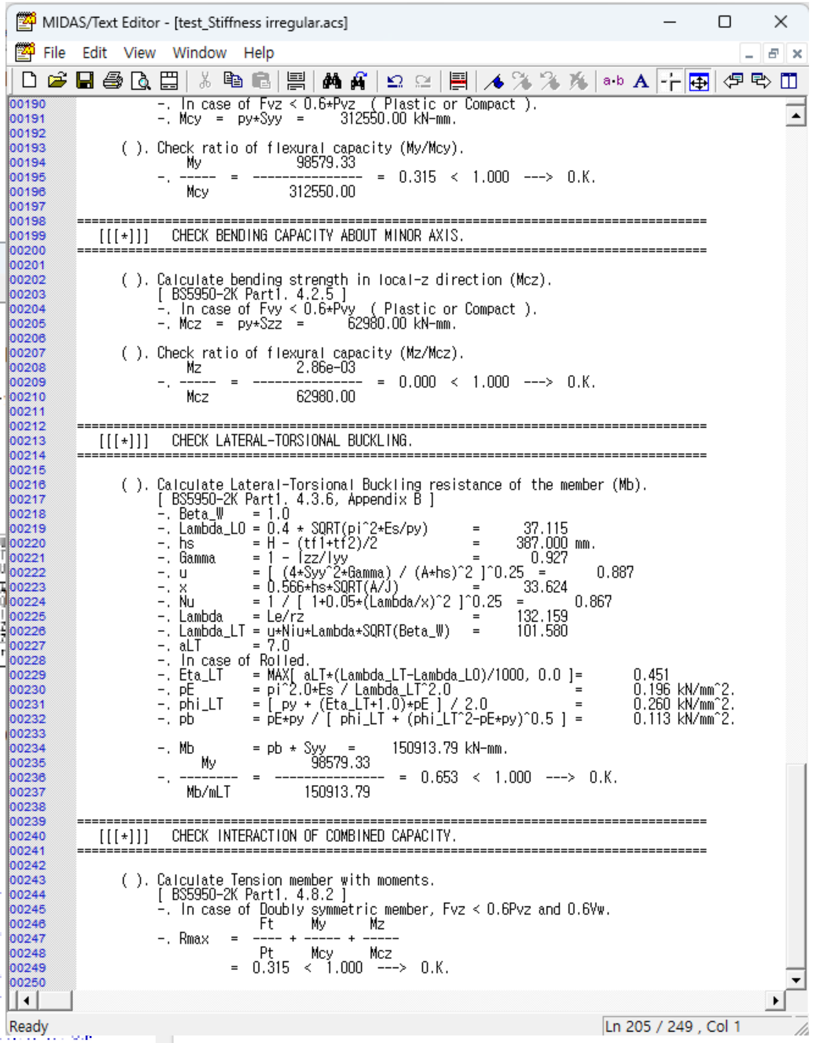
- **Design Result**



▲ *Table Result*



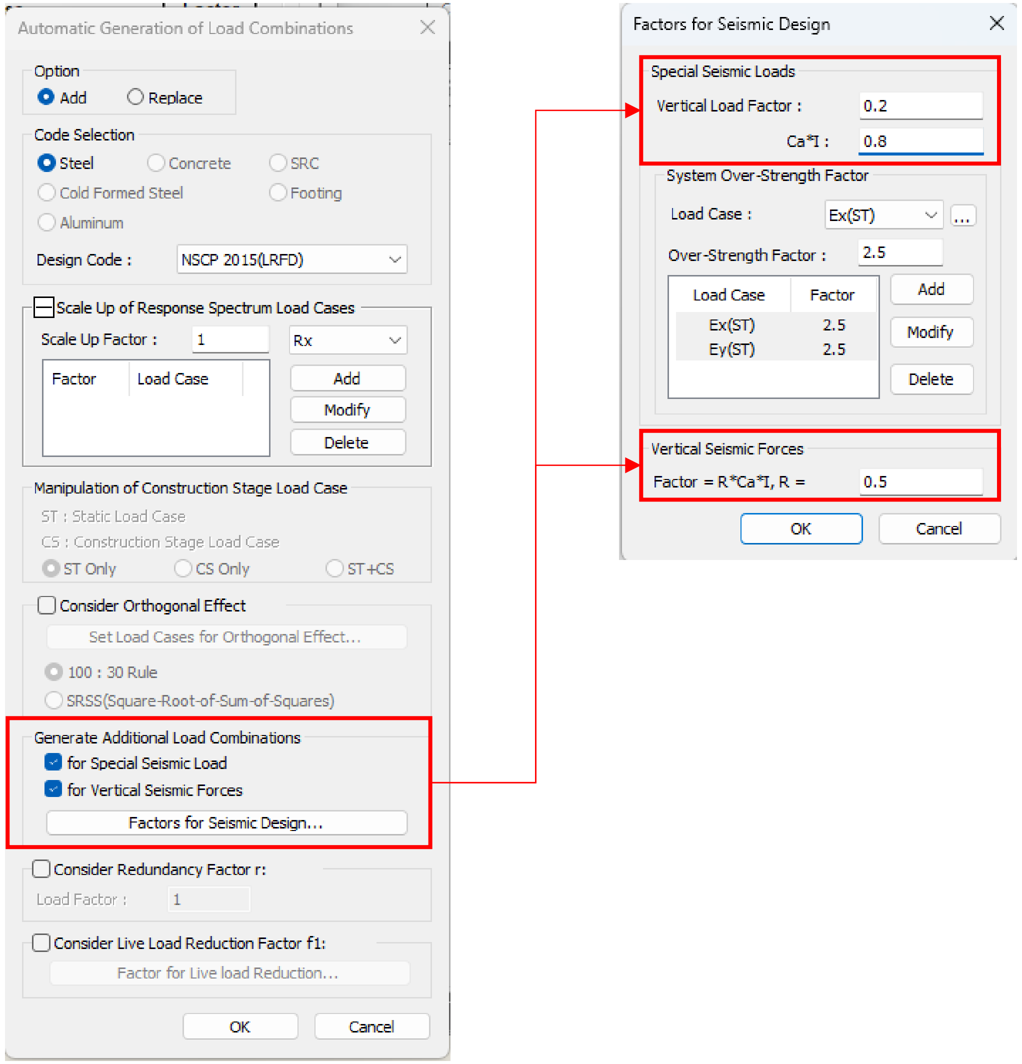
◀ *Graphic Report*



▲ *Detail Report*

1. Added Auto-Generation of Special and Vertical Seismic Load per NSCP 2015

- **Load Combination Setting for NTC-DCEC 2023**



- **Load Combination Table**

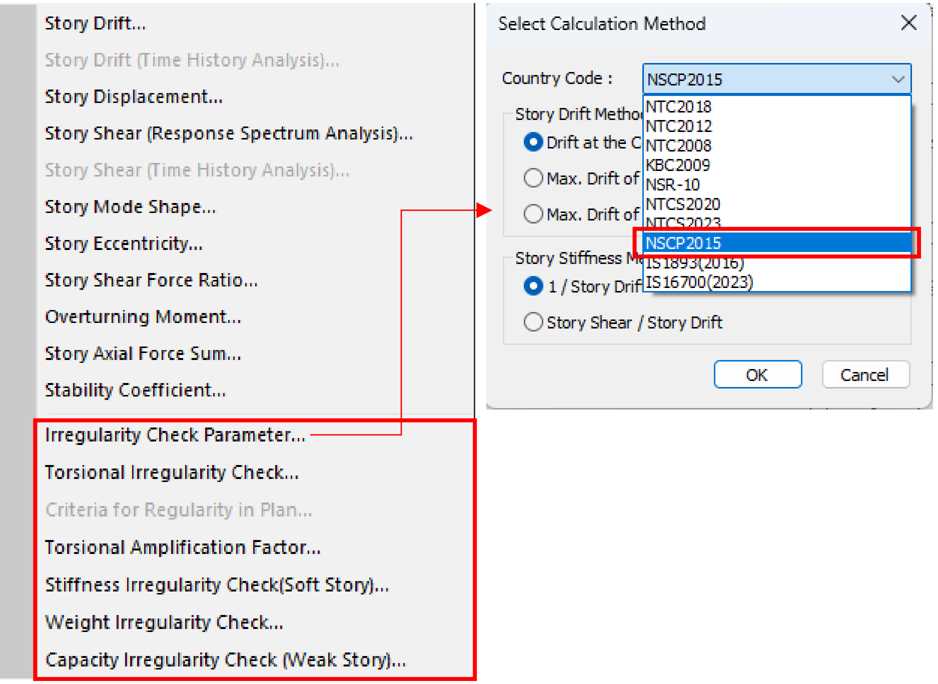
No	Name	Active	Type	Description
1	sLCB42	Special	Add	1.4(D)
2	sLCB43	Special	Add	1.2(D) + 1.6L
3	sLCB44	Special	Add	1.2D + 1.0(2.5)Ex + 1.0(1.0L) + (0.2)(0.8)D
4	sLCB45	Special	Add	1.2D + 1.0(2.5)Ey + 1.0(1.0L) + (0.2)(0.8)D
5	sLCB46	Special	Add	1.2D - 1.0(2.5)Ex + 1.0(1.0L) + (0.2)(0.8)D
6	sLCB47	Special	Add	1.2D - 1.0(2.5)Ey + 1.0(1.0L) + (0.2)(0.8)D
7	sLCB56	Special	Add	0.9D + 1.0(2.5)Ex - (0.2)(0.8)D
8	sLCB57	Special	Add	0.9D + 1.0(2.5)Ey - (0.2)(0.8)D
9	sLCB58	Special	Add	0.9D - 1.0(2.5)Ex - (0.2)(0.8)D
10	sLCB59	Special	Add	0.9D - 1.0(2.5)Ey - (0.2)(0.8)D
11	sLCB68	Vertical	Add	1.4(D)
12	sLCB69	Vertical	Add	1.2(D) + 1.6L
13	sLCB70	Vertical	Add	1.2D + 1.0Ex + 1.0(1.0L)
14	sLCB71	Vertical	Add	1.2D + 1.0Ey + 1.0(1.0L)
15	sLCB72	Vertical	Add	1.2D - 1.0Ex + 1.0(1.0L)
16	sLCB73	Vertical	Add	1.2D - 1.0Ey + 1.0(1.0L)
17	sLCB82	Vertical	Add	0.9D + 1.0Ex
18	sLCB83	Vertical	Add	0.9D + 1.0Ey
19	sLCB84	Vertical	Add	0.9D - 1.0Ex
20	sLCB85	Vertical	Add	0.9D - 1.0Ey
21	sLCB94	Vertical	Add	-(0.4)D + 1.0Ex
22	sLCB95	Vertical	Add	-(0.4)D + 1.0Ey
23	sLCB96	Vertical	Add	-(0.4)D - 1.0Ex
24	sLCB97	Vertical	Add	-(0.4)D - 1.0Ey

Vertical Load Factor * (C_aI) * Dead Load

Vertical Seismic Force → R * C_aI * Dead Load

2. Added Irregularity Check per NSCP 2015

- Torsional Irregularity Check
- Calculation of Torsional Amplification Factory
- Stiffness Irregularity Check (Soft Story)
- Weight Irregularity Check
- Capacity Irregularity Check (Weak Story)



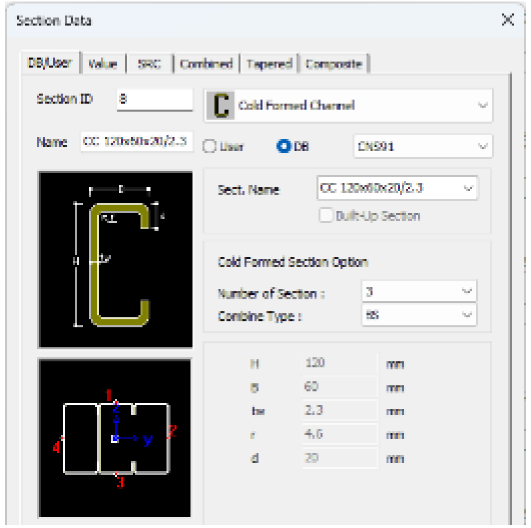
Load Case	Story	Level (mm)	Story Height (mm)	Average Value of Extreme Points: Story Drift (mm)	Average Value of Extreme Points: 1.2*Story Drift (mm)	Maximum Value: Node	Maximum Value: Story Drift (mm)	Remark
Ex	5F	16000.00	4000.00	0.0696	0.0836	18	0.0711	Regular
Ex	4F	12000.00	4000.00	0.1739	0.2086	14	0.1818	Regular
Ex	3F	8000.00	4000.00	1.3834	1.6601	10	1.3897	Regular
Ex	2F	4000.00	4000.00	0.3723	0.4468	2	0.3852	Regular
Ex	1F	0.00	4000.00	6.0428	7.2514	1	6.0506	Regular

Load Case	Story	Level (mm)	Story Height (mm)	Average Displacement of Extreme Points (mm)	Maximum Displacement: Node	Maximum Displacement: Displacement (mm)	Torsional Amplification Factor (Ax)	Note
To obtain right results, the torsional amplification factors in 'Story/Seismic Tab' dialogue box must be all set to '1'.								
Ex	Roof	20000.00	0.00	8.0420	22	8.0784	0.701	
Ex	5F	16000.00	4000.00	7.9724	18	8.0073	0.701	
Ex	4F	12000.00	4000.00	7.7985	14	7.8255	0.699	
Ex	3F	8000.00	4000.00	6.4151	10	6.4358	0.699	
Ex	2F	4000.00	4000.00	6.0428	2	6.0506	0.696	
Ex	1F	0.00	4000.00	0.0000	0	0.0000	0.000	No Diaphragm

Load Case	Story	Level (mm)	Story Height (mm)	Story Drift (mm)	Story Shear Force (kN)	Story Stiffness	Upper Story Stiffness: 0.7Ku1	Upper Story Stiffness: 0.8Ku123	Story Stiffness Ratio	Story Drift Angle Ratio	Remark
Ex	5F	16000.00	4000.00	0.0696	3.61	57436.14	0.00	0.00	0.000	0.000	Regular
Ex	4F	12000.00	4000.00	0.1739	7.40	23005.77	40205.30	0.00	0.572	2.497	Irregular
Ex	3F	8000.00	4000.00	1.3834	10.13	2891.46	16104.04	0.00	0.180	7.956	Irregular
Ex	2F	4000.00	4000.00	0.3723	11.95	10743.61	2024.02	22222.23	0.483	0.269	Regular
Ex	1F	0.00	4000.00	6.0428	12.86	661.94	7520.53	9770.89	0.068	16.230	Irregular

Load Case	Story	Level (mm)	Story Height (mm)	Story Weight (kN)	Adjacent Story Weight: 1.5M(Upper) (kN)	Adjacent Story Weight: 1.5M(Lower) (kN)	Story Weight Ratio	Story Drift Angle Ratio	Remark
Ex	Roof	20000.00	0.00	31.047	0.000	61.159	0.508	0.000	Regular
Ex	5F	16000.00	4000.00	40.773	46.571	58.672	0.875	0.000	Regular
Ex	4F	12000.00	4000.00	39.115	61.159	58.672	0.667	2.497	Regular
Ex	3F	8000.00	4000.00	39.115	58.672	58.672	0.667	7.956	Regular
Ex	2F	4000.00	4000.00	39.115	58.672	0.000	0.667	0.269	Regular
Ex	1F	0.00	4000.00	9.404	58.672	0.000	0.160	16.230	Regular

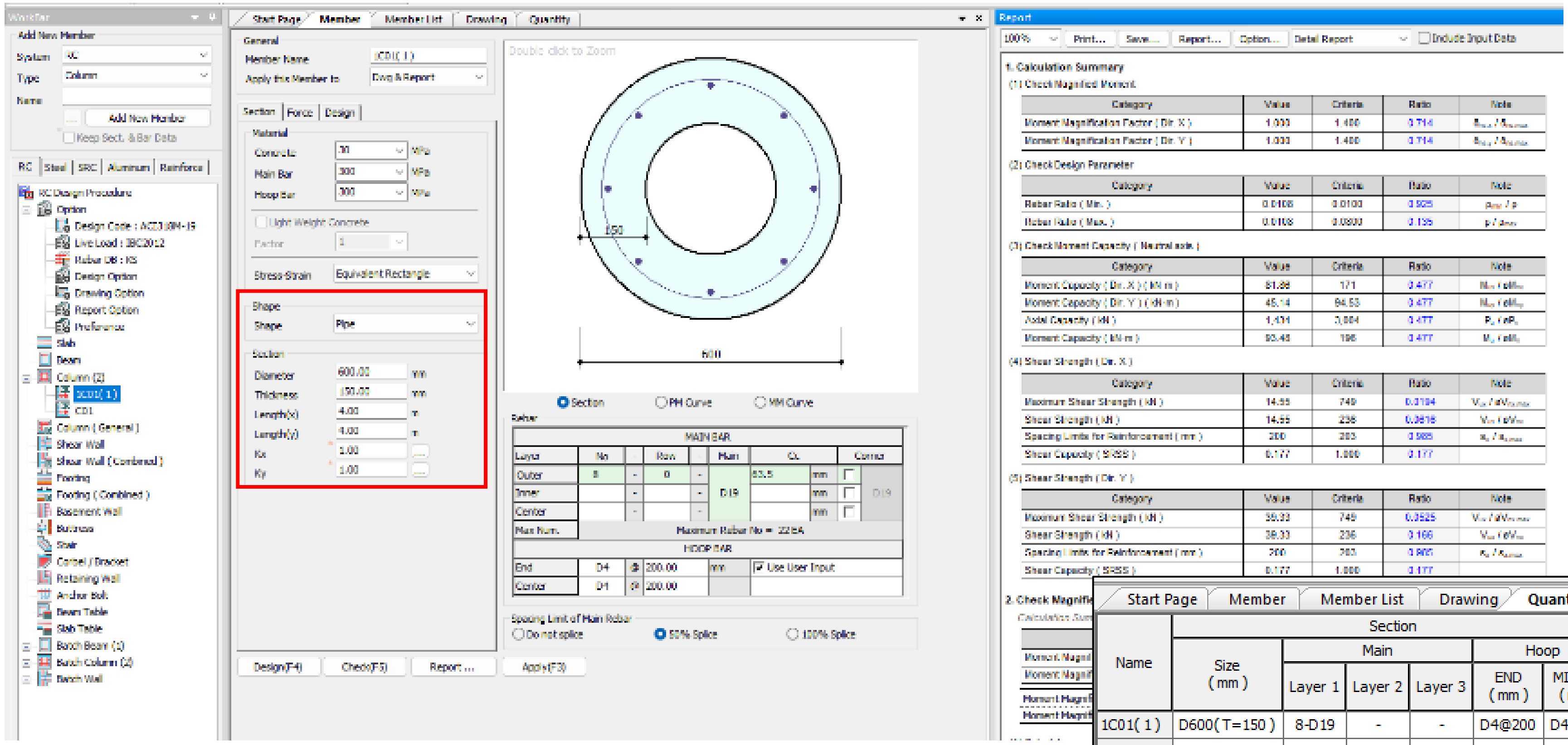
Story	Level (mm)	Story Height (mm)	Angle1 ([deg])	Story Shear Strength1 (kN)	Upper Story Shear Strength1 (kN)	Story Shear Strength Ratio1	Remark1	Angle2 ([deg])	Story Shear Strength2 (kN)	Upper Story Shear Strength2 (kN)	Story Shear Strength Ratio2	Remark2
Angle = 0 [Deg]												
Input angle and press the 'Apply' button to change the angle.			0.00	Apply								
5F	16000.00	4000.00	0.00	2440.7805	0.0000	0.0000	Regular	90.00	5416.9485	0.0000	0.0000	Regular
4F	12000.00	4000.00	0.00	2440.7805	2440.7805	1.0000	Regular	90.00	4840.9998	5416.9485	0.8937	Regular
3F	8000.00	4000.00	0.00	1861.2000	2440.7805	0.7625	Regular	90.00	5416.9485	4840.9998	1.1190	Regular
2F	4000.00	4000.00	0.00	2440.7805	1861.2000	1.3114	Regular	90.00	4840.9998	5416.9485	0.8937	Regular
1F	0.00	4000.00	0.00	1861.2000	2440.7805	0.7625	Regular	90.00	5416.9485	4840.9998	1.1190	Regular

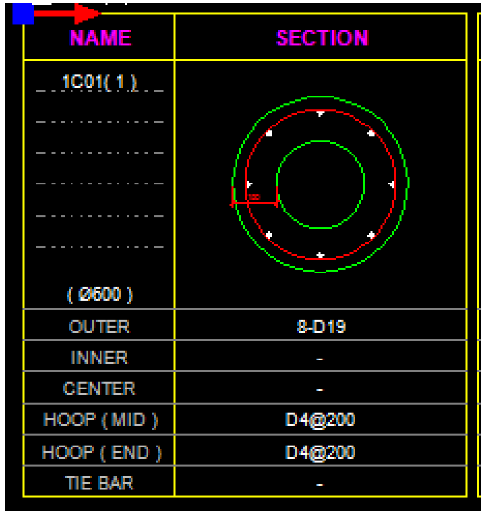
Items	Detail	
Cold Formed Section	• Supported the combined cold-formed section. • <u>Combined No. : 1~4</u> • <u>Combined Type (IS, IW, ES, EW)</u> - I : Web-to-web shape (I-shape) - E : Flange-to-Flange shape (Box-shape) - S : Bolt(Screw) connection - W : Welded connection	
Lateral Load as per DTP (Thailand)	• Fixed the bug for static seismic load & response spectrum • Fixed the bug for static wind load	
Interface	• Add “Master Design”	Only for Italy
IDEA statica Connection	• The member forces by each load combination can be exported to IDEA statica connection.	
Addition of Rebar DB	• Thailand : TIS(SI), TIS(MKS) • Mexico : NMX-2013 (SI), NMX-2013 (MKS) • Russia : GOST-SP, GOST-SNiP, SP 63-2018 • Austria / New Zealand : AS / NZS • South Africa : TMH7	

DESIGN +

1. Added RC pipe column design as per ACI

- **Supported Design Code : ACI 318(M) -11, ACI 318(M) -14, ACI 318(M) -19, KDS 41 20 : 2022, KDS 41 20 : 2018, NSR-10**
- **Supported Features : Column Design, Patch Column Design, Drawing, Quantity**





