



**STRUCTURAL
DESING**

MEMORIA DE CÁLCULO

PLAZA DEL PARQUE

REVO

18 MARZO / 2023

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1. Utilidad que se le va a dar a la estructura.

Edificación destinada a uso comercial (almacenes, restaurantes) y áreas de reunión.

2. Descripción del Proyecto.

Sentido X / Y._ Pórtico a momento del tipo ESPECIAL de acero (SMF), $R=8$ (Conservadoramente se utiliza $R=7$).

SMF (Special moment frames).

Son pórticos detallados y diseñados para responder con alto grado de ductilidad.

7.2.1.1 SMF Design

Steel SMFs, permitted to be designed using a value of $R = 8$, are expected to sustain multiple cycles of significant inelastic response when subjected to design-level ground motion. However, many steel SMF structures have substantial overstrength. This overstrength results from oversizing of columns to meet strong-column/weak-beam criteria, use of oversize sections to provide sufficient stiffness for drift control, and variability in the strength of the steel material itself. As a result, although the R value of 8 specified by the code would imply initiation of inelastic behavior shaking with an intensity $1/8$ that of the design earthquake, many steel SMF structures are likely to remain elastic at an intensity of $1/3$ that of the design earthquake. Therefore, in a manner of speaking, the effective R value for an SMF may be as little as 3.

Es importante mencionar que muchas de las estructuras que poseen Pórticos especiales a momento (SMF), suelen permanecer elásticas a una intensidad de $1/3$ del sismo de diseño. Es decir, el valor efectivo de R para un pórtico a momento del tipo especial (SMF) puede ser incluso tan pequeño como 3.

3. Cálculo de Cargas.

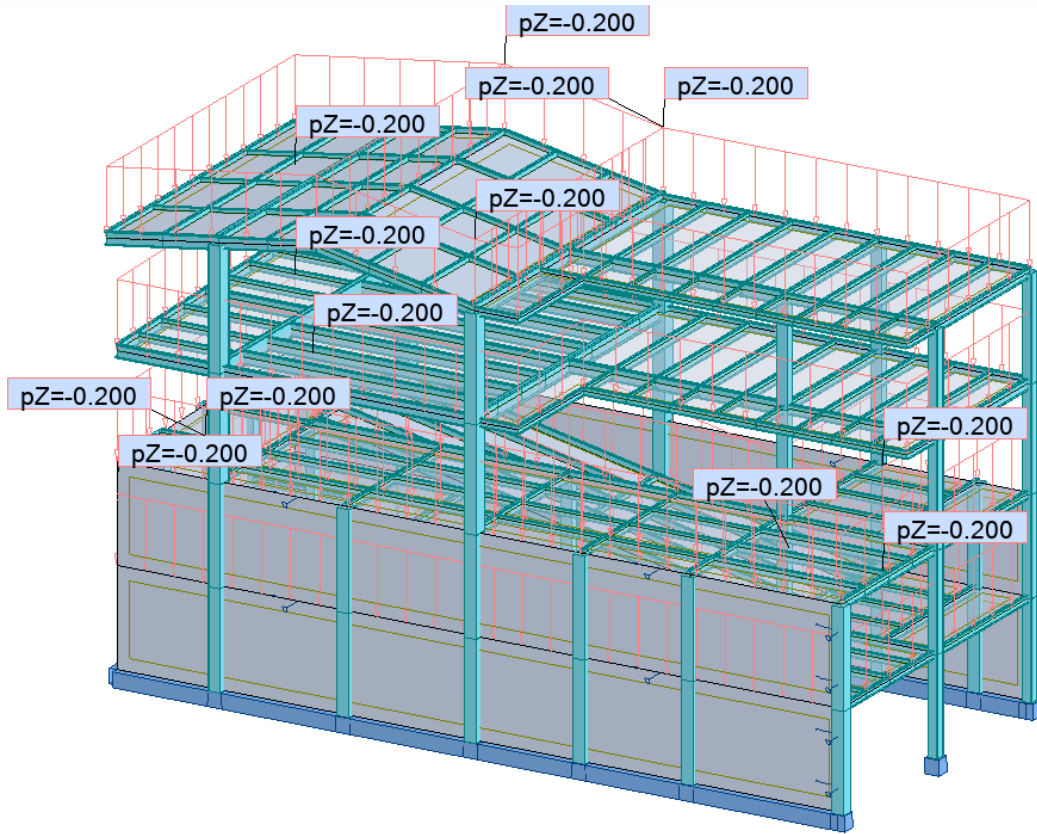
a. Carga muerta.