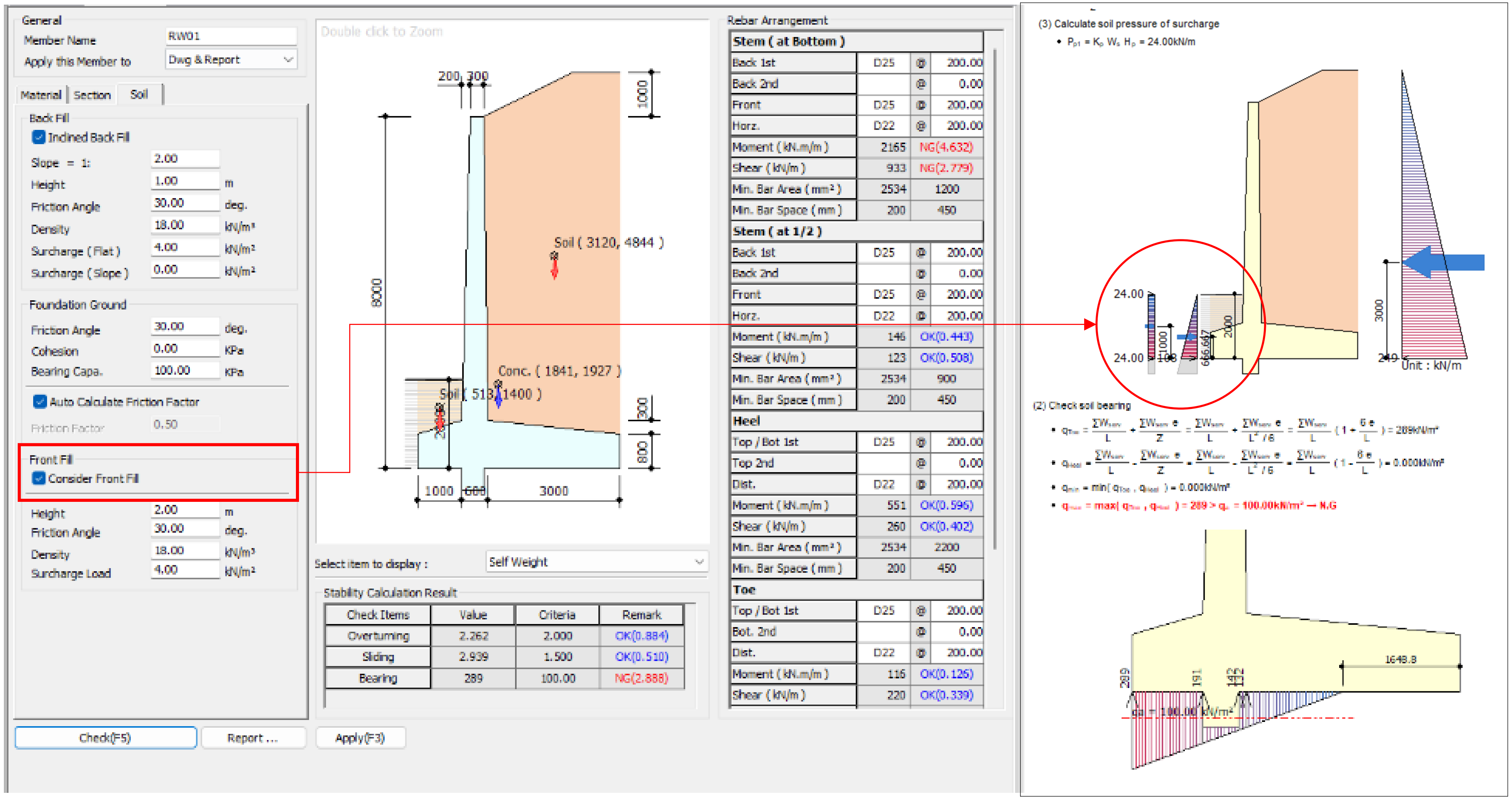
▲ *Drawing : Member List*

Name	Section						Quantity per Unit Length						
	Size (mm)	Main			Hoop		Concrete (m³)	Form (m²)	Main (kN)	Hoop		Sum	
		Layer 1	Layer 2	Layer 3	END (mm)	MIDDLE (mm)				END (kN)	MIDDLE (kN)	END (kN)	MIDDLE (kN)
1C01(1)	D600(T=150)	8-D19	-	-	D4@200	D4@200	0.212	2.827	0.177	0.013	0.013	0.190	0.190
C01	D500	10-D19	-	-	D4@100	D4@100	0.196	1.571	0.221	0.014	0.014	0.234	0.234

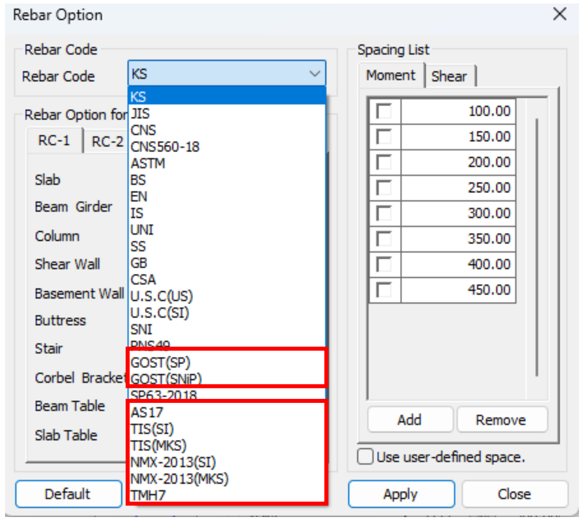
▲ *Quantity*

1. Improved retained wall design as follows

- **Application of load (Active Soil Pressure) by Front Fill**
- **Reinforcement of Detail Report. : Explain the design process and formulas in more detail**



Items	Detail	
Addition of Rebar DB	• Thailand : TIS(SI), TIS(MKS) • Mexico : NMX-2013 (SI), NMX-2013 (MKS) • Russia : GOST-SP, GOST-SNiP, SP 63-2018 • Australia : AS • South Africa : TMH7	



Thank you

Appendix 01

NTC & NMX 2023

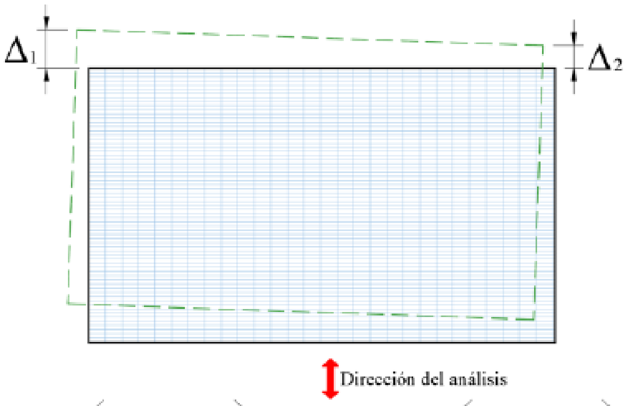
Revised provisions (reflected to midas Gen)

▲ Go to Index

Torsional Irregular Check (NTC 2023)

Irregular

5.2.1.1 It will be considered that a structure is irregular in torsion when in any story there is a point with a lateral displacement that exceeds in more than 15% the average lateral displacement of the extreme points of the story in the analysis direction.



$$1.15\left(\frac{\Delta_1+\Delta_2}{2}\right) < \Delta_1 \leq 1.3\left(\frac{\Delta_1+\Delta_2}{2}\right)$$

Strongly Irregular

5.2.2.1 A structure is strongly irregular in torsion when in any story there is a point with a lateral displacement that exceeds in more than 30% the average lateral displacement of the extreme points of the story in the analysis direction.

$$\Delta_1 > 1.3\left(\frac{\Delta_1+\Delta_2}{2}\right)$$

PBE

5.5.3 If in any story there is a point with a lateral displacement that exceeds in more than 40% the average lateral displacement of the extreme points of the story in the analysis direction, a non-linear time history analysis must to be performed as a review of the structure regardless of his height in order to verify the allowable drift indicated in 4.3 and reduced by 50%.

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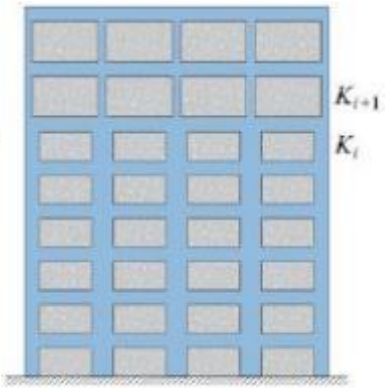
Stiffness Irregular Check (NTC 2023)

Irregular

5.3.2.1 A structure will be considered as Irregular in elevation due to sudden changes in lateral stiffness in height when the lateral stiffness of a story is 15% less than the lateral stiffness of the story immediately above or 30% greater than the stiffness of the story immediately above.

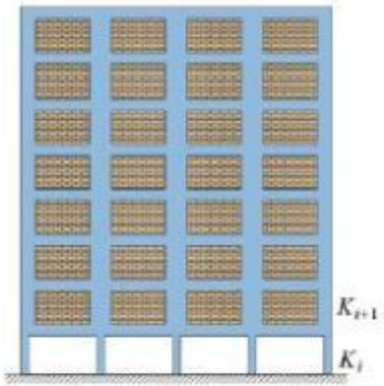
$K_i < 0.85\, K_{i+1}$



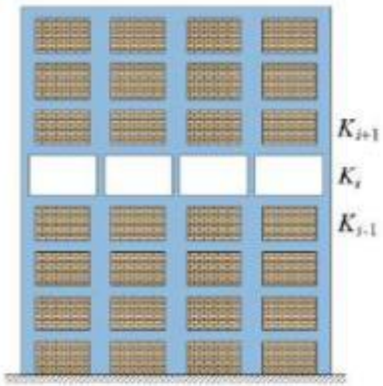
$K_i > 1.3\, K_{i+1}$

Strongly Irregular

5.3.3.1 A structure will be considered as strongly irregular due to sudden reductions in lateral stiffness, that is when the lateral stiffness of a story is less than 50% of the lateral stiffness of the story immediately above or less than 50% of the lateral stiffness average of the stories immediately below and above a story.



$K_i < 0.5\, K_{i+1}$



$K_i < 0.5 \left(\frac{K_{i-1} + K_{i+1}}{2} \right)$

PBE

5.6.3 If the lateral stiffness of any stroy is less than 40% of the lateral stiffness of the story immediately above it or less than 40% of the average lateral stiffness of the stories immediately below and above the story, a non-linear time history analysis must to be performed as a review of the structure regardless of his height in order to verify that at the story with abrupt reduction in lateral stiffness and all stories below the structure will have linear behavior under established ground accelerations according to 7.4.1.

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Added the Mexican design code (NTC 2023)

- *Changed the reference 'f'_c' value to apply to 'β1' formula. : 28MPa → 30MPa*

2017		2023	
3.5		**3.6.1**	
$\beta_1 = 0.85;$	$if\ f'_c \le 28MPa(280kg/cm^2)$	$\beta_1 = 0.85;$	$if\ f'_c \le 30MPa(300kg/cm^2)$
$\beta_1 = 1.05 - \frac{f'_c}{140} \ge 0.65;$	$if\ f'_c > 28MPa$	$\beta_1 = 1.05 - \frac{f'_c}{140} \ge 0.65;$	$if\ f'_c > 30MPa$
$\left(\beta_1 = 1.05 - \frac{f'_c}{1400} \ge 0.65;\right.$	$\left. if\ f'_c > 280kg/cm^2\right)$	$\left(\beta_1 = 1.05 - \frac{f'_c}{1400} \ge 0.65;\right.$	$\left. if\ f'_c > 300kg/cm^2\right)$

- *Changed a limitation formula for design shear force.*

2017	2023
5.3.4 Limitation for design shear force	**5.5.2 Sizing and limits on material strength**
In no case shall the design shear force, V_u, be allowed to exceed the following values:	5.5.2.2 The cross-section dimensions shall be selected to comply with Eq. 5.5.2.2:
a) In beams $V_u < F_R 0.8\sqrt{f'_c}bd$ (5.3.27) $\left(V_u < F_R 2.5\sqrt{f'_c}bd\right)$	$V_u \le F_R\left(V_c + 0.66\sqrt{f'_c}b_w d\right)$ (5.5.2.2) $\left(V_u \le F_R\left(V_c + 2.2\sqrt{f'_c}b_w d\right)\right)$

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Added the Mexican design code (NTC 2023)

- *Changed a formula for a concrete shear resistance.*

2017

5.3.3.1 Concrete resistance to shear force

5.3.3.1a Non-prestressed elements

In beams with a span to total depth ratio, L/h, not less than 5, the shear force taken by the concrete, VcR, shall be calculated using the following criteria:

if p < 0.015

$$V_{cR} = F_R(0.2 + 20p)0.3\sqrt{f_c'}bd \quad \left(V_{cR} = F_R(0.2 + 20p)\sqrt{f_c'}bd\right)$$

if p ≥ 0.015

$$V_{cR} = F_R 0.16\sqrt{f_c'}bd \quad \left(V_{cR} = F_R 0.5\sqrt{f_c'}bd\right) \quad (5.3.2)$$

In any case, V_{cR} must comply with:

$$V_{cR} \le F_R 0.47\sqrt{f_c'}bd \quad \left(V_{cR} \le F_R 1.5\sqrt{f_c'}bd\right) \quad (5.3.2)$$

2023

5.5.3 Resistance to shear force in one-way

5.5.3.1.1 In all concrete elements, except those included in table 6.3.5.4.1 and in 5.5.3.8, a minimum amount of transverse reinforcement shall be placed in the web, even if it is not required for strength, as prescribed in table 6.3.5.4.4. If this minimum amount is available, the shear force resisted by the concrete shall be calculated with either of the equations 5.5.3.1.1.a or 5.5.3.1.1.b taking into account the limitations established in 5.5.3.1.2. The force Pu shall be considered positive if it is compressive, negative if it is tensile, and zero in the case of beams. V_{cR} shall not be less than zero. The factor λ shall be taken from 2.3.3.2.

$$V_{cR} = F_R\left(0.17\lambda\sqrt{f_c'} + \frac{P_u}{6A_g}\right)b_w d$$

(5.5.3.1.1.a)

$$\left(V_{cR} = F_R\left(0.5\lambda\sqrt{f_c'} + \frac{P_u}{6A_g}\right)b_w d\right)$$

$$V_{cR} = F_R\left[0.66\lambda(p)^{1/3}\sqrt{f_c'} + \frac{P_u}{6A_g}\right]b_w d$$

(5.5.3.1.1.b)

$$\left(V_{cR} = F_R\left[0.66\lambda(p)^{1/3}\sqrt{f_c'} + \frac{P_u}{6A_g}\right]b_w d\right)$$

5.5.3.1.2 In Eqs. 5.5.3.1.1.a and 5.5.3.1.1.b limitations a) and b) shall be taken into account:

a) $F_R 0.08\sqrt{f_c'}bd \le V_{cR} \le F_R 0.42\sqrt{f_c'}bd$

$$\left(F_R 0.25\sqrt{f_c'}bd \le V_{cR} \le F_R 1.25\sqrt{f_c'}bd\right)$$

aken greater than 0.05f'$_c$

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Added the Mexican design code (NTC 2023)

- *Changed a formula to calculate 'F_R' : Calculating Fr by 'ε_t' instead of 'c/d_t'*

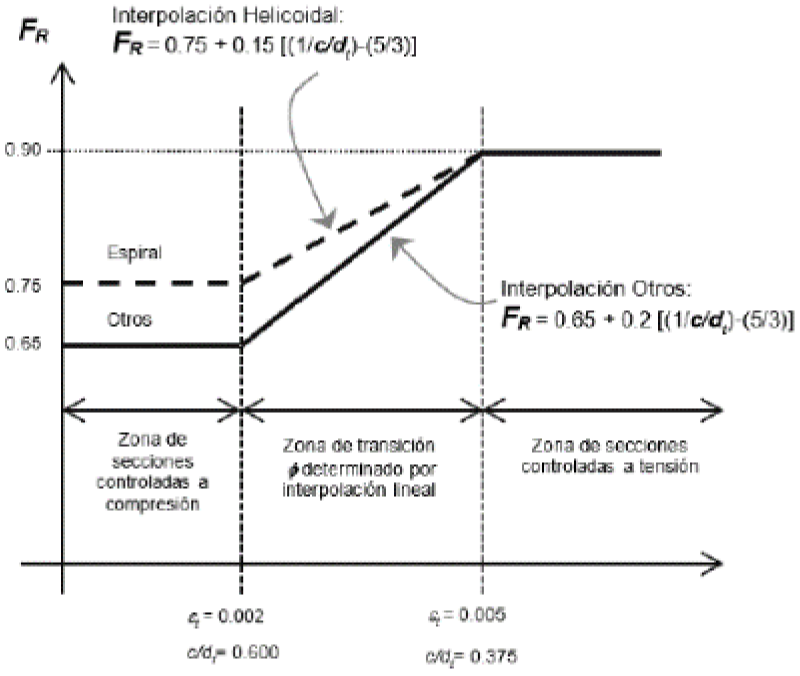

Tabla 3.8.2.2 – Valores del factor de resistencia F_R para momento, fuerza axial, o momento con fuerza axial

Deformación unitaria neta de tensión ε_t	Clasificación	F_R – Tipo de refuerzo transversal: Refuerzo helicoidal (zunchos) que cumple con 14.7.4		F_R – Tipo de refuerzo transversal: Otros	
$\varepsilon_t \le \varepsilon_{ty}$	Controladas por compresión	0.75	a)	0.65	b)
$\varepsilon_{ty} < \varepsilon_t < \varepsilon_{ty}+0.003$	Transición	$0.75 + 0.15\frac{(\varepsilon_t - \varepsilon_{ty})}{(0.003)}$	c)	$0.65 + 0.25\frac{(\varepsilon_t - \varepsilon_{ty})}{(0.003)}$	d)
$\varepsilon_t \ge \varepsilon_{ty}+0.003$	Controlada por tensión	0.90	e)	0.90	f)

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Added the Mexican design code (NTC 2023)

- *Beam design of* ***low ductility structure***

2017	2023
5.1.4 Flaxural reinforcement 5.1.4.1 Minimum reinforcement $A_{s,min} = \frac{0.22\sqrt{f_c'}}{f_y}bd \quad \left(A_{s,min} = \frac{0.7\sqrt{f_c'}}{f_y}b_w d\right)$	**6.3.5 Reinforcement limits** 6.3.5.1 Minimum flexural reinforcement in non-prestressed beams 6.3.5.1.2 $A_{s,min}$ a) $\frac{0.25\sqrt{f_c'}}{f_y}b_w d \quad \left(\frac{0.80\sqrt{f_c'}}{f_y}b_w d\right)$ b) $\frac{1.4}{f_y}b_w d \quad \left(\frac{14}{f_y}b_w d\right)$

6.3.7.6 Transverse reinforcement

Tabla 6.3.7.6.2.2 – Separación máxima de ramas del refuerzo para cortante en vigas de ductilidad baja

V_s requerido		Separación máxima s			
		Vigas no presforzadas		Vigas presforzadas	
		A lo largo del claro	A través del ancho	A lo largo del claro	A través del ancho
$\leq 0.33\sqrt{f_c'}b_w d$ $(\leq 1.1\sqrt{f_c'}b_w d)$	El menor de:	$d/2$	d	$3h/4$	$3h/2$
			600 mm		
$> 0.33\sqrt{f_c'}b_w d$ $(> 1.1\sqrt{f_c'}b_w d)$	El menor de:	$d/4$	$d/2$	$3h/8$	$3h/4$
			300 mm		

Tabla 6.3.5.4.4 – $A_{v,min}$ requerida

Tipo de viga		$A_{v,min}/s$	
No presforzada y presforzada con $A_{ps}f_{se} < 0.4(A_{ps}f_{pu} + A_s f_y)$	El mayor de:	$0.062\sqrt{f_c'}\frac{b_w s}{f_{yt}}$ $\left(0.2\sqrt{f_c'}\frac{b_w s}{f_{yt}}\right)$	a)
		$0.35\frac{b_w s}{f_{yt}}$ $\left(3.5\frac{b_w s}{f_{yt}}\right)$	b)

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Added the Mexican design code (NTC 2023)

- *Column design of low ductility structure - 1*

2017	2023
7.3.1 Geometry The ratio of the largest transverse dimension of a column to the smallest shall not exceed 4. **The smallest transverse dimension shall be at least equal to 200 mm.**	**6.4.2 General requirements** 6.4.2.1.1 The ratio between the largest transverse dimension of a column and the smallest shall not exceed 4. **The smallest transverse dimension shall be at least equal to 250 mm for structures of Subgroup B2 and equal to 300 mm for structures of Group A and Subgroup B1.**
	6.4.3 Limits of reinforcement 6.4.3.2 Minimum shear reinforcement 6.4.3.2.1 A minimum area of shear reinforcement, $\mathbf{A_{v,min}}$, shall be provided in each column that is the greater of a) and b): a) $0.062\sqrt{f'_c}\frac{b_w s}{f_{yt}}$ $\left(0.2\sqrt{f'_c}\frac{b_w s}{f_{yt}}\right)$ b) $0.35\frac{b_w s}{f_{yt}}$ $\left(3.5\frac{b_w s}{f_{yt}}\right)$
	6.4.4.4 Transverse reinforcement

Tabla 6.4.4.4.5.1 – Separación máxima del refuerzo por cortante en columnas de estructuras de ductilidad baja

V_s requerido	Separación máxima, s		
		Columnas no presforzadas	Columnas presforzadas
$\leq 0.33\sqrt{f_c'}b_w d$ $\left(1.1\sqrt{f_c'}b_w d\right)$	La menor de:	$d/2$	$3h/4$
		600 mm	
$> 0.33\sqrt{f_c'}b_w d$ $\left(1.1\sqrt{f_c'}b_w d\right)$	La menor de:	$d/4$	$3h/8$
		300 mm	

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Added the Mexican design code (NTC 2023)

- *Column design of **low ductility structure** - 2*

2017	2023
	6.4.4.4 Transverse reinforcement 6.4.4.4.2.4 At both ends of the column, closed stirrups complying with 14.7.3 shall be provided with spacing so over a distance Lo measured from the face of the node. The spacing so shall not exceed the lesser of a) to d): a) For Grade 42 bars, the lesser of 8db of the thinnest longitudinal bar and 200 mm b) For Grade 56 bars, the lesser of 6db of the thinnest longitudinal bar and 150 mm c) For Grade 70 bars, the lesser of 5db of the thinnest longitudinal bar and 150 mm d) One-fourth of the smallest cross-sectional dimension of the column. The length Lo shall not be less than the maximum value of a) to d): a) One-sixth of the clear height of the column b) The largest cross-sectional dimension of the column c) 600 mm d) H/2 for ground floor or first floor columns subject to earthquake, where H is the clear height of the column.

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Added the Mexican design code (NTC 2023)

- *Wall design of* ***low ductility structure*** *- 1*

2017	2023
7.4.2.4 Shear force a) Shear force by the concrete The shear force, V_{cR} by the concrete in walls will be determined with the following criteria: 1. If the ratio of total height to length, **H_m/L of the wall** or H_s /L of the segment (see item 8.4.2.4) does not exceed **1.5**, equation 7.4.3 will be applied. $V_{cR} = 0.27F_R\sqrt{f'_c}tL$ (7.4.3) $\left(V_{cR} = 0.85F_R\sqrt{f'_c}tL\right)$ 2. If **H_m/L** or H_s /L is **equal to 2.0 or greater**, **equations 5.3.1 or 5.3.2 shall apply**, where b shall be replaced by the thickness of the wall, t; and the effective height of the wall shall be taken as 0.8L. **When H_m/L or H_s /L is between 1.5 and 2.0, it may be linearly interpolated.**	6.5.5.3 Shear force in the plane 6.5.5.3.2 V_u at any horizontal section shall not exceed $0.63\sqrt{f'_c}A_{cv}$ $\left(2\sqrt{f'_c}A_{cv}\right)$ (6.5.5.3.2) 6.5.5.3.3 V_R shall be calculated as: $V_R = F_R\left(\alpha_c\lambda\sqrt{f'_c} + p_t f_{yt}\right)A_{cv}$ (6.5.5.3.3) where: α_c = 0.25, if SI is used (0.80, if MKS is used) for $H_m/L_m \leq 1.5$ α_c = 0.17, if SI is used (0.53, if MKS is used) for $H_m/L_m \leq 2.0$ α_c varies linearly between the values above if $1.5 < H_m/L_m < 2.0$ 6.5.5.3.4 Where walls are subject to net tension forces, α_c in Eq. 6.5.5.3.3 shall be taken as: $\alpha_c = 0.17\left(1 + \frac{P_u}{3.5A_g}\right) \geq 0.0$ (6.5.5.3.4) $\left(\alpha_c = 0.53\left(1 + \frac{P_u}{35A_g}\right) \geq 0.0\right)$ where P_u is negative under tension.

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Added the Mexican design code (NTC 2023)

- *Wall design of* ***low ductility structure*** *- 2*

2017	2023
7.4.2.1 Scope and general requirements The thickness of these walls will not be less than 130 mm.	**6.5.3 Design limits** 6.5.3.2 The minimum thickness of solid walls shall be as indicated in table 6.5.3.2. Smaller thicknesses shall be accepted if their strength and stability are demonstrated by structural analysis and/or laboratory tests.

Tabla 6.5.3.2 – Espesor mínimo de muros macizos

Tipo de muro	Espesor mínimo *t*		
De carga[1]	El mayor de:	140 mm	a)
		0.06 veces la altura no restringida lateralmente	b)
De relleno (no de carga)	El mayor de:	100 mm	c)
		0.04 veces la altura no restringida lateralmente	d)
En muros en contacto con el terreno y cimentaciones	200 mm		e)

[1] Para ser diseñados con el método simplificado de 6.5.5.2.

6.5.4.2.2 En muros en que $H_m/L \geq 2$ se considerará al momento flexionante de diseño a lo largo de Hcr con un valor constante e igual al momento Mu obtenido del análisis en la base del muro. La altura crítica Hcr será igual al valor mayor de L o $M_u/4V_u$. A partir de la altura del muro, Hcr, se usará un diagrama de momentos flexionantes lineal tal que sea paralelo a la línea que une los momentos calculados en la base y en la punta del muro (fig. 6.5.4.2.2). En edificios con muros perimetrales de cimentación, se considerará el momento flexionante de magnitud constante a lo largo del primer nivel del sótano y de la altura crítica, Hcr, medida desde el desplante del muro en la parte superior del cajón hacia arriba.

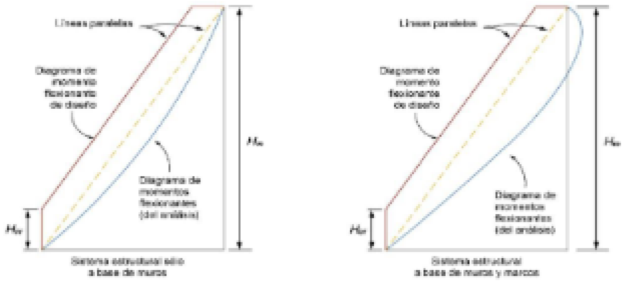

Figura 6.5.4.2.2 – Diagrama de momento flexionante de diseño para un muro de ductilidad baja

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Added the Mexican design code (NTC 2023)

- *Wall design of low ductility structure - 3*

2017	2023

7.4.3.2

b) Shear force taken by the web steel

The amount of reinforcement parallel to the direction of the design shear force, p_m, shall be calculated using the expression

$$p_m = \frac{V_u - V_{cR}}{F_R f_y A_{cm}}$$

(7.4.4)

and that of the reinforcement perpendicular to the design shear force, p_n, with

$$p_n = 0.0025 + 0.5\left(2.5 - \frac{H_m}{L}\right)(p_m - 0.0025)$$

(7.4.5)

where:

$$p_m = \frac{A_{vm}}{s_m t}; \qquad p_n = \frac{A_{vn}}{s_n t};$$

c) Minimum reinforcement, spacing and anchoring of the reinforcement

The reinforcement quantities p_m and p_n will not be less than 0.0025.

6.5.6 Reinforcement limits

6.5.6.1 In case of

$$V_u \le 0.04 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv} \quad \left(V_u \le 0.13 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv}\right)$$

Tabla 6.5.6.1 – Refuerzo mínimo para muros con $V_u \le 0.04 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv}$ $\left(V_u \le 0.13 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv}\right)$

Tipo de muro	Tipo de refuerzo no presforzado	Tamaño de barra o alambre	f_y, MPa (kg/cm^2)	p_l mínima (longitudinal)[1]	p_t mínima (transversal)
Colado en sitio	Barras corrugadas	≤ No. 5	≥ 420 (4 200)	0.0012	0.0020
		> No. 5	≥ 420 (4 200)	0.0015	0.0025
	Alambres soldados, corrugados	≤ 16 mm	Cualquier	0.0012	0.0020

6.5.6.2 In case of

$$V_u > 0.04 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv} \quad \left(V_u > 0.13 F_R \alpha_c \lambda \sqrt{f_c'} A_{cv}\right)$$

a) p_l shall be the greater of the value calculated from **Eq. 6.5.6.2 and 0.0025**, but need not exceed p_t calculated to resist shear from 6.5.5.3.3

$$p_l \ge 0.0025 + 0.5\left(2.5 - \frac{H_m}{L_m}\right)(p_t - 0.0025)$$

(6.5.6.2)

b) $p_t \ge 0.0025$.

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Added the Mexican design code (NTC 2023)

- *Wall design of* ***low ductility structure*** *- 4*

2017	2023
7.4.3.2 c) Minimum reinforcement, spacing and anchoring of the reinforcement The reinforcement will be placed uniformly distributed with a separation of no more than 350 mm.	**6.5.7 Detailing of reinforcement** 6.5.7.1.1 The **spacing s of longitudinal (vertical) reinforcement** in cast-in-place walls shall not exceed the lesser of **3t and 450 mm**. If reinforcement is required to resist shear in the plane of the wall, the spacing of longitudinal reinforcement shall not exceed L/3. 6.5.7.2.1 The **transverse (horizontal) reinforcement spacing s** in cast-in-place walls shall not exceed the lesser of **3t and 450 mm**. If reinforcement is required to resist shear in the plane, s shall not exceed L/5.

- *Joint design of* ***low ductility structure***

: Applied the joint design under low ductility system.

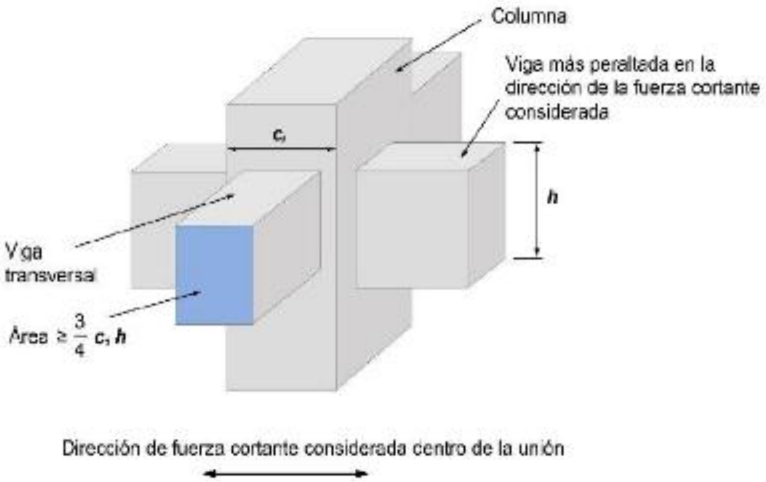

Figura C6.9.2 – Sección mínima de una viga transversal para propósitos de confinamiento de la unión viga-columna (adaptada de ACI CODE-318-19)

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Added the Mexican design code (NTC 2023)

- *Beam design of* ***medium ductility structure***

2017	2023

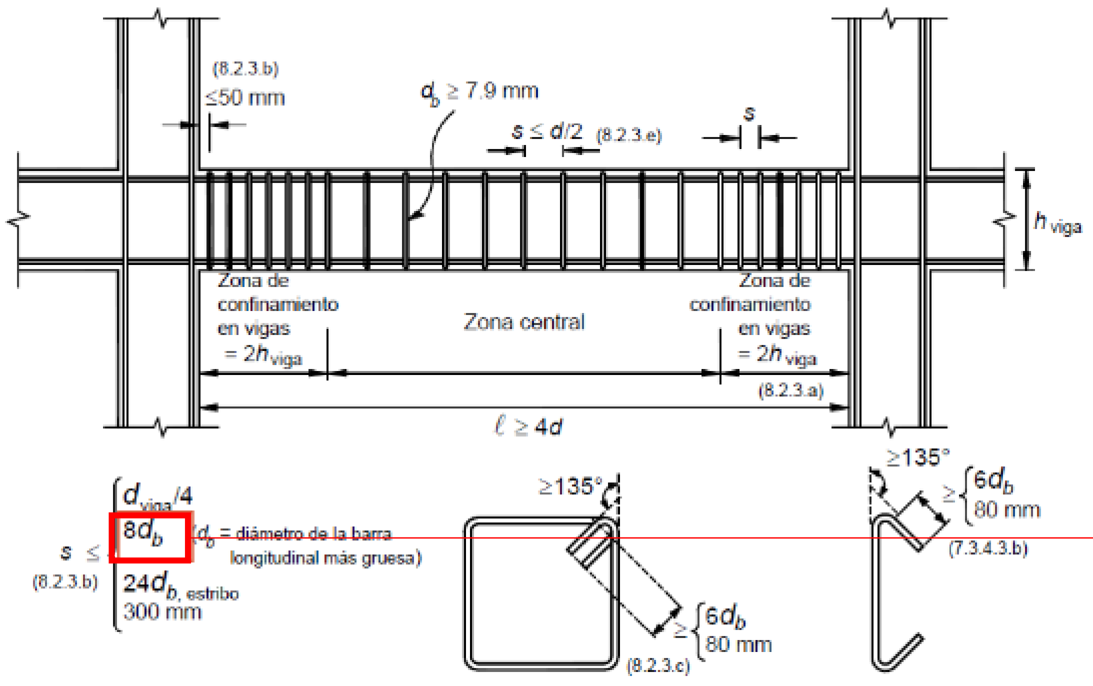

Figura 8.2.1 Detallado de elementos a flexión de ductilidad media

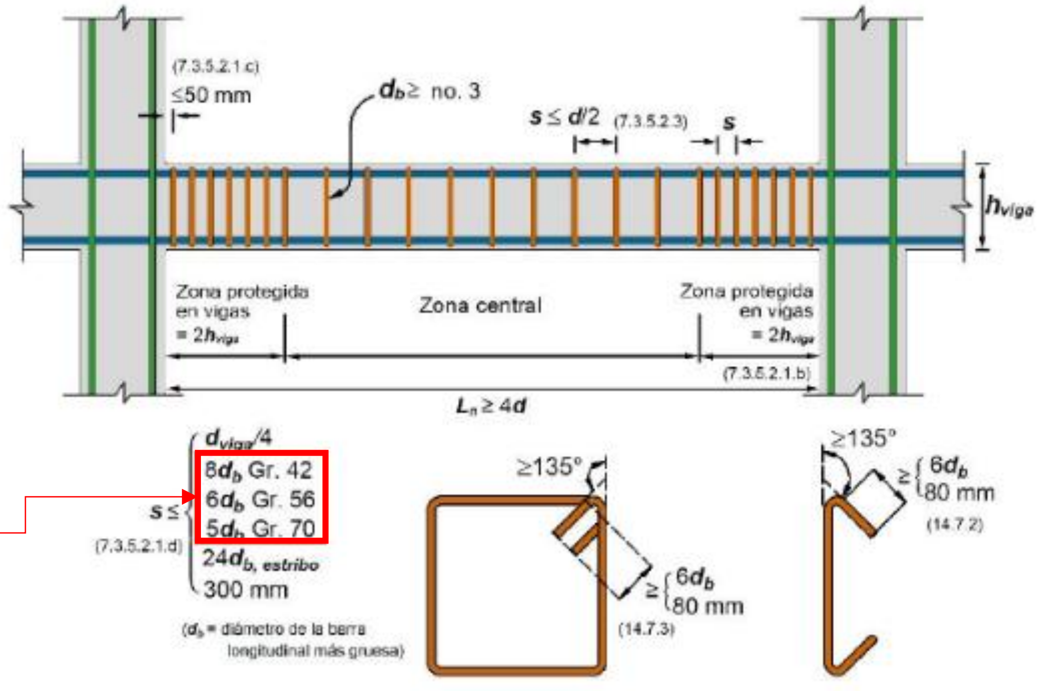

Figura 7.3.5.1.2 - Detallado de vigas de ductilidad media

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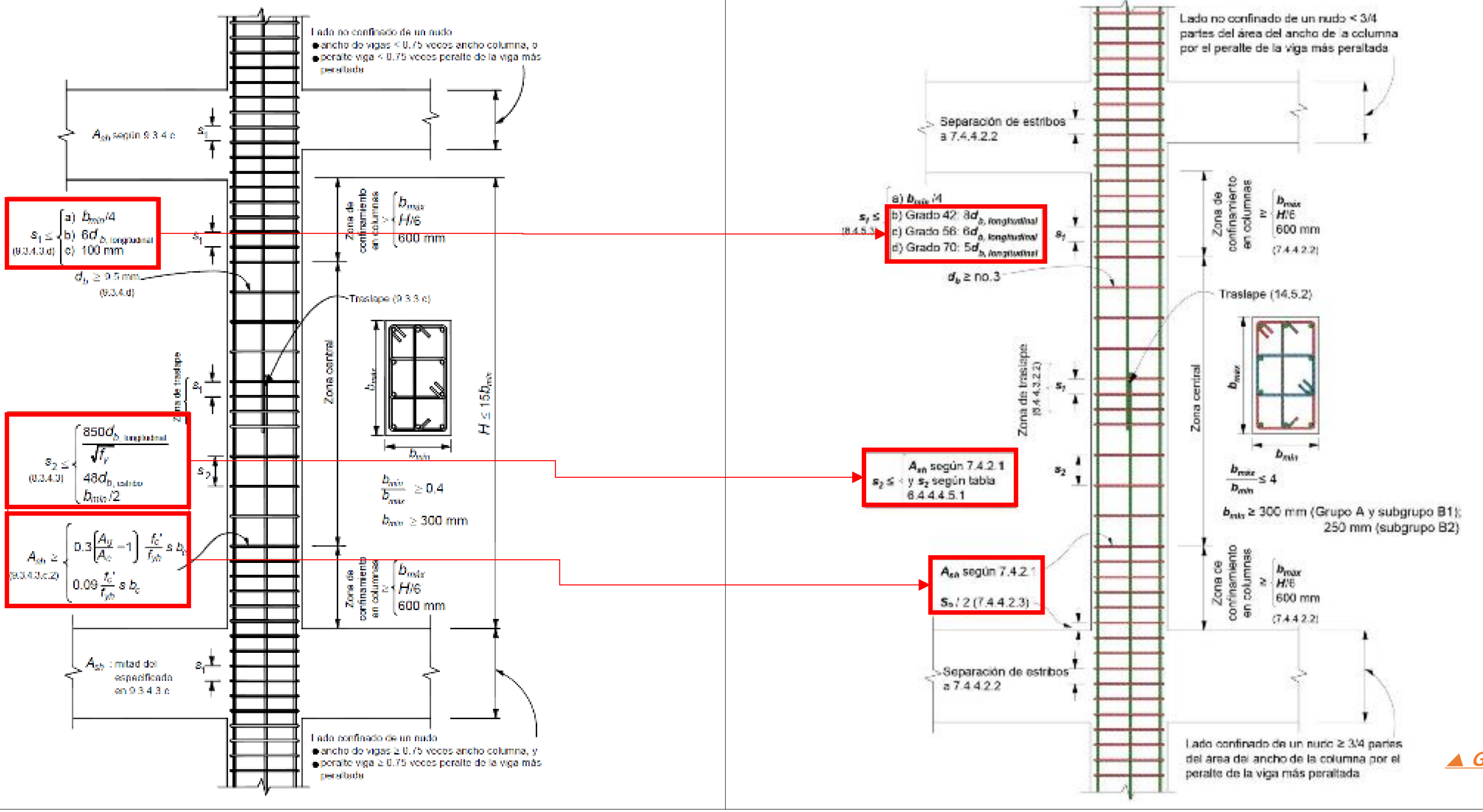
Added the Mexican design code (NTC 2023)

- *Column design of medium ductility structure - 1*

2017	2023



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Added the Mexican design code (NTC 2023)

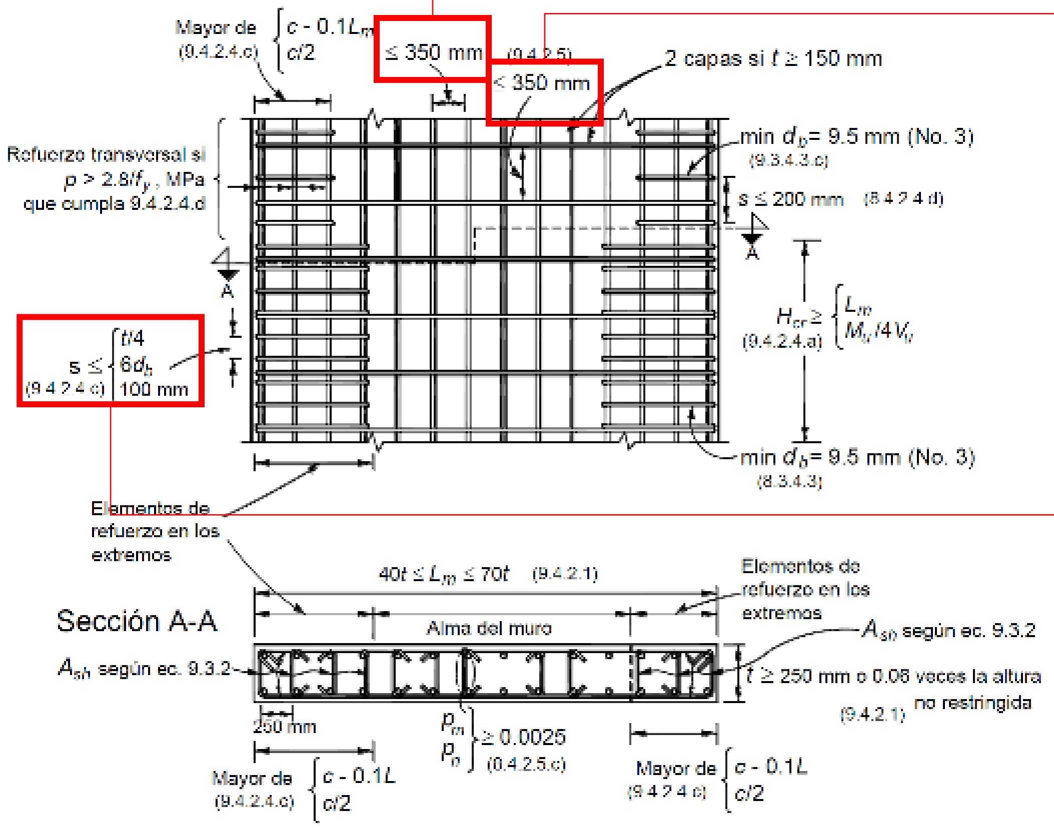
- *Column design of medium ductility structure - 2*