- Peso propio de la estructura.

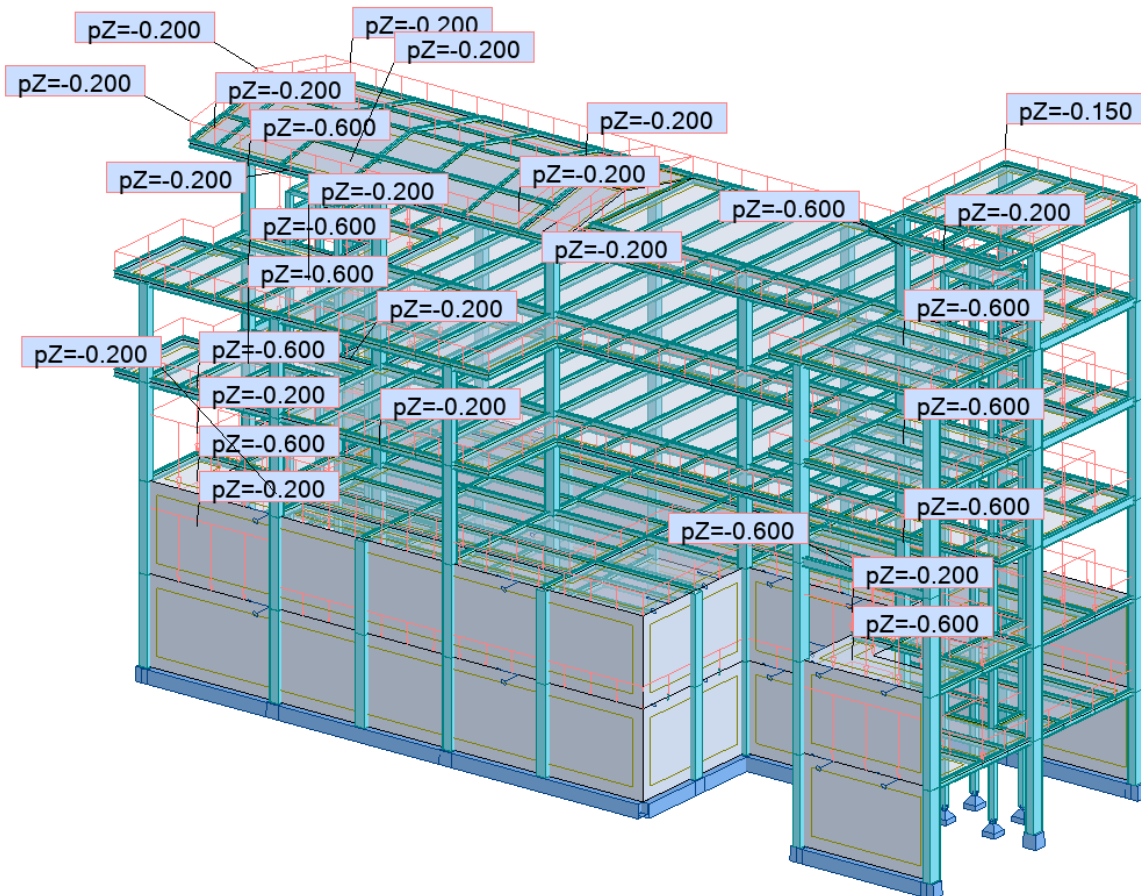
Cálculo realizado por el software de análisis estructural. Considerando 2.40t/m^3 para el peso volumétrico del hormigón armado, 7.85t/m^3 para el peso volumétrico del acero.

- 200 kg/m^2 , correspondientes al peso de la tabiquería, vidrierías divisorias y acabados.
- 150 kg/m^2 , correspondiente al peso de los acabados / paneles sobre losas inaccesibles.