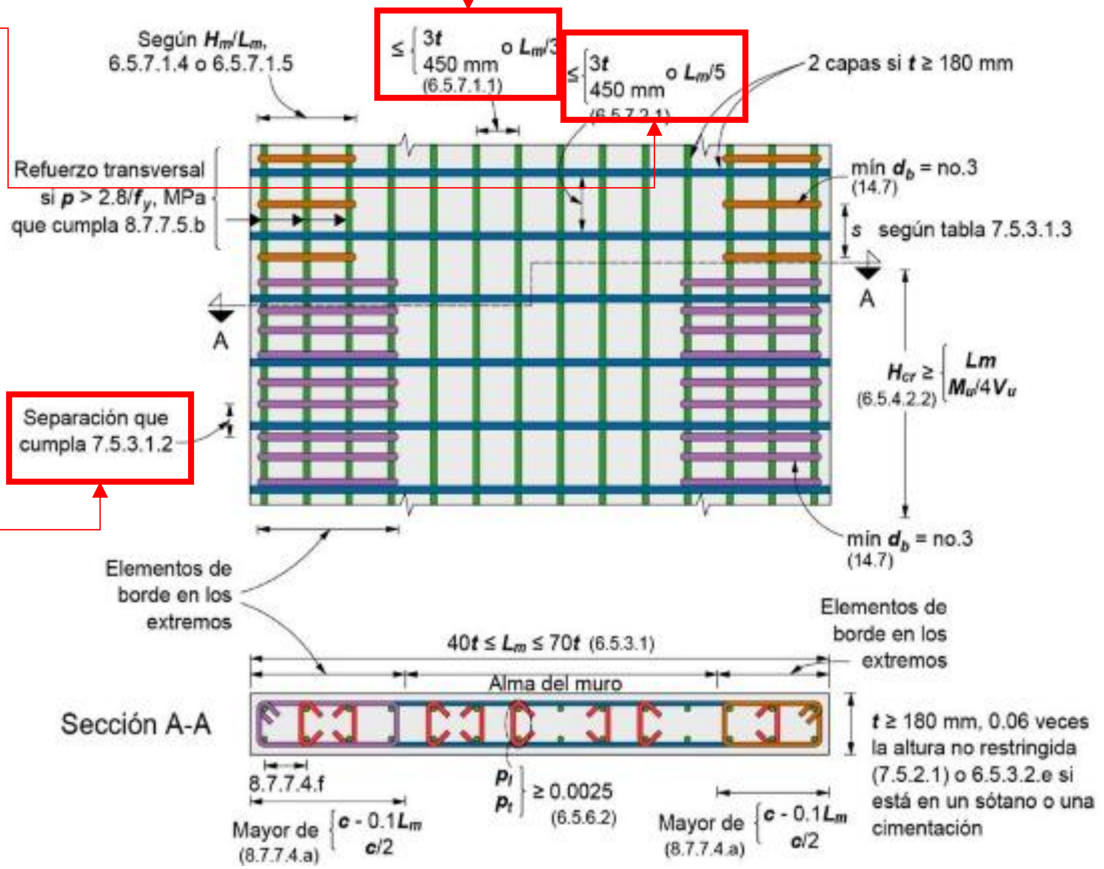
2017	2023
	7.4.4.2.2 Closed stirrups complying with 14.7.3 shall be provided at both ends of the column with spacing so over a distance Lo measured from the face of the node. Spacing so shall not exceed the lesser of a) to d): a) For Grade 42 bars, the lesser of 8db of the thinnest longitudinal bar and 200 mm b) For Grade 56 bars, the lesser of 6db of the thinnest longitudinal bar and 150 mm c) For Grade 70 bars, the lesser of 5db of the thinnest longitudinal bar and 150 mm d) One-fourth of the smallest cross-sectional dimension of the column. The length Lo shall not be less than the maximum value of a) to d): a) One sixth of the free height of the column b) The largest dimension of the cross section of the column c) 600 mm d) H/2 for ground floor columns or the first level subject to earthquakes, where H is the free height of the column.
8.3.2 Minimum flexural strength of columns 8.3.2.1 General procedure The flexural strengths of columns at a node must satisfy equation 8.3.1 **ΣMe ≥ 1.2ΣMg (8.3.1)** where: ΣMe adds to the node span of the moments of resistance in the analysis plane calculated with a resistance factor equal to one, of the columns that reach that node; the moment of resistance will be that which corresponds to the factored axial load that, in an interaction diagram of the column, produces the lowest moment of resistance. When calculating the moments of resistance in the analysis plane, the moments that act in the perpendicular plane will not be considered; and ΣMg adds to the node span of the moments of resistance calculated with a resistance factor equal to one, of the beams that reach the node. The above sums must be made so that the moments of the columns oppose those of the beams. The condition must be met for both directions in which the earthquake can act.	7.4.2.2 Minimum flexural strength of columns 7.4.2.2.2 The flexural strengths of the columns shall satisfy Eq. 7.4.2.2.2: **$\Sigma M_{nc} \geq 1.2\ \Sigma M_{nb}$ (7.4.2.2.2)** where: ΣM_{nc} sum of the nominal flexural strengths of the column sections above and below the node, in the analysis plane, calculated at the node span. The nominal resistant moment shall be that corresponding to the factored axial force that, in an interaction diagram of the column, produces the smallest resistant moment. ΣM_{nb} sums to the node span of the **nominal flexural strengths of the beams** reaching the node. In the case of monolithically cast beams with slabs, it will not be necessary to consider the contribution of the slab reinforcement steel to the flexural strength. The flexural strength of columns and beams shall be calculated with a steel stress of fy and a resistance factor equal to 1.0. The above sums must be made so that the moments of the columns oppose those of the beams. This condition must be met for both directions in which the earthquake can act. ▲ Go to Index

Added the Mexican design code (NTC 2023)

- *Wall design of medium ductility structure*

2017	2023





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Added the Mexican design code (NTC 2023)

- *Joint design of medium ductility structure*

2017	2023
	7.9.2 Beam-column connections 7.9.2.1 Beam-column connections shall satisfy the detailing requirements of 6.9.7.1.1.2 and 6.9.7.1.1.3. and 7.9.2.2 through
	7.9.2.7 Shear strength of beam-column connections 7.9.2.7.1 Vu of the node shall be determined in accordance with 8.5.5.1. (Same as Low Ductility System) 7.9.2.7.2 VR of a beam-column connection shall be calculated in accordance with 6.9.5.2.

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Added the Mexican design code (NTC 2023)

- *Beam design of High ductility structure*

2017	2023

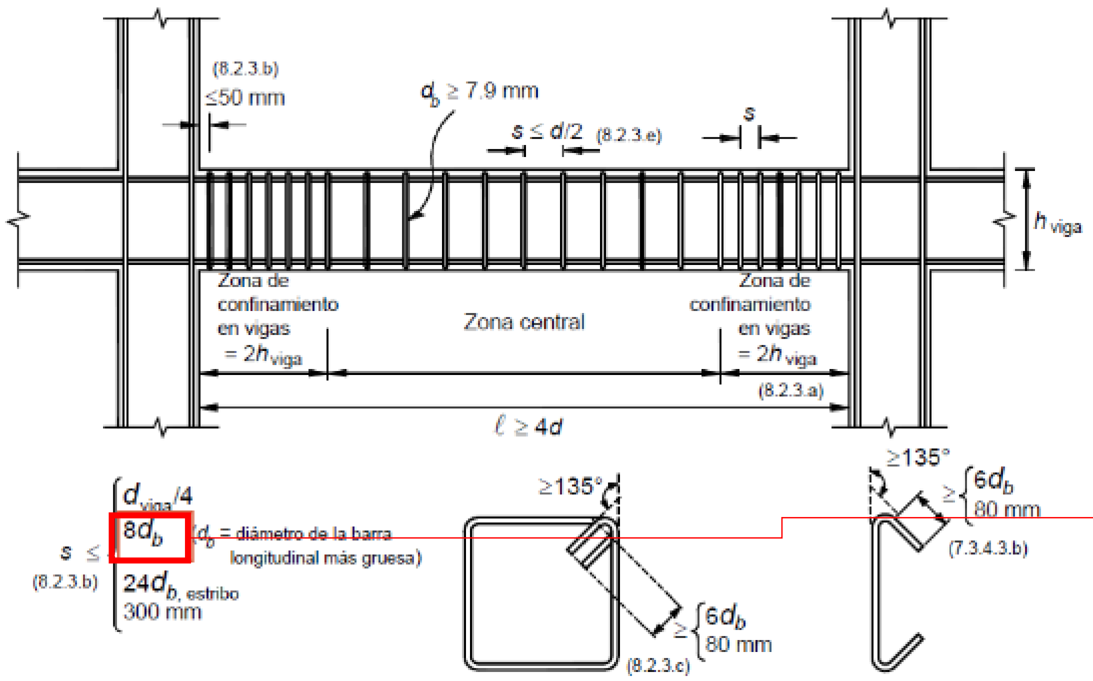

Figura 8.2.1 Detallado de elementos a flexión de ductilidad media

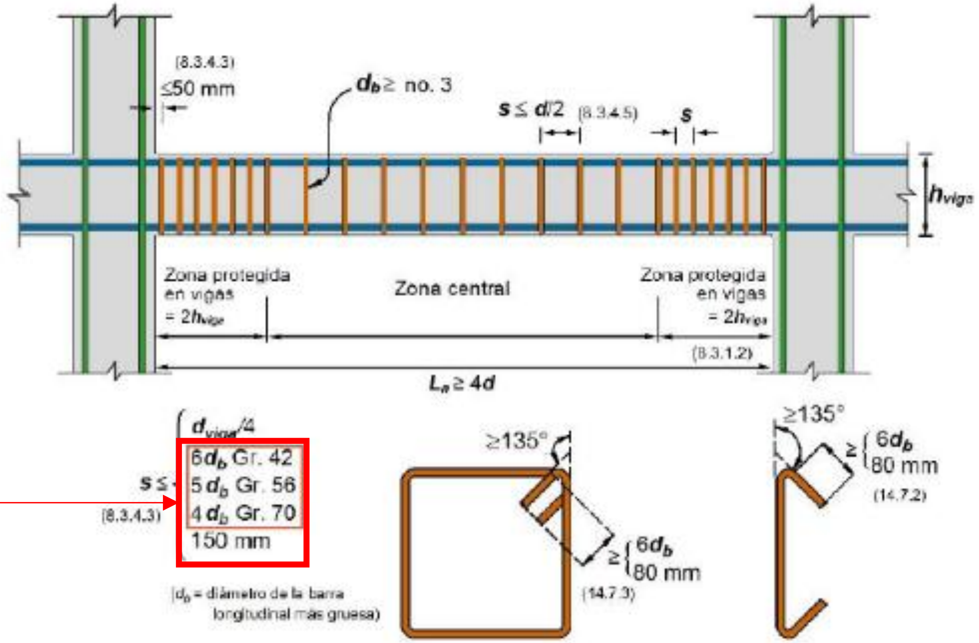

Figura 8.3.1.1 - Detallado de vigas de ductilidad alta

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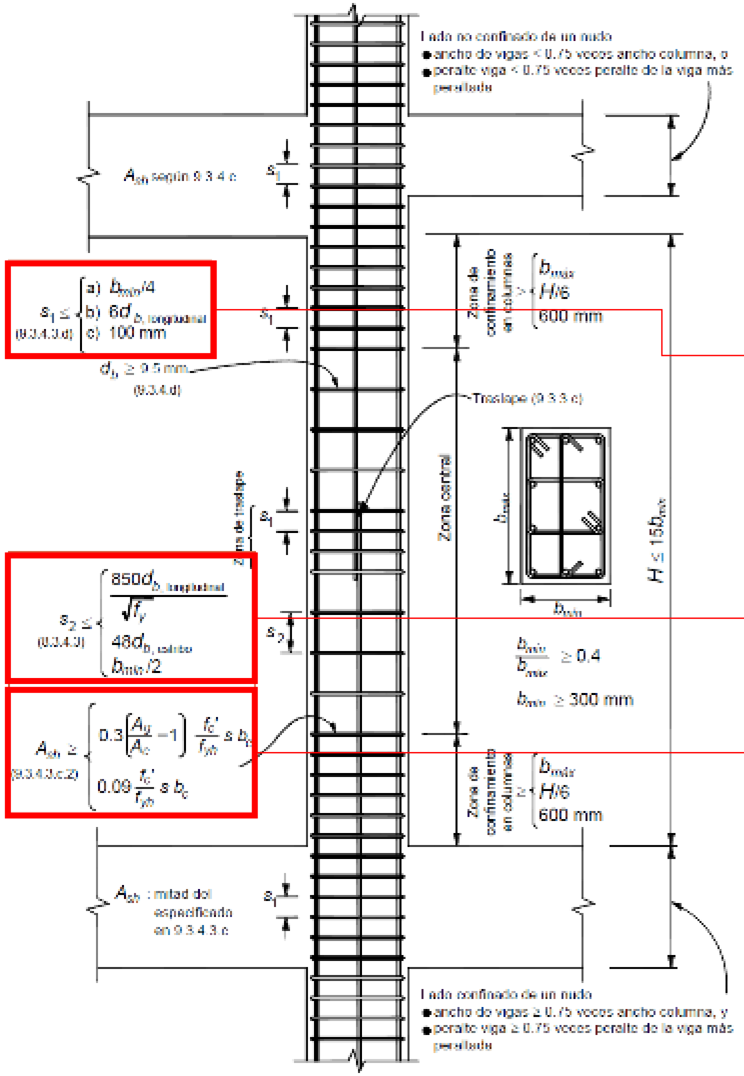
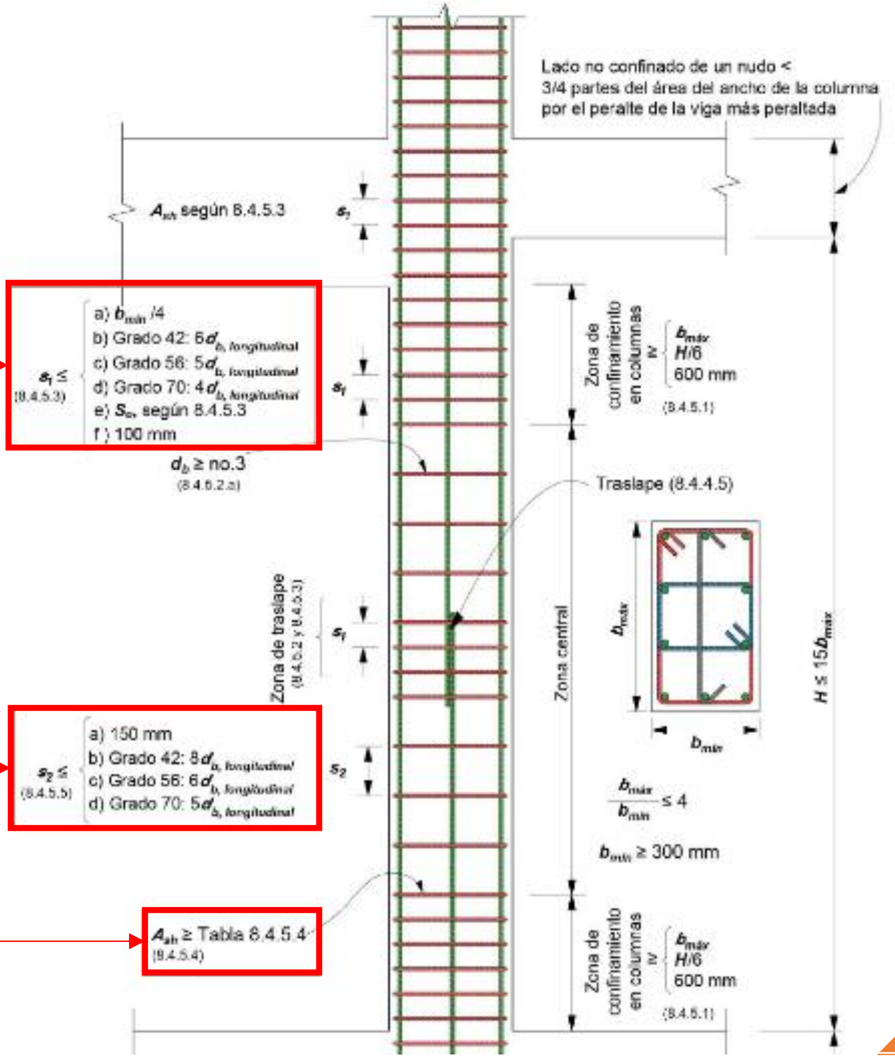
Added the Mexican design code (NTC 2023)

- *Column design of [High ductility structure - 1]*

2017	2023





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Added the Mexican design code (NTC 2023)

- *Column design of High ductility structure - 2*

2017	2023
	8.4.5.3 The spacing of transverse reinforcement shall not exceed the lesser of a) to e):
	8.4.5.4 The amount of transverse reinforcement shall be as obtained from table 8.4.5.4.

2023 — 8.4.5.3

8.4.5.3 The spacing of transverse reinforcement shall not exceed the lesser of a) to e):

a) One-fourth of the smallest transverse dimension of the element
b) 6db of the thinnest longitudinal bar of the primary flexural reinforcement Grade 42
c) 5db of the thinnest longitudinal bar of the primary flexural reinforcement Grade 56
d) 4db of the thinnest longitudinal bar of the primary flexural reinforcement Grade 70
e) or according to Eq. 8.4.5.3:

$$s_o = 100 + \frac{350 + h_z}{3}$$

The value of so in Eq. 8.4.5.3 shall not exceed 150 mm and shall not be less than 100 mm.

2023 — 8.4.5.4

8.4.5.4 The amount of transverse reinforcement shall be as obtained from table 8.4.5.4. The kf factor associated with the concrete strength and the kn factor on the confinement effectiveness shall be calculated according to eqs. 8.4.5.4.a and 8.4.5.4.b, respectively:

$$k_f = \frac{f_c'}{175} + 0.6 \geq 1.0 \qquad (8.4.5.4.a)$$

$$k_n = \frac{n_l}{n_l - 2} \qquad (8.4.5.4.b)$$

where nl is the number of bars or bundles of longitudinal reinforcement around the perimeter of the core of a column with rectangular stirrups that are laterally supported by stirrup corners or by standard 135-degree hooks.

Tabla 8.4.5.4 – Refuerzo transversal en columnas de marcos de ductilidad alta

Refuerzo transversal	Condiciones	Ecuación aplicable	
A_{sh}/sb_c para estribos rectangulares	$P_u \leq 0.3A_g f_c'$ y $f_c' \leq 70$ MPa (700 kg/cm²)	El mayor de a) y b):	a) $0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ b) $0.09\frac{f_c'}{f_{yt}}$ c) $0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$
	$P_u > 0.3A_g f_c'$ o $f_c' > 70$ MPa (700 kg/cm²)	El mayor de a), b) y c):	
p_s para refuerzo helicoidal o estribos circulares	$P_u \leq 0.3A_g f_c'$ y $f_c' \leq 70$ MPa (700 kg/cm²)	El mayor de d) y e):	d) $0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ e) $0.12\frac{f_c'}{f_{yt}}$ f) $0.35k_f \frac{P_u}{f_{yt}A_{ch}}$
	$P_u > 0.3A_g f_c'$ o $f_c' > 70$ MPa (700 kg/cm²)	El mayor de d), e) y f):	

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Added the Mexican design code (NTC 2023)

- *Column design of <u>High ductility structure - 3</u>*

2017	2023
	8.4.5.5 Beyond the length calculated in accordance with 8.4.5.1, helical reinforcement complying with 14.7.4 or stirrups and clips in accordance with 14.7.3 and 14.7.2, respectively, shall be provided. The spacing s shall not exceed the lesser of a) through d), unless a greater amount of transverse reinforcement is required in accordance with 8.4.4.5 and 8.4.6: a) 150 mm b) 8db of the thinnest longitudinal bar, for Grade 42 bars c) 6db of the thinnest longitudinal bar, for Grade 56 bars d) 5db of the thinnest longitudinal bar, for Grade 70 bars.
	8.4.5.1 The minimum confining transverse reinforcement specified in 8.4.5.2 to 8.4.5.4 shall be provided in a length lo at both ends of the member and on both sides of any section where longitudinal reinforcement is likely to yield in flexure under lateral displacements in the inelastic range of behaviour. The length lo shall be the greater of a) to c): a) The greatest depth of the column at the node face or at the section where longitudinal reinforcement is likely to yield in flexure (see Fig. 8.4.1) b) H/6, where H is the clear height of the column c) 600 mm.
9.3.2 Minimum flexural strength of columns The flexural strengths of columns at a node must satisfy equation 9.3.1 **$\Sigma Me \geq 1.2 \Sigma Mg$ (9.3.1)** where: ΣMe adds to the node span the moments of resistance in the analysis plane calculated with a resistance factor equal to one, of the columns that reach that node; the moment of resistance will be that which corresponds to the factored axial load that, in an interaction diagram of the column, produces the lowest moment of resistance. When calculating the moments of resistance in the analysis plane, the moments that act in the perpendicular plane will not be considered; and ΣMg adds to the node span the moments of resistance calculated with a resistance factor equal to one, of the beams that reach the node. The above sums must be made so that the moments of the columns oppose those of the beams. The condition must be met for both directions in which the earthquake can act. It will not be necessary to comply with equation 9.3.1 at the roof nodes.	8.4.3 Minimum flexural strength of columns 8.4.3.2 The flexural strengths of the columns shall satisfy Eq. 8.4.3.2: **$\Sigma Mnc \geq 1.2\ \Sigma Mprb$ (8.4.3.2)** where: ΣMnc sum of the nominal flexural strengths of the column sections above and below the node, in the analysis plane, calculated at the node span. The nominal resisting moment shall be that corresponding to the factored axial load that, in an interaction diagram of the column, produces the smallest resisting moment in the analysis direction, using a resistance factor equal to 1.0. <u>$\Sigma Mprb$ sums to the node span the probable flexural strengths of the beams</u> reaching the node. In the case of beams cast monolithically with slabs, when the slab is in tension due to moments at the node face, the slab reinforcing steel within the effective width established in 8.5.2.2 shall be considered to contribute to Mprb if the slab reinforcing steel can develop its yield strength at the critical section by bending. <u>The flexural strength of the beams shall be calculated with a steel stress of 1.25 fy and a resistance factor equal to 1.0.</u> The above sums must be made so that the moments of the columns oppose those of the beams. The condition must be met for both directions in which the earthquake can act.

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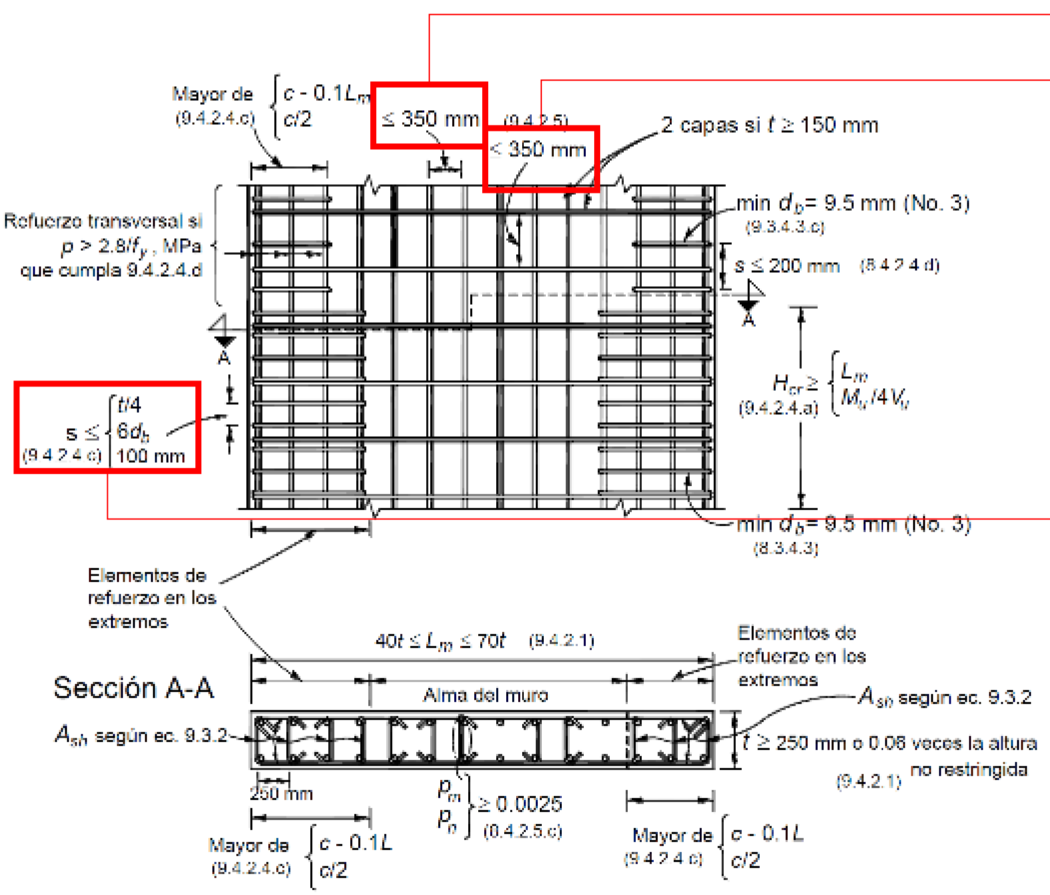
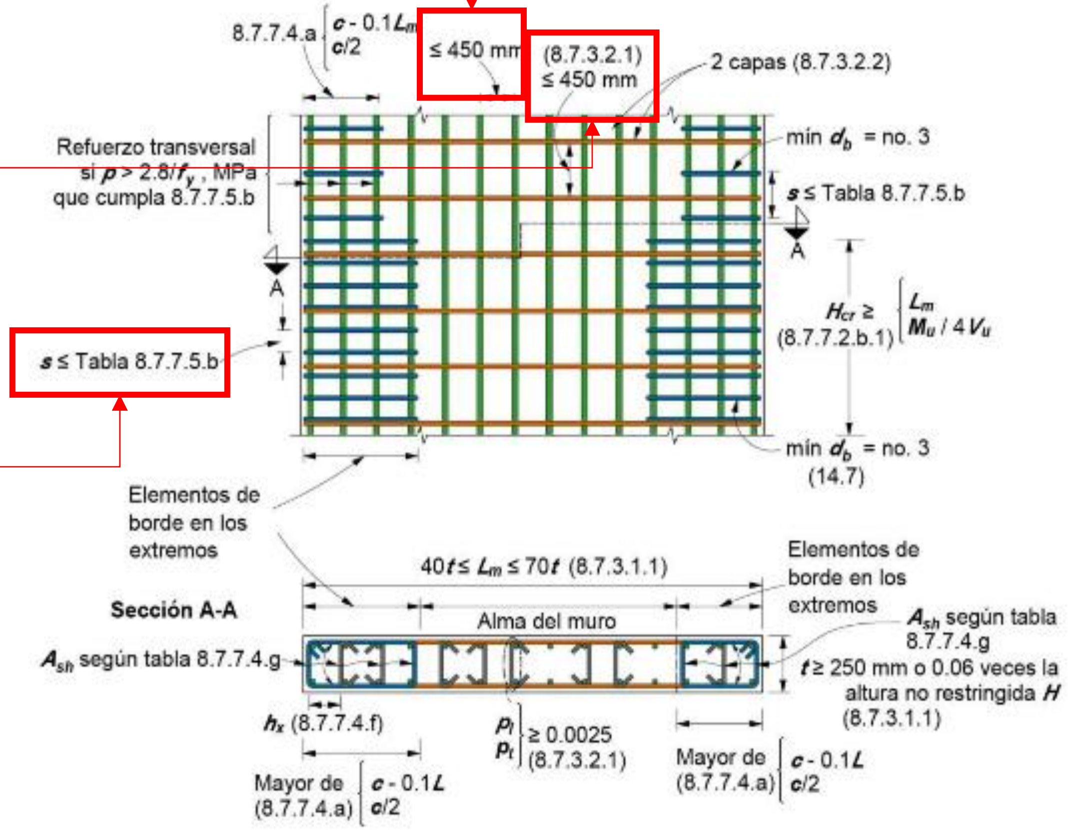
Added the Mexican design code (NTC 2023)

- *Wall design of* ***High ductility structure - 1***

2017



2023



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Added the Mexican design code (NTC 2023)

- *Wall design of* ***High ductility structure - 2***

2017	2023
	8.7.4 Design Shear Force

8.7.4 Design Shear Force

8.7.4.3.2 If the factored shear force V_{ush} is determined from an elastic structural analysis, the portion of the design shear force of the wall due to the horizontal component of the earthquake, calculated according to NTC-Earthquake, shall be amplified by the product $\Omega_v\ \omega_v$, where Ω_v and ω_v are defined in 8.7.4.3.3 to 8.7.4.3.5.

8.7.4.4.3.3 Ωv and ωv shall be calculated in accordance with Table 8.7.4.3.3, except that Ωv shall be permitted to be calculated as Mpr/Mu at the critical flexural section for the applicable load combination that includes earthquake.

Tabla 8.7.4.3.3 – Factores Ω_v y ω_v

Condición	Ω_v	ω_v [1]
$H_{wcs}/L_w \leq 1.0$	1.0	1.0
$1.0 < H_{wcs}/L_w < 2.0$	Se permite la interpolación lineal 1.0 y 1.5	1.0
$H_{wcs}/L_w > 2.0$	1.5	$0.8 + 0.13\ H_n^{1/3}$

[1] H_n is the height of the structure from the base to the highest level of the seismic-resisting system, in meters. The base of the structure is the level where the seismic motion is considered to be applied.

8.7.4.3.4 The product $\Omega_v\omega_v$ shall not exceed 2.0.

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Added the Mexican design code (NTC 2023)

- *Joint design of High ductility structure*

2017	2023
	8.5.2 General requirements 8.5.2.1 The forces in the non-prestressed deformed bars of the longitudinal reinforcement of the beams at the node face shall be calculated assuming that the stress in the tensile flexural reinforcement is $1.25f_y$.
	8.5.5 Shear strength resistance 8.5.5.1 The joint shear resistance in each principal direction of the section shall be checked independently. The joint shear force V_{uj} shall be calculated in the horizontal plane leading to the largest value of V_{uj}/A_j from the calculated forces at the joint face using the values of beam tension and compression forces determined in accordance with 8.5.2.1 and 8.5.2.3, as applicable, and *the column shear consistent with the probable beam flexural strengths M_{pr}.*
	8.5.5.3 V_{Rj}

Tabla 8.5.5.3 - Resistencia de diseño a cortante de nudos de marcos de ductilidad alta

Columna	Viga en la dirección de V_{uj}	Confinamiento por vigas transversales de acuerdo con 8.5.5.3	V_{Rj}
Continua o cumple con 6.9.5.2.3	Continua o cumple con 8.5.5.3.a y 8.5.5.3.b	Confinada	$1.7F_R\sqrt{f'_c}A_j$ $\left(5.5F_R\sqrt{f'_c}A_j\right)$
		No confinada	$1.3F_R\sqrt{f'_c}A_j$ $\left(4.5F_R\sqrt{f'_c}A_j\right)$
	Otra	Confinada	$1.3F_R\sqrt{f'_c}A_j$ $\left(4.5F_R\sqrt{f'_c}A_j\right)$
		No confinada	$1.0F_R\sqrt{f'_c}A_j$ $\left(3.5F_R\sqrt{f'_c}A_j\right)$
Otra	Continua o cumple con 8.5.5.3.a y 8.5.5.3.b	Confinada	$1.3F_R\sqrt{f'_c}A_j$ $\left(4.5F_R\sqrt{f'_c}A_j\right)$
		No confinada	$1.0F_R\sqrt{f'_c}A_j$ $\left(3.5F_R\sqrt{f'_c}A_j\right)$
	Otra	Confinada	$1.0F_R\sqrt{f'_c}A_j$ $\left(3.5F_R\sqrt{f'_c}A_j\right)$
		No confinada	$0.67F_R\sqrt{f'_c}A_j$ $\left(2.0F_R\sqrt{f'_c}A_j\right)$

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Added the Mexican design code (NTC 2023)

- *Transverse Rebar Detail*

2017	2023
5.3.5 Shear reinforcement 5.3.5.1 Reinforcement in beams and columns without prestressing b) If Vu is greater than VcR but less than or equal to the value calculated using equation 5.3.4, the spacing of stirrups perpendicular to the axis of the element shall not be greater than 0.5d. c) If Vu is greater than the value calculated using equation 5.3.4, the spacing of stirrups perpendicular to the axis of the element shall not be greater than 0.25d.	**14.7.3 Close stirrups** 14.7.3.2 Closed stirrups shall be made of deformed bars with spacings indicated in a) and b): a) Clear spacing at least equal to 1.5tmag b) Center spacing not exceeding the lesser of 16db of the longitudinal bar, 48db of the stirrup bar and the distance required according to the type of member (beam, column) and the expected level of ductility (low, medium or high). $s_{max} = min(d_t + 1.5t_{mag}, 16d_b, 48d_t)$

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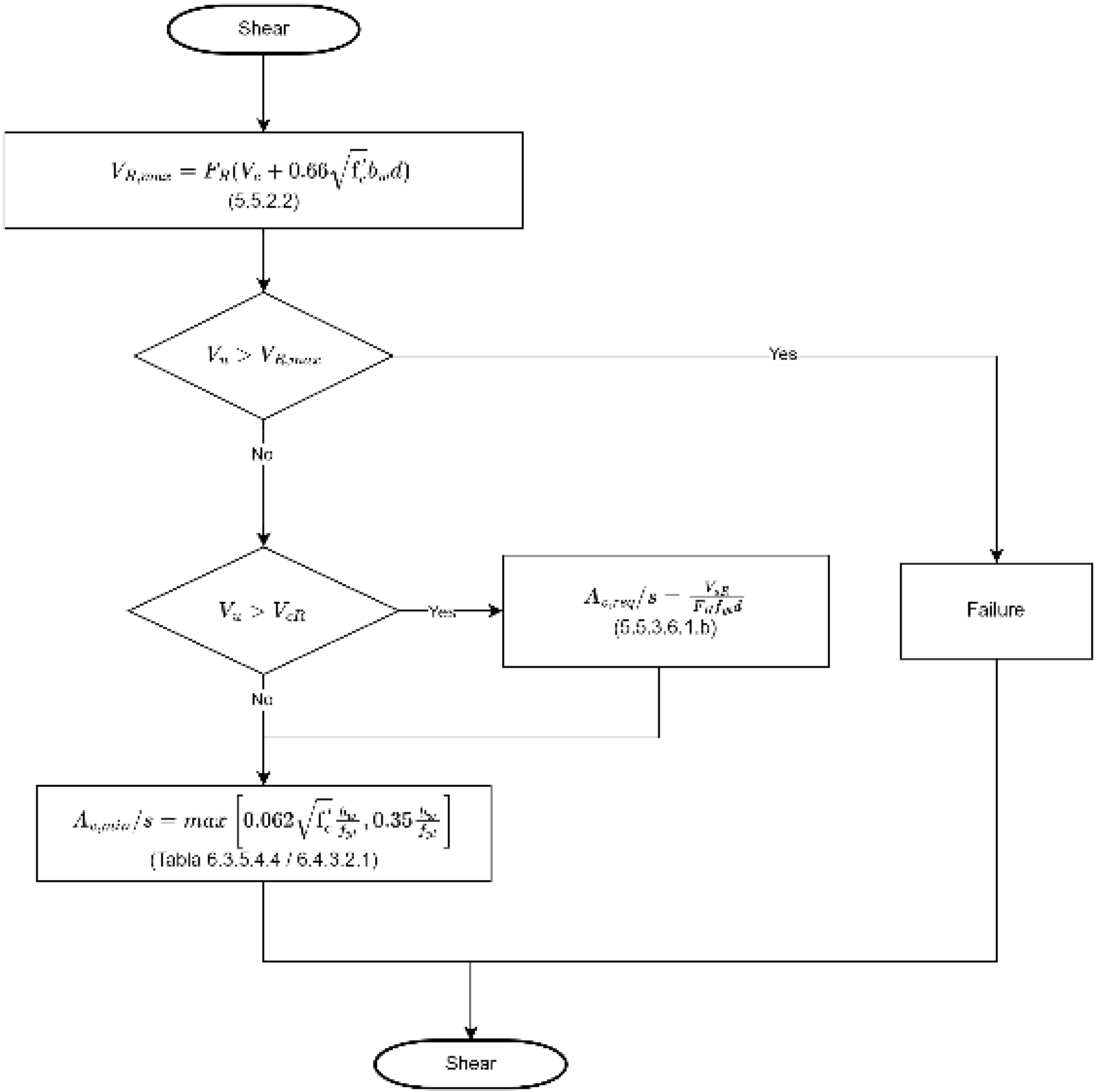
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → Shear Reinforcement*



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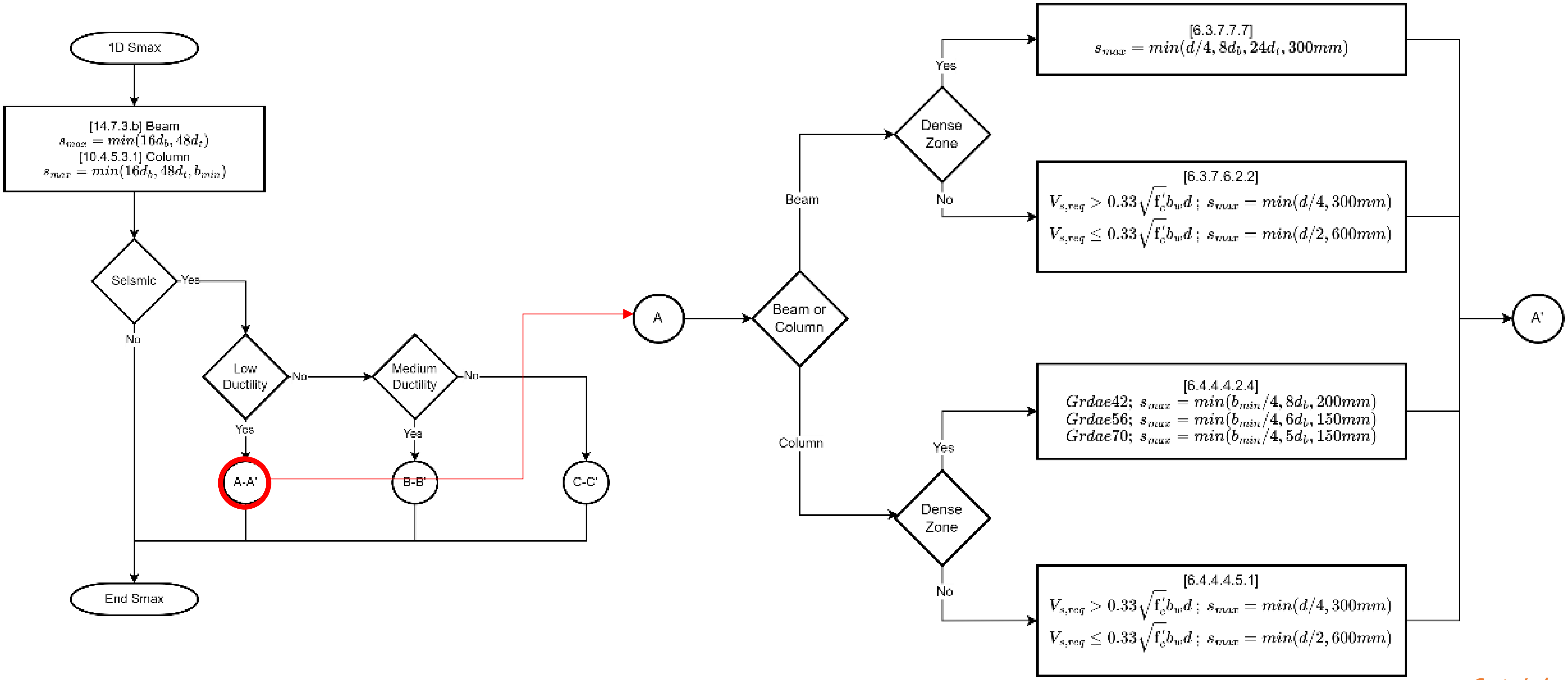
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → Max. Space of Shear Reinforcement in 1D member - 01*



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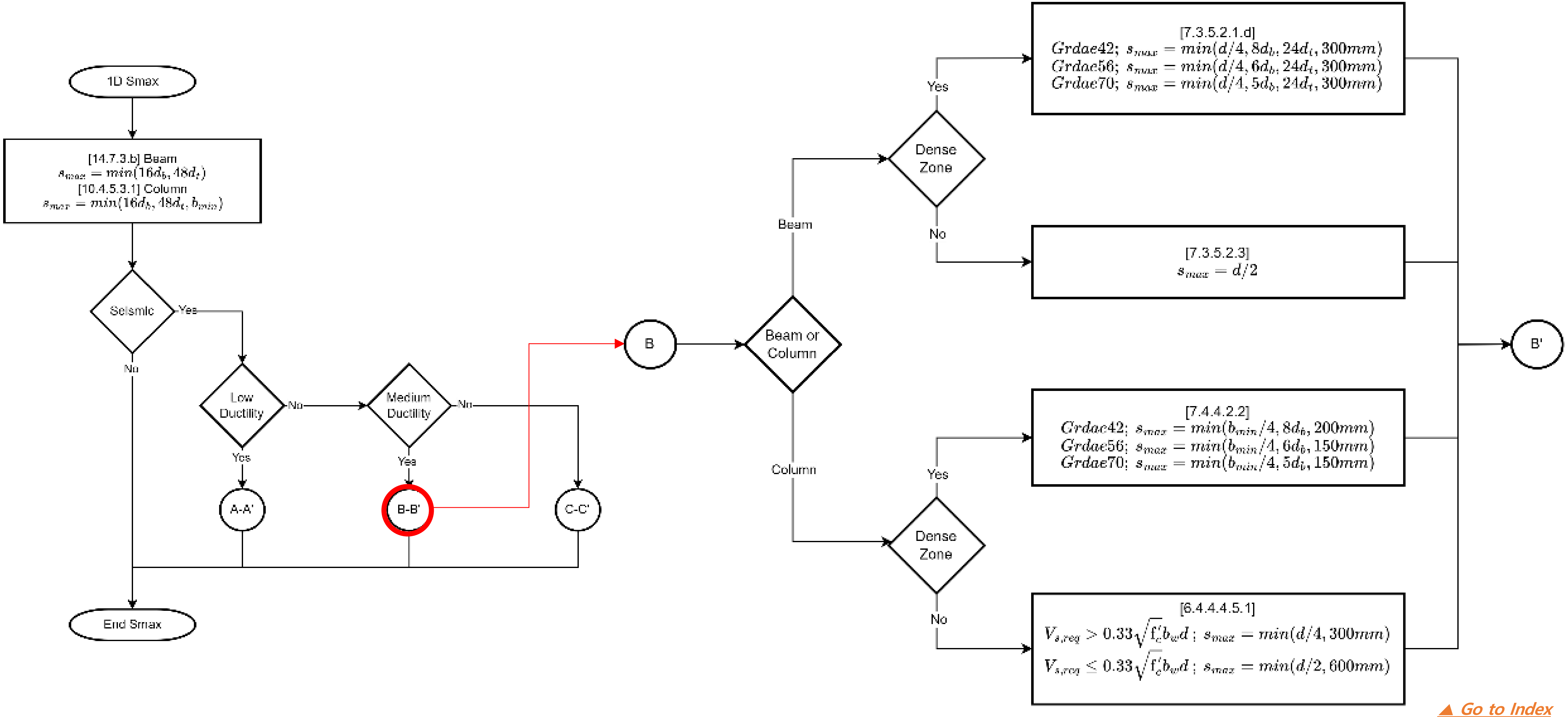
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → Max. Space of Shear Reinforcement in 1D member - 02*



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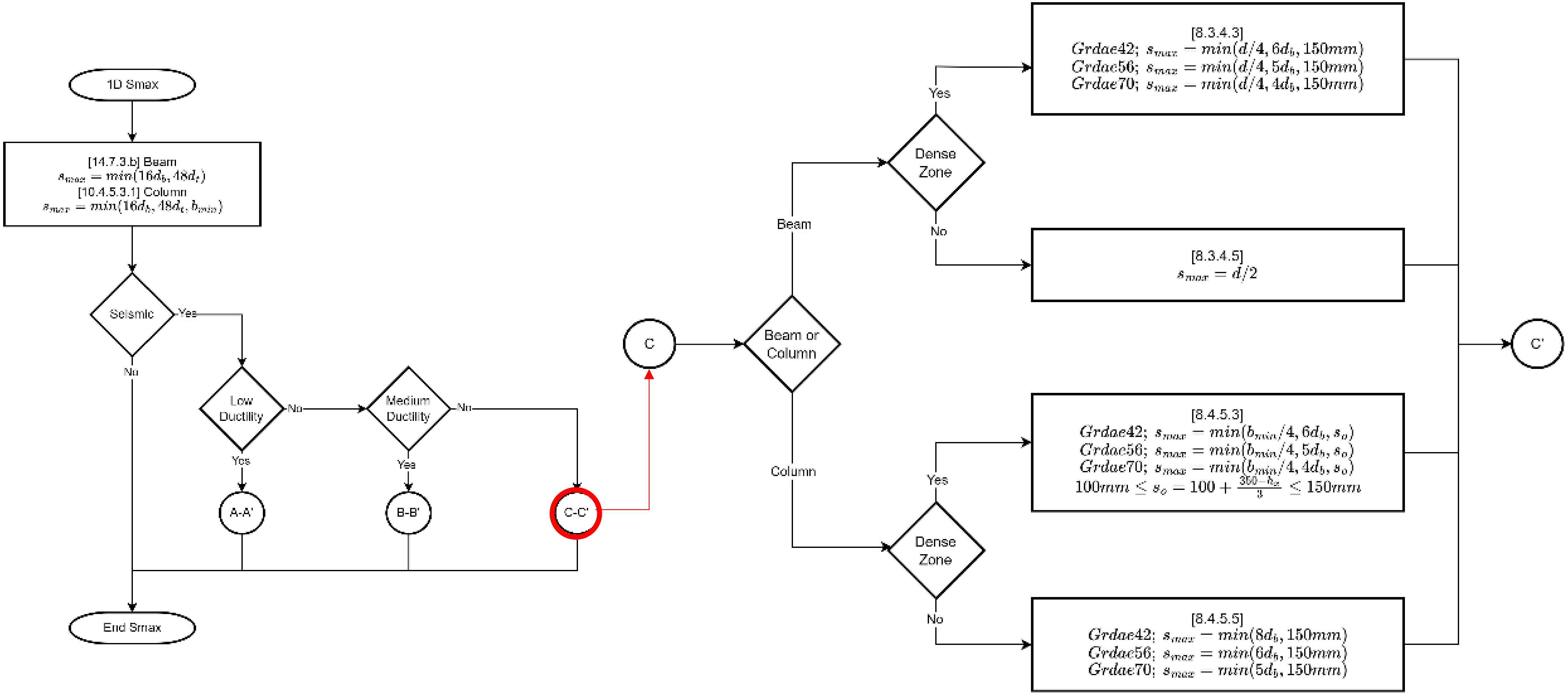
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → Max. Space of Shear Reinforcement in 1D member - 03*



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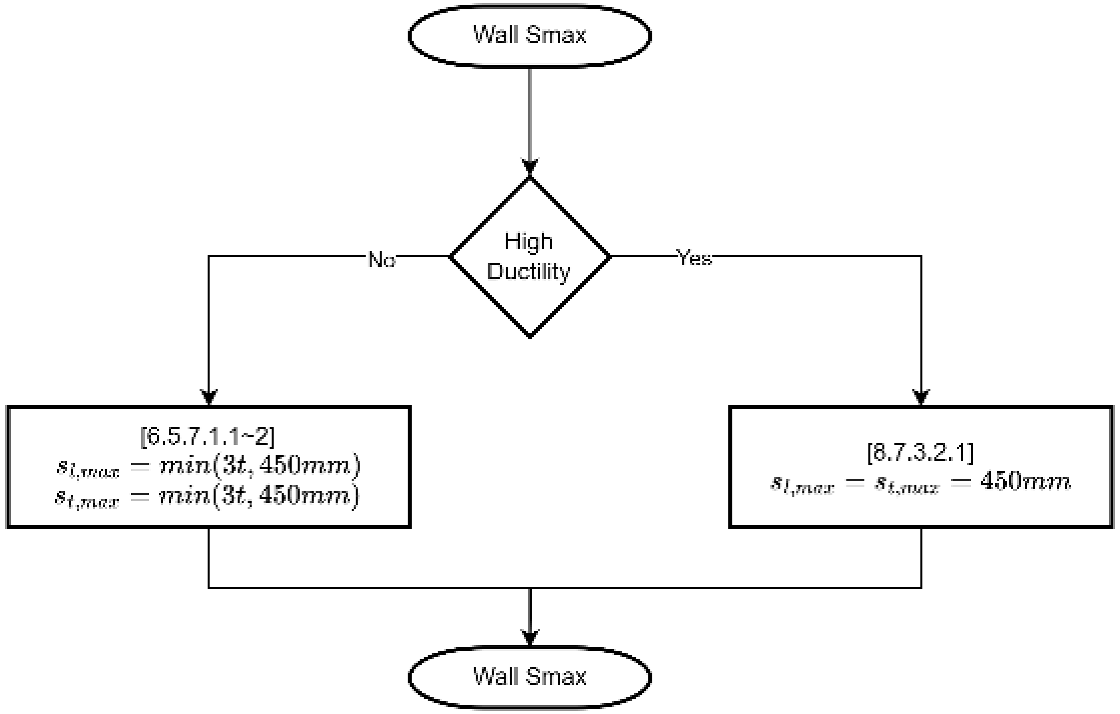
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → S_{max} in Wall Shear Design*



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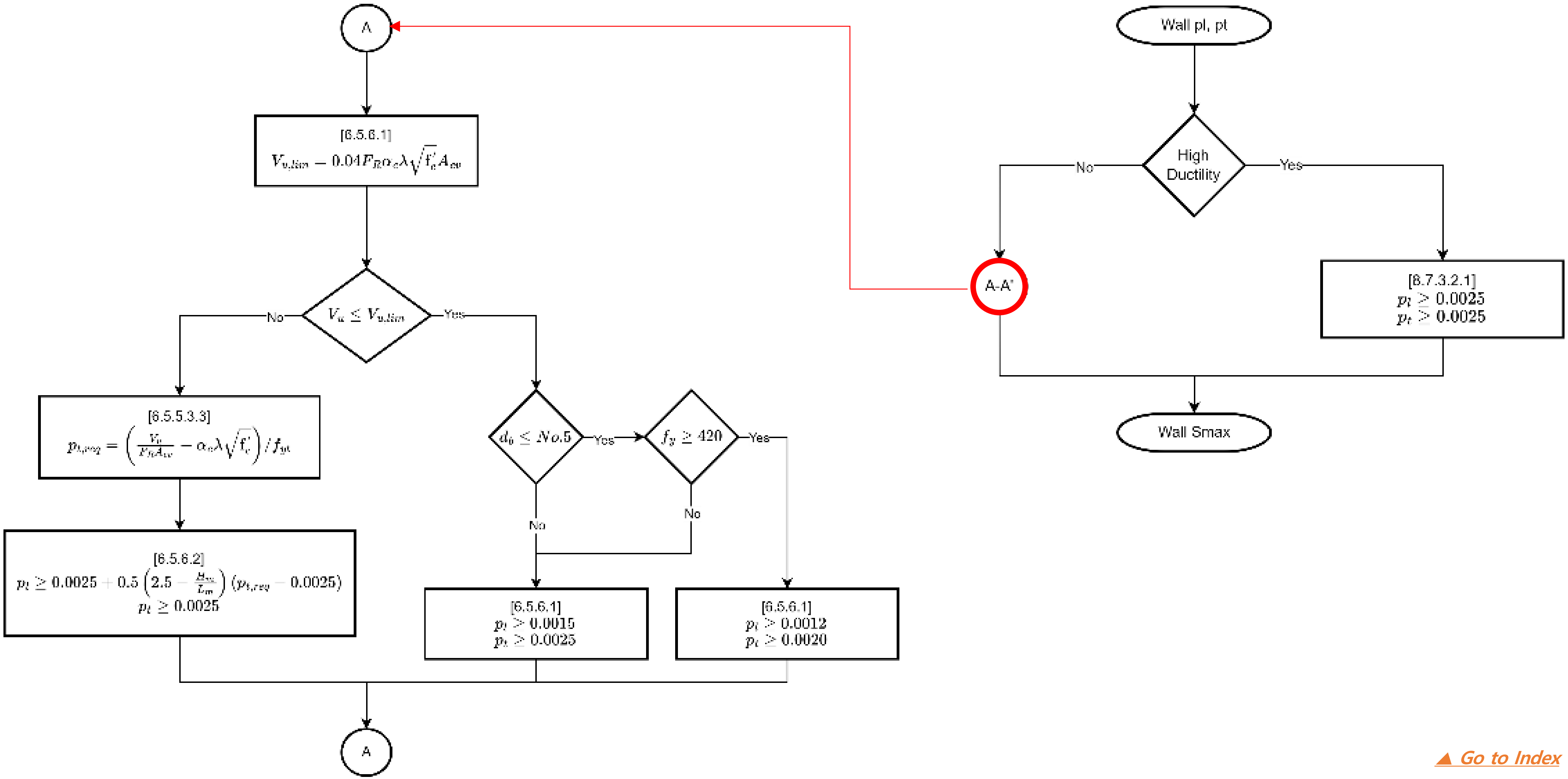
Added the Mexican design code (NTC 2023)

- *Design Flow Chart → Rebar Ratio Limit in Wall Design*



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Appendix 02

Russian code in midas Gen

- ***Added design of steel according to SP 16.13330.2017***
  1. *Calculation of section strength*
  2. *Calculation of the stability of a plane bending form*
  3. *Calculation of stability of off-centre compressed elements*
  4. *Check of local stability of webs and flanges of the cross section*
- ***Added design of reinforced concrete structures according to SP 63.13330.2018***
  1. *Strength check of the normal section of a beam/column element*
  2. *Checking the strength of the inclined section of a beam/column, element against the action of moment and shear force*
  3. *Calculation of crack resistance and crack opening width in the normal section of a beam element*
- ***Special structural requirements are taken into account in the design of structures***
  1. *Added consideration of structural requirements in seismic design according to SP 14.13330.2018*
  2. *Added accounting of responsibility of the structure according to GOST 27751-2014*
- ***Added load combinations according to SP 20.13330.2016***
  1. *Main, special, crane, seismic load combinations*
  2. *Added special load combinations according to SP 296.1325800.2017*
- ***Implemented calculation of the pulsation component of the wind load according to SP 20.13330.2016***
  1. *Calculation of the pulsation force acting on a rigid floor diaphragm*
- ***Added response spectrum according to SP 14.13330.2018***
- ***Added base of materials and profiles for calculation according to Russian code***

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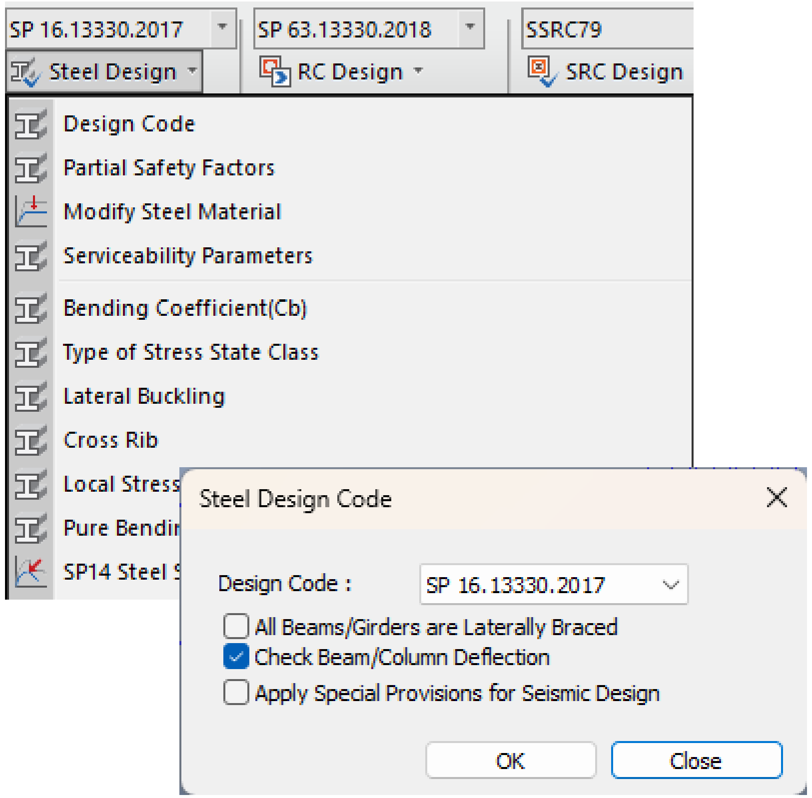
Added design of steel according to SP 16.13330.2017

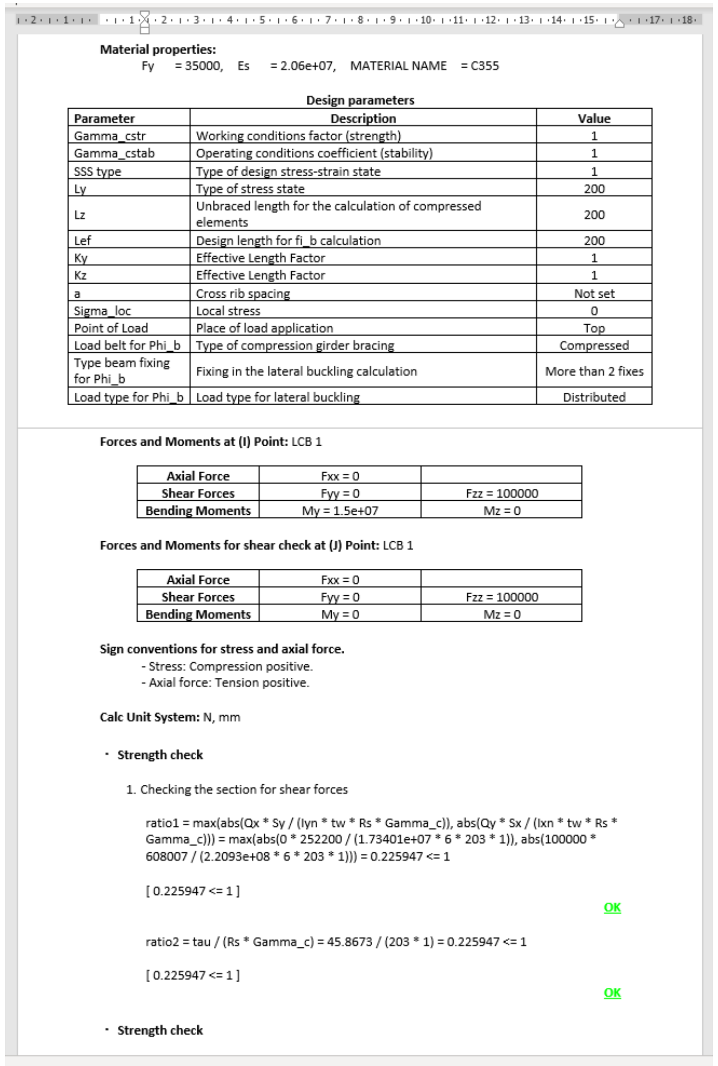
1. Calculation of section strength
2. Calculation of the stability of a plane bending form
3. Calculation of stability of off-centre compressed elements



Dialog box for setting parameters for steel structures calculation according to SP 16.13330.2017

Realised calculation types

- **Par. 7.1.1**- Strength calculation of elements in central tension or compression
- **Par. 8.2.1, 8.2.3** - Strength calculation of bending elements
- **Par. 9.1.1, 9.1.3** - Strength calculation of eccentrically compressed and eccentrically tensile elements
- **Par. 8.4.1** - Calculation of stability of I-beams of class 1: under the action of a moment
- **Par. 7.1.3** - Calculation of stability of elements of continuous section under central compression
- **Par. 9.2.2, 9.2.8, 9.2.9, 9.2.10, 9.2.4** - Calculation of stability of elements of continuous section under off-centre compression



Material properties:
Fy = 35000, Es = 2.06e+07, MATERIAL NAME = C355

Design parameters

Parameter	Description	Value
Gamma_cstr	Working conditions factor (strength)	1
Gamma_cstab	Operating conditions coefficient (stability)	1
SSS type	Type of design stress-strain state	1
Ly	Type of stress state	200
Lz	Unbraced length for the calculation of compressed elements	200
Lef	Design length for fi_b calculation	200
Ky	Effective Length Factor	1
Kz	Effective Length Factor	1
a	Cross rib spacing	Not set
Sigma_loc	Local stress	0
Point of Load	Place of load application	Top
Load belt for Phi_b	Type of compression girder bracing	Compressed
Type beam fixing for Phi_b	Fixing in the lateral buckling calculation	More than 2 fixes
Load type for Phi_b	Load type for lateral buckling	Distributed

Forces and Moments at (I) Point: LCB 1

Axial Force	Fxx = 0	
Shear Forces	Fyy = 0	Fzz = 100000
Bending Moments	My = 1.5e+07	Mz = 0

Forces and Moments for shear check at (J) Point: LCB 1

Axial Force	Fxx = 0	
Shear Forces	Fyy = 0	Fzz = 100000
Bending Moments	My = 0	Mz = 0

Sign conventions for stress and axial force.
- Stress: Compression positive.
- Axial force: Tension positive.

Calc Unit System: N, mm

- **Strength check**

1. Checking the section for shear forces

ratio1 = max(abs(Qx * Sy / (Iyn * tw * Rs * Gamma_c)), abs(Qy * Sx / (Ixn * tw * Rs * Gamma_c))) = max(abs(0 * 252200 / (1.73401e+07 * 6 * 203 * 1)), abs(100000 * 608007 / (2.2093e+08 * 6 * 203 * 1))) = 0.225947 <= 1

[0.225947 <= 1]

OK

ratio2 = tau / (Rs * Gamma_c) = 45.8673 / (203 * 1) = 0.225947 <= 1

[0.225947 <= 1]

OK

- **Strength check**

Text report in RTF format.

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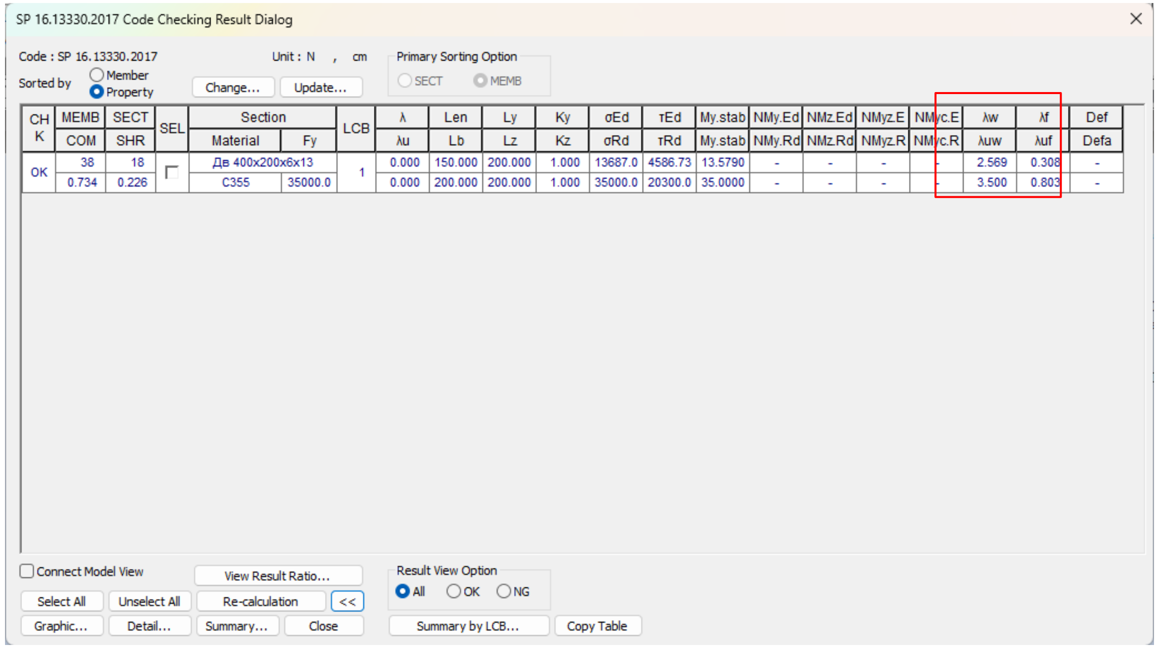
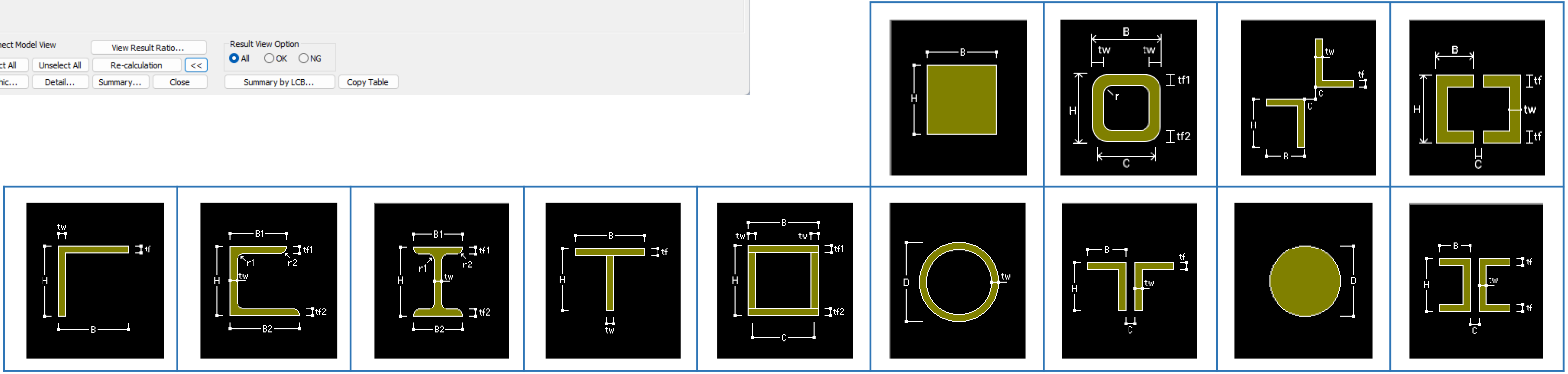
Added design of steel according to SP 16.13330.2017

4. Check of local stability of walls and flanges of the cross section



Realised calculation types

- **Par. 7.3.2, 7.3.3** - Web stability of centrally compressed elements of continuous section
- **Par. 8.5.1, 8.2.2, 8.5.3, 8.5.7, 8.5.8** - Stability of webs under the action of the moment
- **Par. 9.4.2** - Stability of web of off-centre compressed elements
- **Par. 11.2.2** - Web stability calculation of seamless or electrically welded pipes
- **Par. 7.3.8** - Stability of belt plates (flanges) of centrally compressed elements of continuous section
- **Par. 8.5.18, 8.5.19** - Stability of compressed beam flanges
- **Par. 9.4.7** - Stability of girdles (flanges) of off-centre compressed bars



Supported section types for calculation according to SP 16.13330.2017

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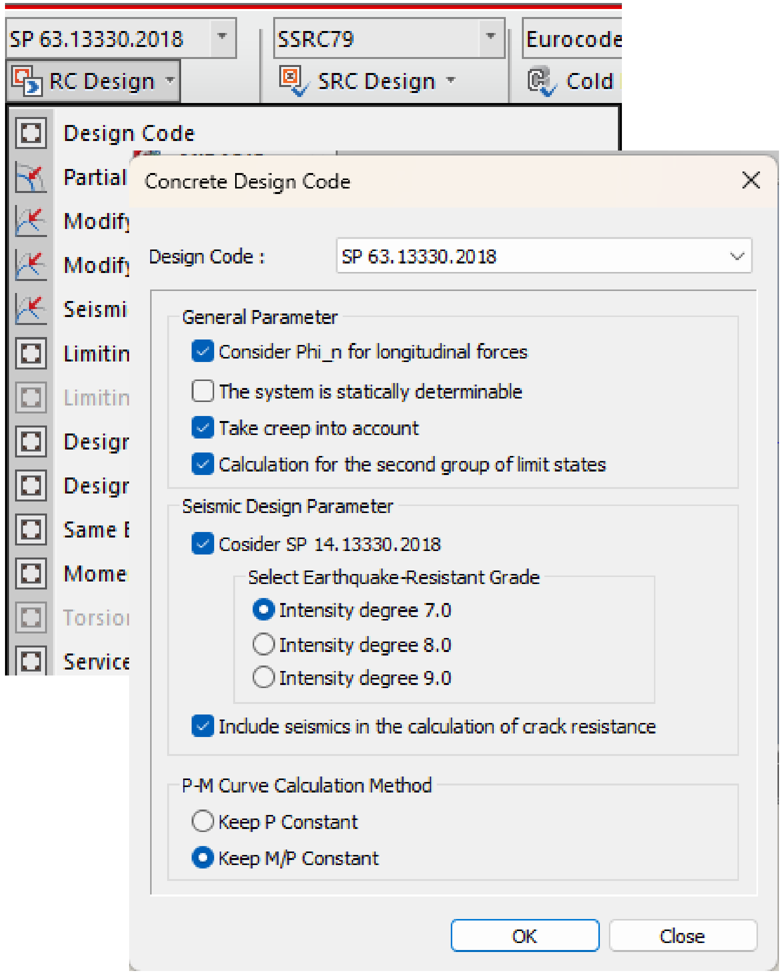
Added design of reinforced concrete structures according to SP 63.13330.2018

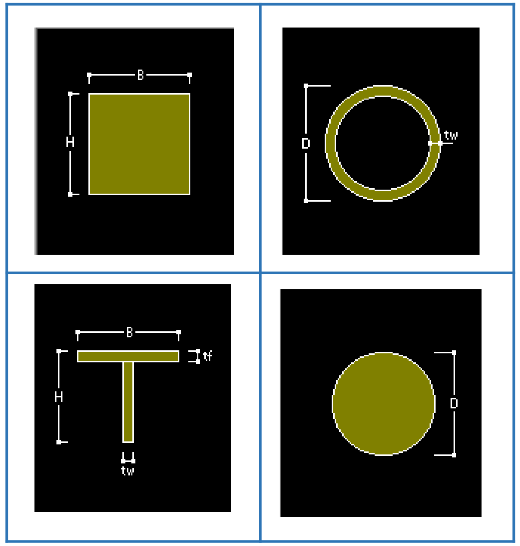
1. Strength check of the normal section of a beam/column element
2. Checking the strength of the inclined section of a beam/column, element against the action of moment and shear force



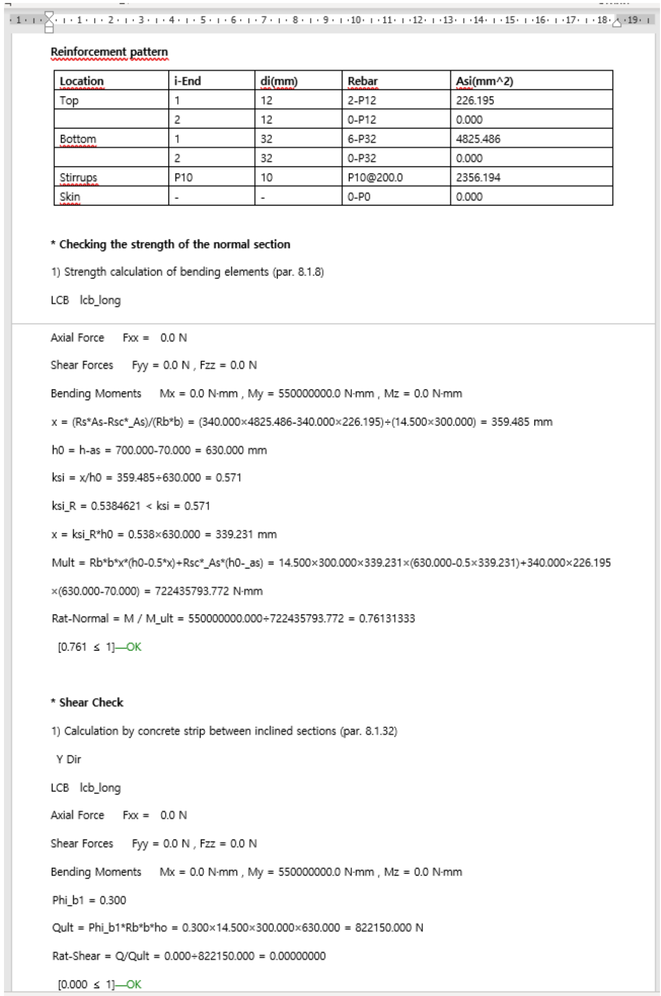
Dialog box for specifying parameters for calculation of reinforced concrete structures according to SP 63.13330.2018

Realised calculation types

- **Par. 8.1.8, 8.1.14, 8.1.18, 8.1.19, 8.1.24** - Strength of normal section against longitudinal forces and bending moments
- **Par. 8.1.33, 8.1.35, 8.1.37, 8.1.38, 8.1.40, 8.1.41, 8.1.42** – Strength of the inclined section under shear force and torque action



Supported section types for calculation according to SP 63.13330.2018



Reinforcement pattern

Location	i-End	di(mm)	Rebar	Asi(mm^2)
Top	1	12	2-P12	226.195
	2	12	0-P12	0.000
Bottom	1	32	6-P32	4825.486
	2	32	0-P32	0.000
Stirrups	P10	10	P10@200.0	2356.194
Skin	-	-	0-P0	0.000

*** Checking the strength of the normal section**

1) Strength calculation of bending elements (par. 8.1.8)

LCB lcb_long

Axial Force Fxx = 0.0 N

Shear Forces Fyy = 0.0 N , Fzz = 0.0 N

Bending Moments Mx = 0.0 N·mm , My = 550000000.0 N·mm , Mz = 0.0 N·mm

x = (Rs*As-Rsc*_As)/(Rb*b) = (340.000×4825.486-340.000×226.195)÷(14.500×300.000) = 359.485 mm

h0 = h-as = 700.000-70.000 = 630.000 mm

ksi = x/h0 = 359.485÷630.000 = 0.571

ksi_R = 0.5384621 < ksi = 0.571

x = ksi_R*h0 = 0.538×630.000 = 339.231 mm

Mult = Rb*b*x*(h0-0.5*x)+Rsc*_As*(h0-_as) = 14.500×300.000×339.231×(630.000-0.5×339.231)+340.000×226.195

×(630.000-70.000) = 722435793.772 N·mm

Rat-Normal = M / M_ult = 550000000.000÷722435793.772 = 0.76131333

[0.761 ≤ 1]—OK

*** Shear Check**

1) Calculation by concrete strip between inclined sections (par. 8.1.32)

Y Dir

LCB lcb_long

Axial Force Fxx = 0.0 N

Shear Forces Fyy = 0.0 N , Fzz = 0.0 N

Bending Moments Mx = 0.0 N·mm , My = 550000000.0 N·mm , Mz = 0.0 N·mm

Phi_b1 = 0.300

Qult = Phi_b1*Rb*b*ho = 0.300×14.500×300.000×630.000 = 822150.000 N

Rat-Shear = Q/Qult = 0.000÷822150.000 = 0.00000000

[0.000 ≤ 1]—OK

Text report in DOC format.

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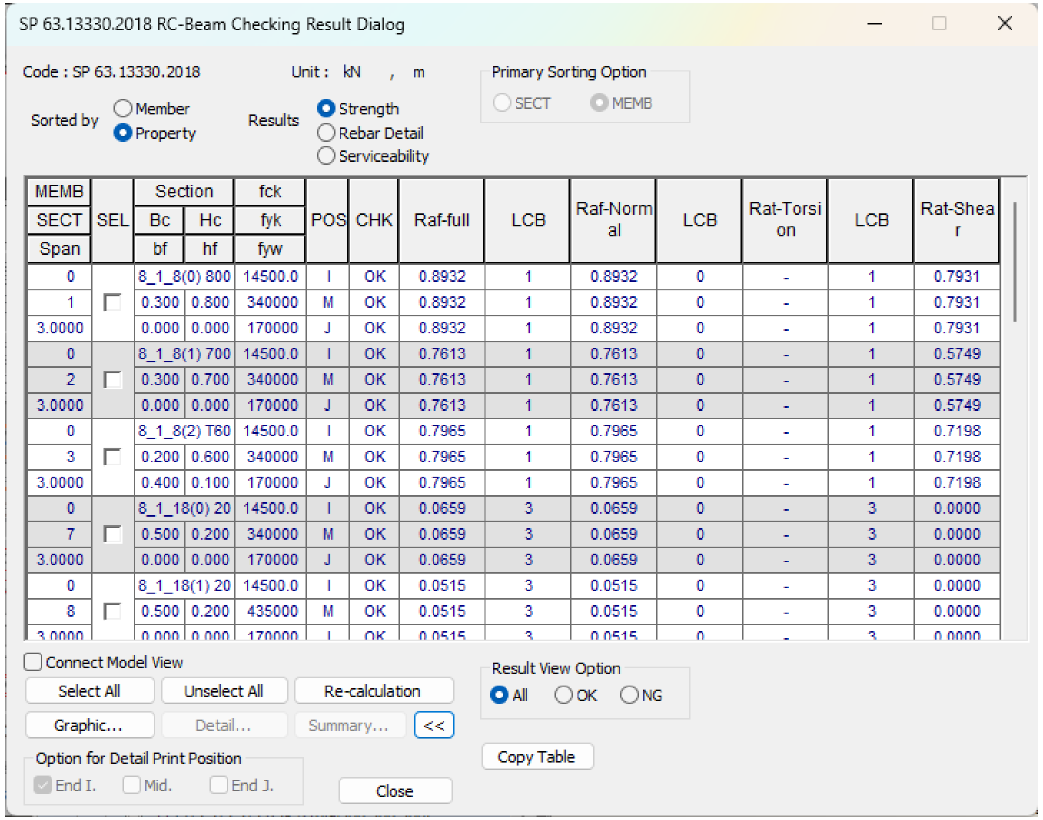
Added design of reinforced concrete structures according to SP 63.13330.2018

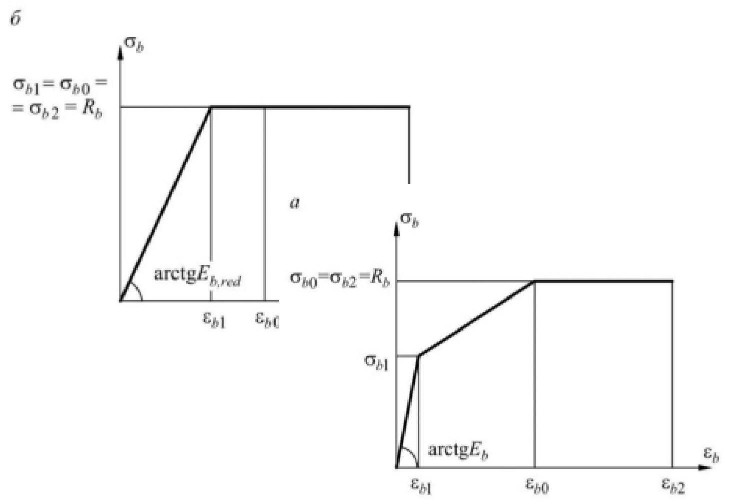
3. Calculation of crack resistance and crack opening width in the normal section of a beam element



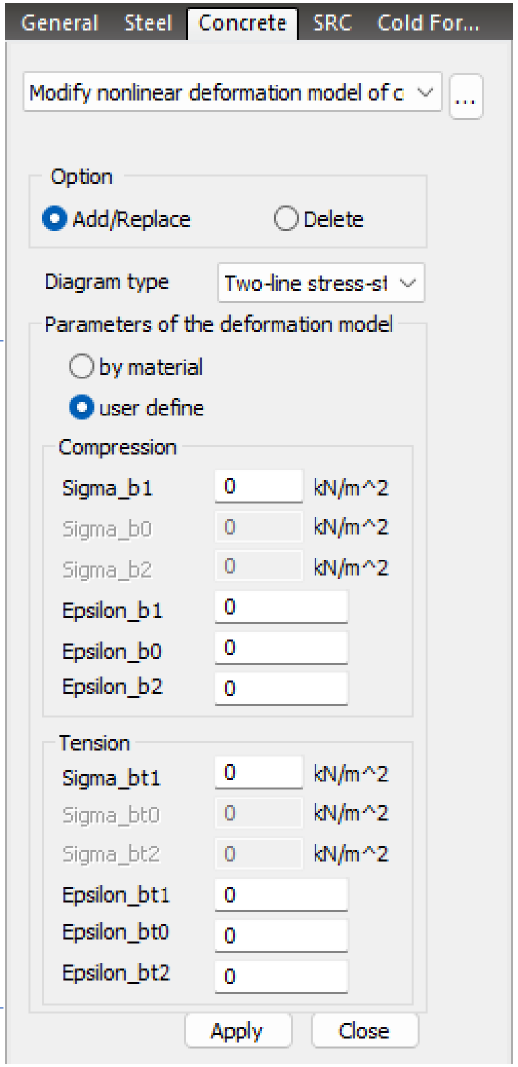
MEMB / SECT / Span	SEL	Section Bc / bf	Hc / hf	fck / fyk / fyw	POS	CHK	Raf-full	LCB	Raf-Normal	LCB	Rat-Torsion	LCB	Rat-Shear
0		8_1_8(0) 800		14500.0	I	OK	0.8932	1	0.8932	0	-	1	0.7931
1	☐	0.300	0.800	340000	M	OK	0.8932	1	0.8932	0	-	1	0.7931
3.0000		0.000	0.000	170000	J	OK	0.8932	1	0.8932	0	-	1	0.7931
0		8_1_8(1) 700		14500.0	I	OK	0.7613	1	0.7613	0	-	1	0.5749
2	☐	0.300	0.700	340000	M	OK	0.7613	1	0.7613	0	-	1	0.5749
3.0000		0.000	0.000	170000	J	OK	0.7613	1	0.7613	0	-	1	0.5749
0		8_1_8(2) T60		14500.0	I	OK	0.7965	1	0.7965	0	-	1	0.7198
3	☐	0.200	0.600	340000	M	OK	0.7965	1	0.7965	0	-	1	0.7198
3.0000		0.400	0.100	170000	J	OK	0.7965	1	0.7965	0	-	1	0.7198
0		8_1_18(0) 20		14500.0	I	OK	0.0659	3	0.0659	0	-	3	0.0000
7	☐	0.500	0.200	340000	M	OK	0.0659	3	0.0659	0	-	3	0.0000
3.0000		0.000	0.000	170000	J	OK	0.0659	3	0.0659	0	-	3	0.0000
0		8_1_18(1) 20		14500.0	I	OK	0.0515	3	0.0515	0	-	3	0.0000
8	☐	0.500	0.200	435000	M	OK	0.0515	3	0.0515	0	-	3	0.0000
3.0000		0.000	0.000	170000	J	OK	0.0515	3	0.0515	0	-	3	0.0000

Realised calculation types

- **Par. 8.2.4 - 8.2.7** - Calculation of crack resistance and crack width in a normal section of a rod element



Specifying a custom concrete deformation diagram (when calculating using a nonlinear deformation model - par. 8.1.24).



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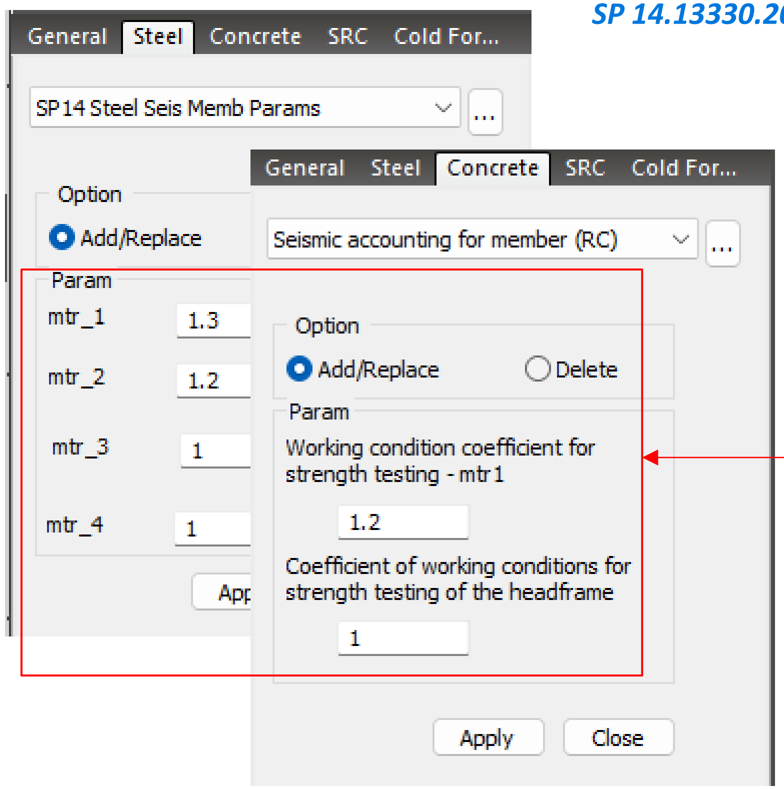
Special structural requirements are taken into account in the design of structures

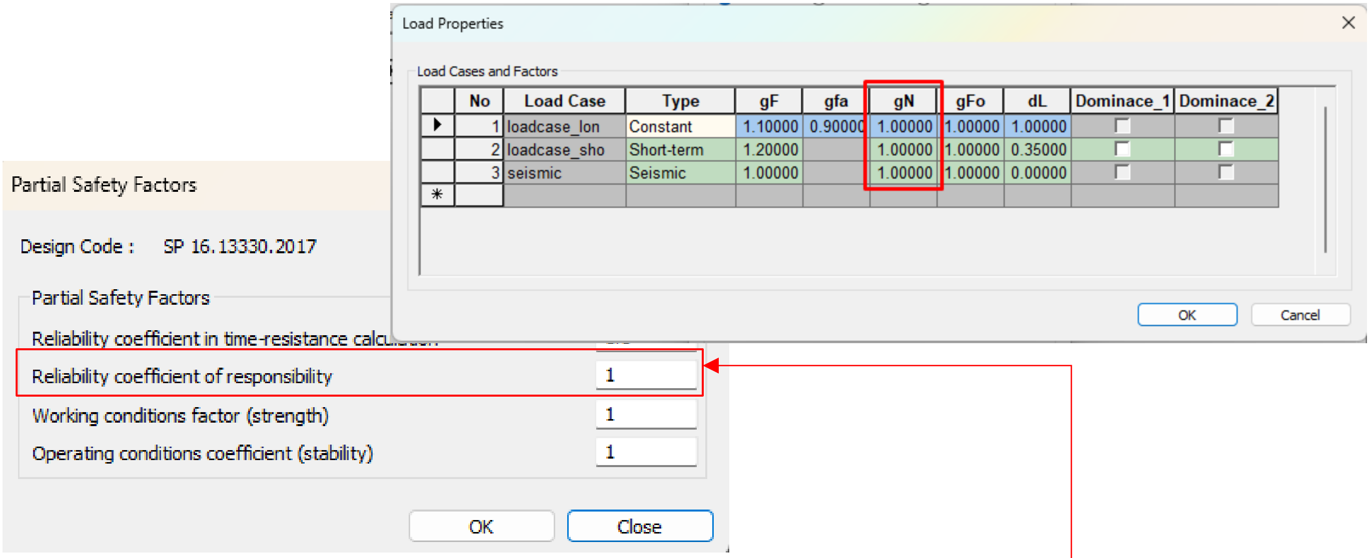
1. Added consideration of structural requirements in seismic design according to SP 14.13330.2018

2. Added accounting of responsibility of the structure according to GOST 27751-2014

SP 14.13330.2018

5.15 When calculating structures for strength and stability, in addition to the coefficients of working conditions accepted in accordance with other current regulatory documents, an additional coefficient of working conditions mtr, determined according to Table 5.4, should be introduced. The mtr coefficient is multiplied by the design resistance of the corresponding material of the structure.





GOST 27751-2014

10.1 Depending on the class and level of responsibility of structures (see 3.1), reliability coefficients for responsibility should be used in their design, the minimum values of which are given in Table 2.

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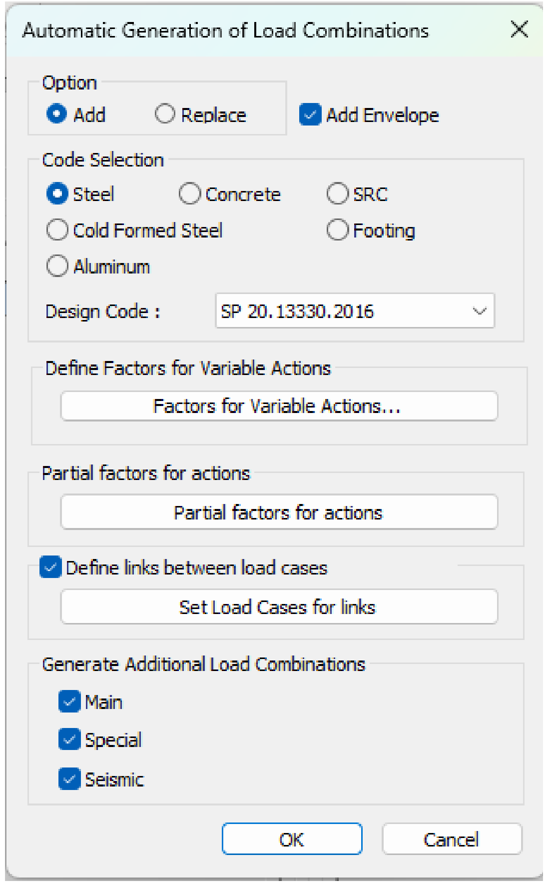
Added load combinations according to SP 20.13330.2016

1. Main, special, crane, seismic load combinations

2. Added special load combinations according to SP 296.1325800.2017



Dialog box for controlling the generation of combinations according to SP 20.13330.2016

SP 20.13330.2016

6.2 Depending on the load composition to be taken into account, a distinction should be made:

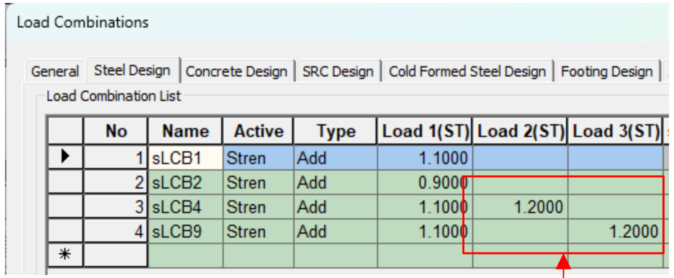
a) the main load combinations consisting of constant, long-term and short-term loads

$$C_m = P_d + (\psi_{l1}P_{l1} + \psi_{l2}P_{l2} + \psi_{l3}P_{l3} + \ldots) + (\psi_{t1}P_{t1} + \psi_{t2}P_{t2} + \psi_{t3}P_{t3} + \ldots); \quad (6.1)$$

б) special load combinations consisting of constant, long-term, short-term and one of the special loads.

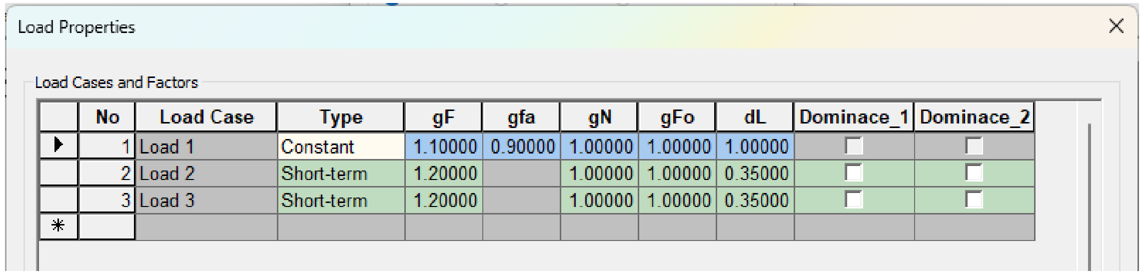
$$C_s = C_m + P_s, \quad (6.2)$$

where C_m - load for the main combination;
C_s - load for a special combination;
ψ_{li} (i = 1, 2, 3,…,) - combination coefficients for long-term loads;
ψ_{ti} (i = 1, 2, 3,…,) - combination factors for short-term loads.



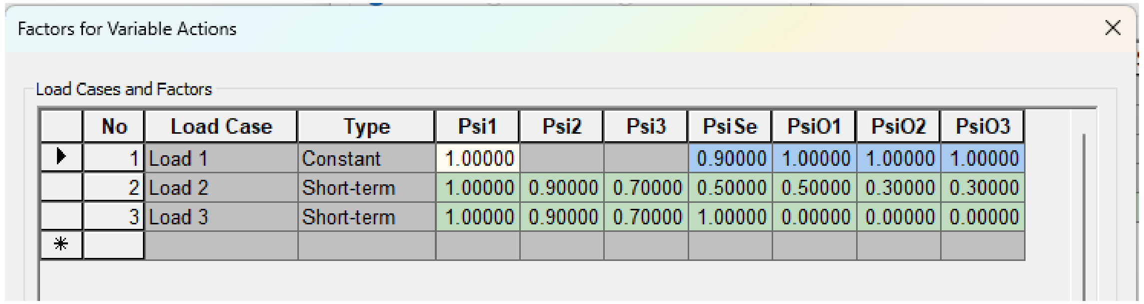
No	Name	Active	Type	Load 1(ST)	Load 2(ST)	Load 3(ST)
1	sLCB1	Stren	Add	1.1000		
2	sLCB2	Stren	Add	0.9000		
3	sLCB4	Stren	Add	1.1000	1.2000	
4	sLCB9	Stren	Add	1.1000		1.2000

The example establishes that Load 2 and Load 3 are mutually exclusive loads, so they are included in different combinations when forming the combination.



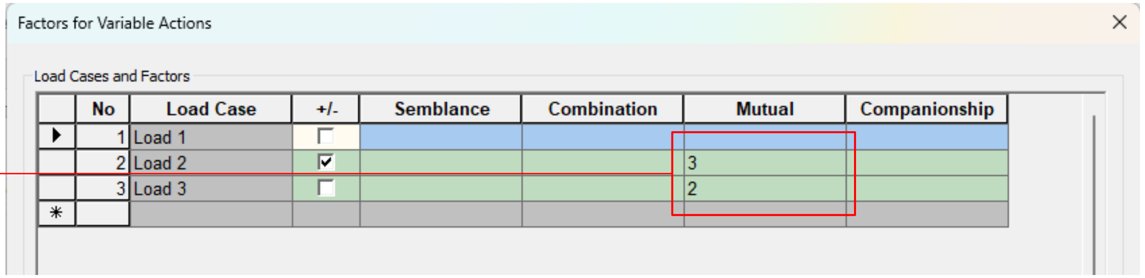
No	Load Case	Type	gF	gfa	gN	gFo	dL	Dominace_1	Dominace_2
1	Load 1	Constant	1.10000	0.90000	1.00000	1.00000	1.00000	☐	☐
2	Load 2	Short-term	1.20000		1.00000	1.00000	0.35000	☐	☐
3	Load 3	Short-term	1.20000		1.00000	1.00000	0.35000	☐	☐

Dialog box for assigning reliability coefficients



No	Load Case	Type	Psi1	Psi2	Psi3	PsiSe	PsiO1	PsiO2	PsiO3
1	Load 1	Constant	1.00000			0.90000	1.00000	1.00000	1.00000
2	Load 2	Short-term	1.00000	0.90000	0.70000	0.50000	0.50000	0.30000	0.30000
3	Load 3	Short-term	1.00000	0.90000	0.70000	1.00000	0.00000	0.00000	0.00000

Dialog box for assigning load combination factors



No	Load Case	+/-	Semblance	Combination	Mutual	Companionship
1	Load 1	☐				
2	Load 2	☑			3	
3	Load 3	☐			2	

Dialog box for assigning links and rules for forming load combinations

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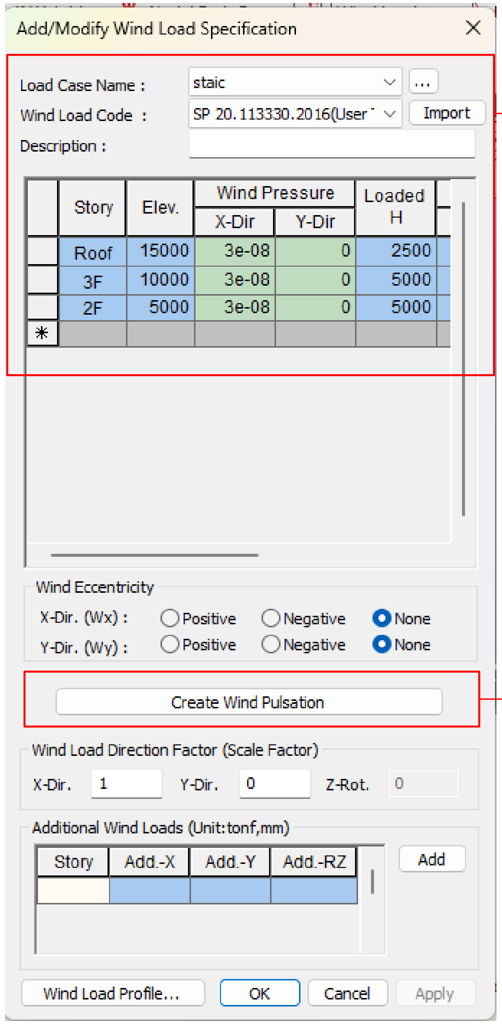
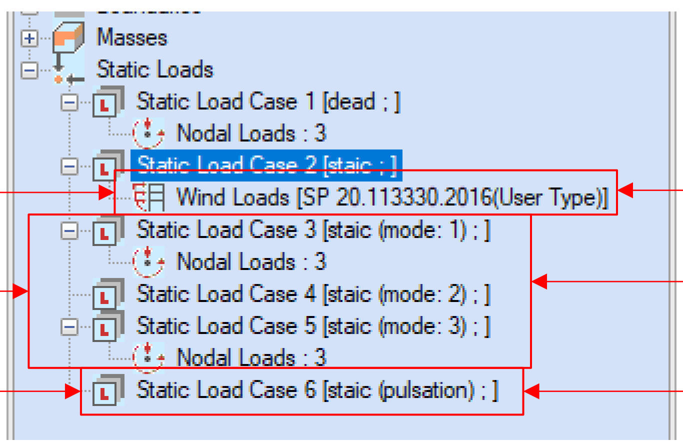
Implemented calculation of the pulsation component of the wind load according to SP 20.13330.2016

1. Calculation of the pulsation force acting on a rigid floor diaphragm

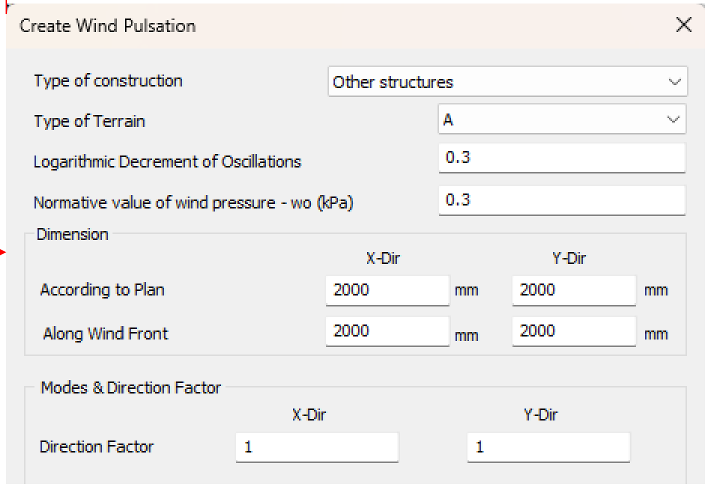


Dialog box for setting the static component of the wind load - Wm



Automatic generation of additional loads with wind pulsation according to the modes. And also formation of special loading with resulting results according to formula (11.9).

Active in result mode if eigenvalue analysis has been performed.



Dialog box for setting the pulsation component of the wind load - Wg

SP 20.13330.2016

11.1.2* In all cases, the standard value of the basic wind load w should be defined as the sum of the average wm and pulsation wg components

$$w = w_m + w_g \quad (11.1)$$

11.1.8* The standard value of the pulsation component of the main wind load wg at the equivalent height ze should be determined as follows:

a) for structures (and their structural elements) with the first frequency of natural vibrations f1, Hz, greater than the limit value of natural frequency flim, (see 11.1.10) - by the formula

$$w_g = w_m \zeta(z_e) v, \quad (11.5)$$

b) for all structures (and their structural elements) with f1 < flim < f2 - according to the formula

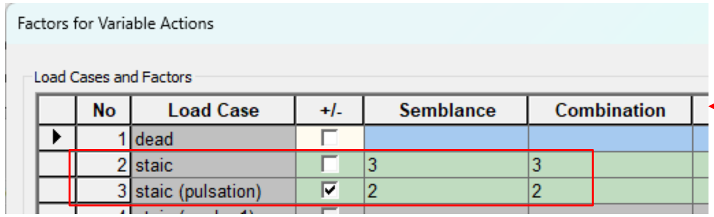
$$w_g = w_m \xi \zeta(z_e) v, \quad (11.7)$$

c) for structures with the second natural frequency less than the limiting frequency, it is necessary to carry out the dynamic calculation taking into account s of the first natural vibration forms. The number s should be determined from the condition fs < flim < fs+1;

d) when designing buildings, it is allowed to take into account the dynamic response of the three lowest natural vibration modes (two bending and one torsional or mixed torsional-bending).

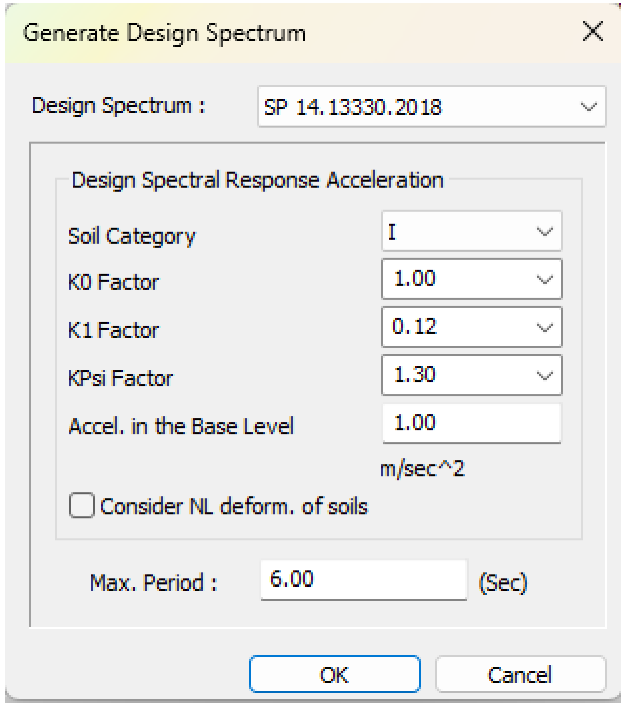
11.1.9 The forces and displacements when considering the dynamic reaction by s eigenforms are determined by the formula

$$X^2 = \sum X_s^2, \quad (11.9)$$

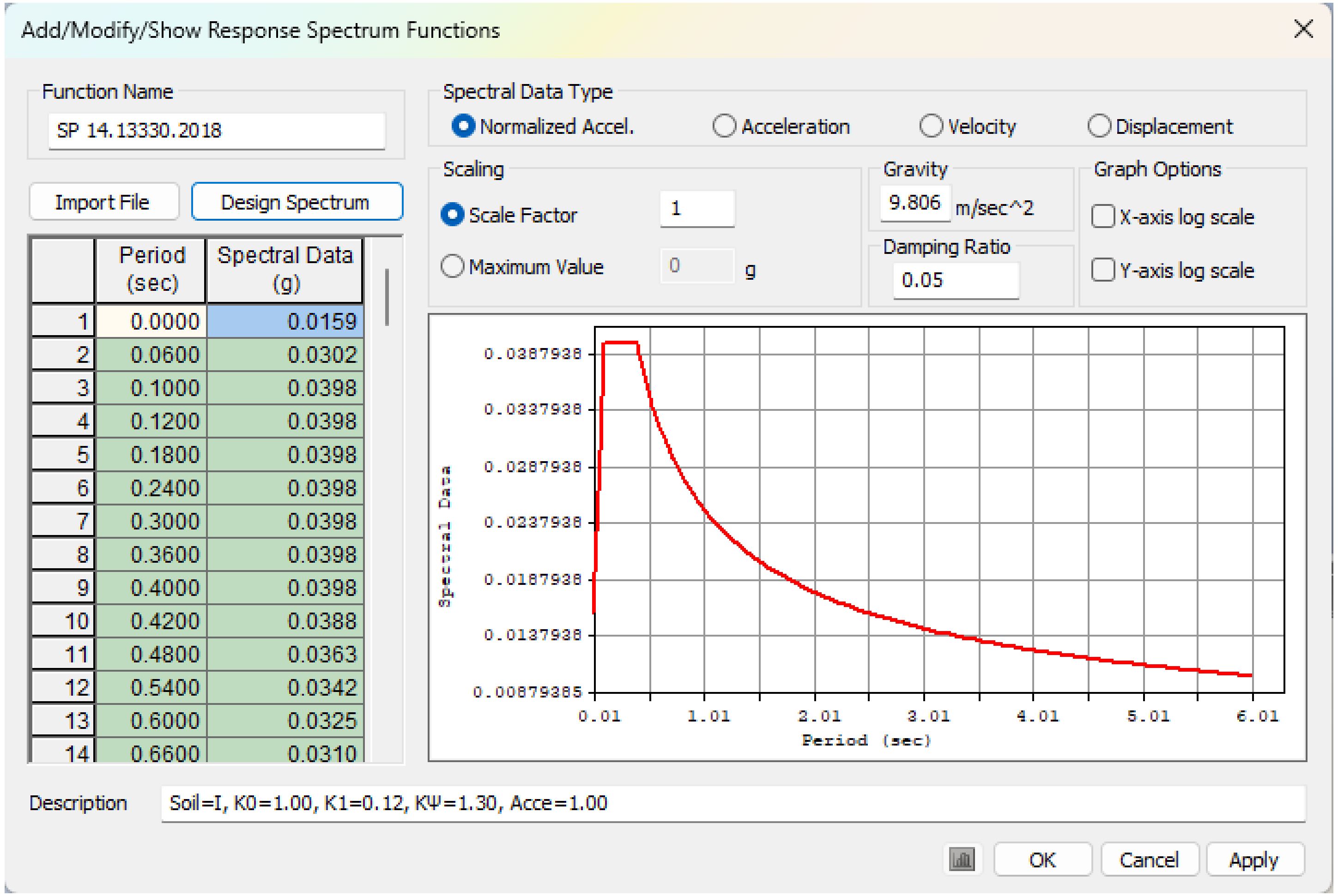


Creation of total wind load

Added response spectrum according to SP 14.13330.2018



Dialog box for generating a response spectrum in accordance with SP 14.13330.2018



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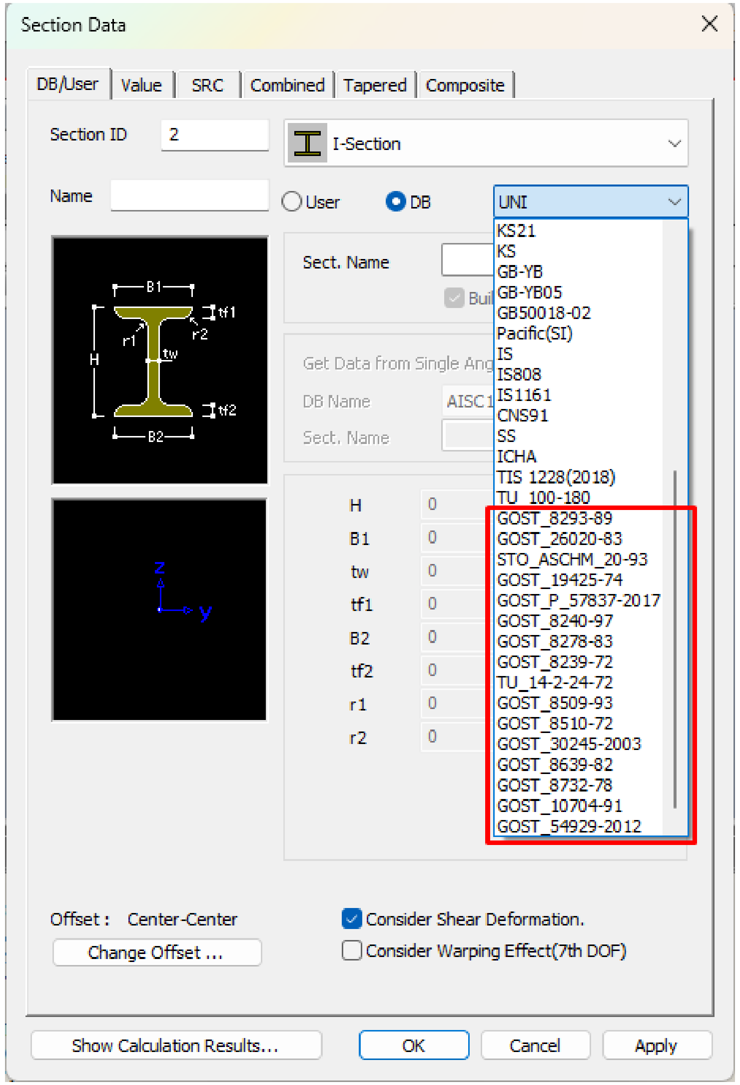
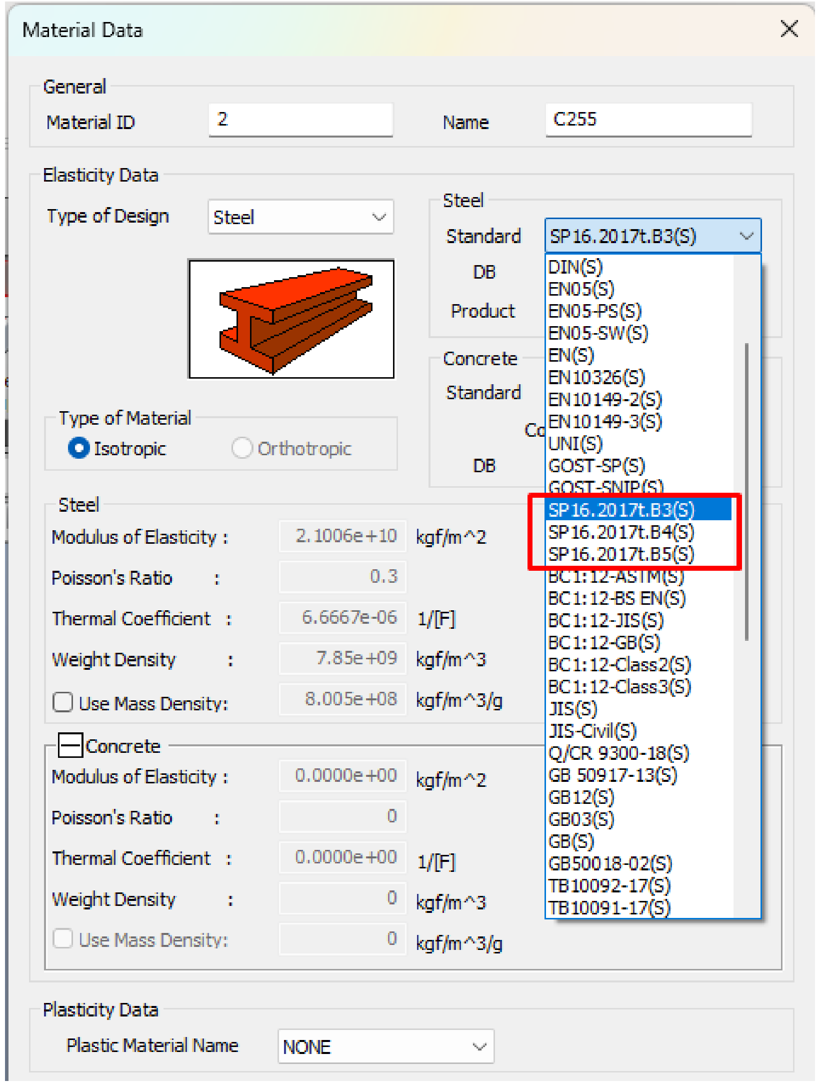
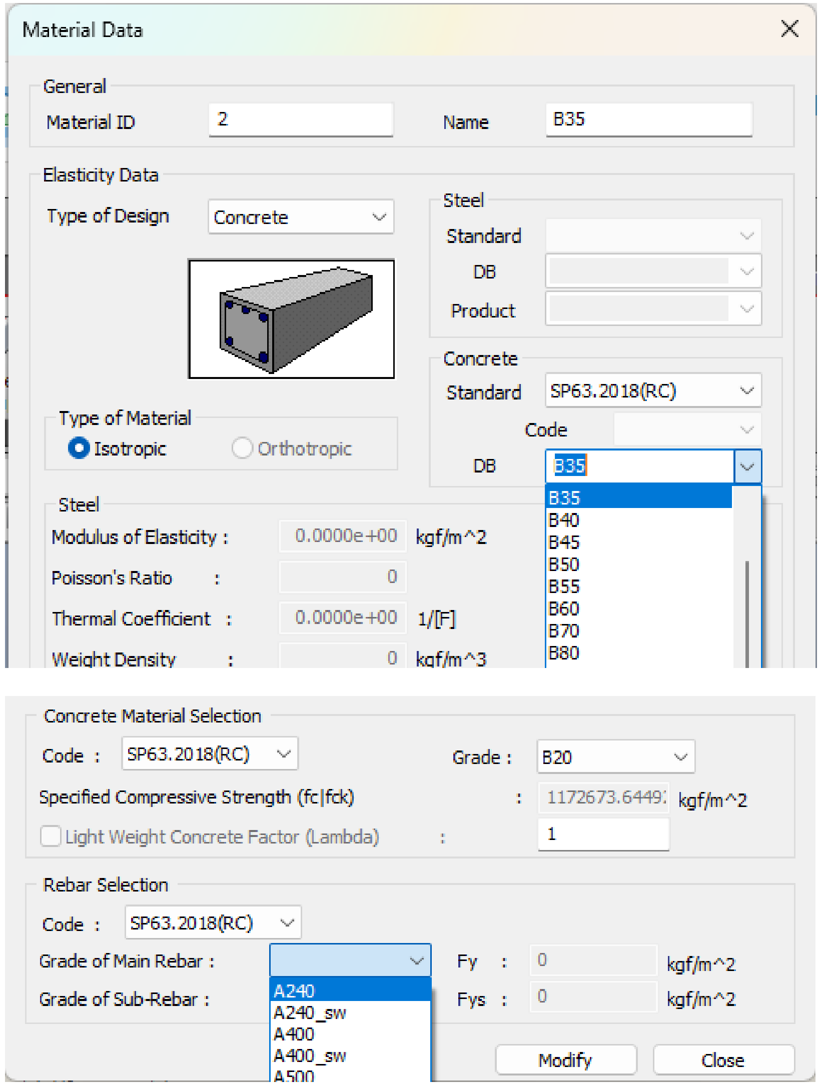
Added base of materials and profiles for calculation according to Russian code



Additional sections of rolled steel sections



Materials of steel elements according to table 3,4,5 SP 16.13330.2017



Materials of reinforced concrete structures according to SP 63.13330.2018

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Thanks