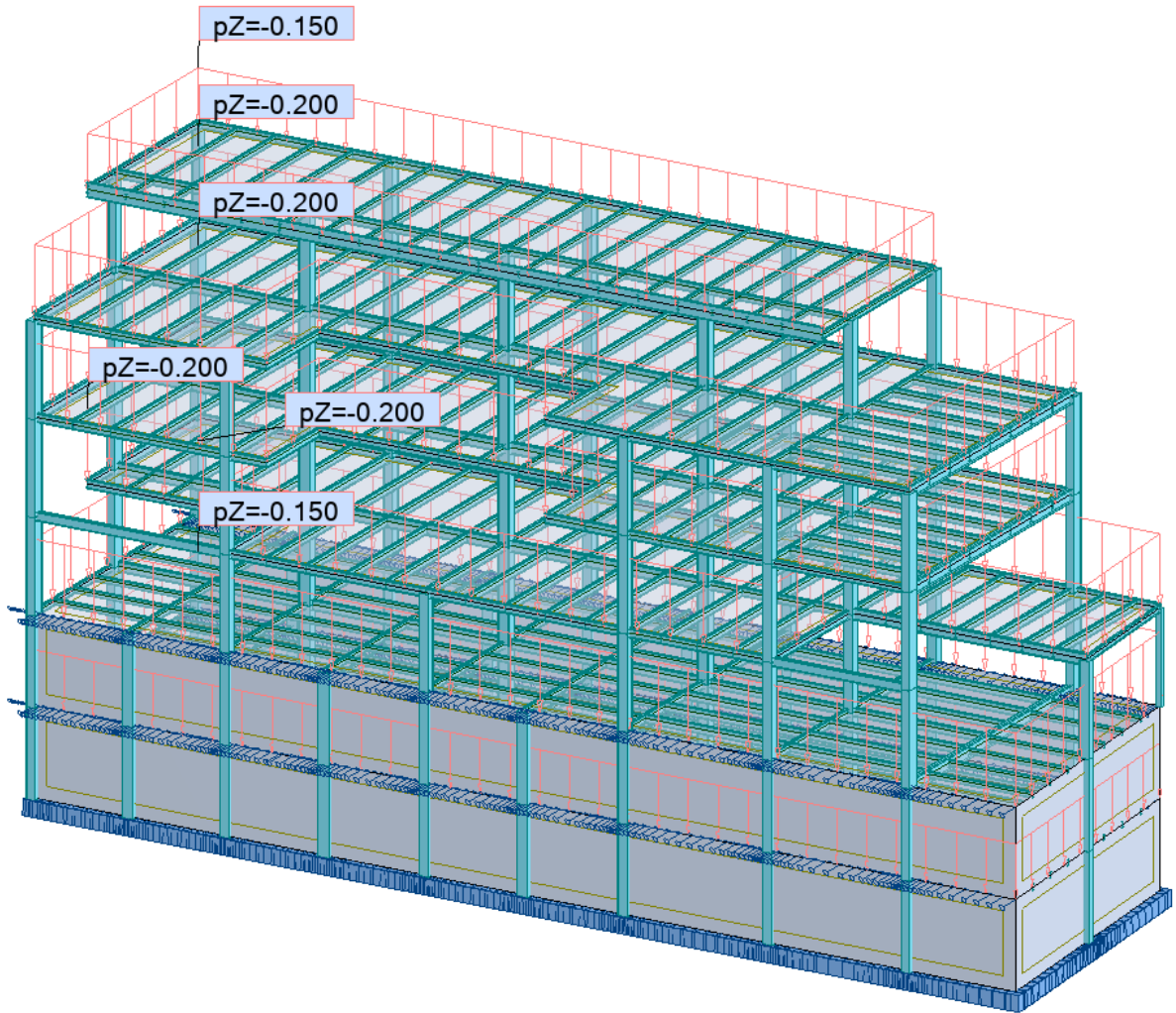
- ET1.



- ET2.



- ET3.



b. Carga viva: (Según Tabla 9 NEC-SE-CG)

ASCE7-16 Table4.3-1:

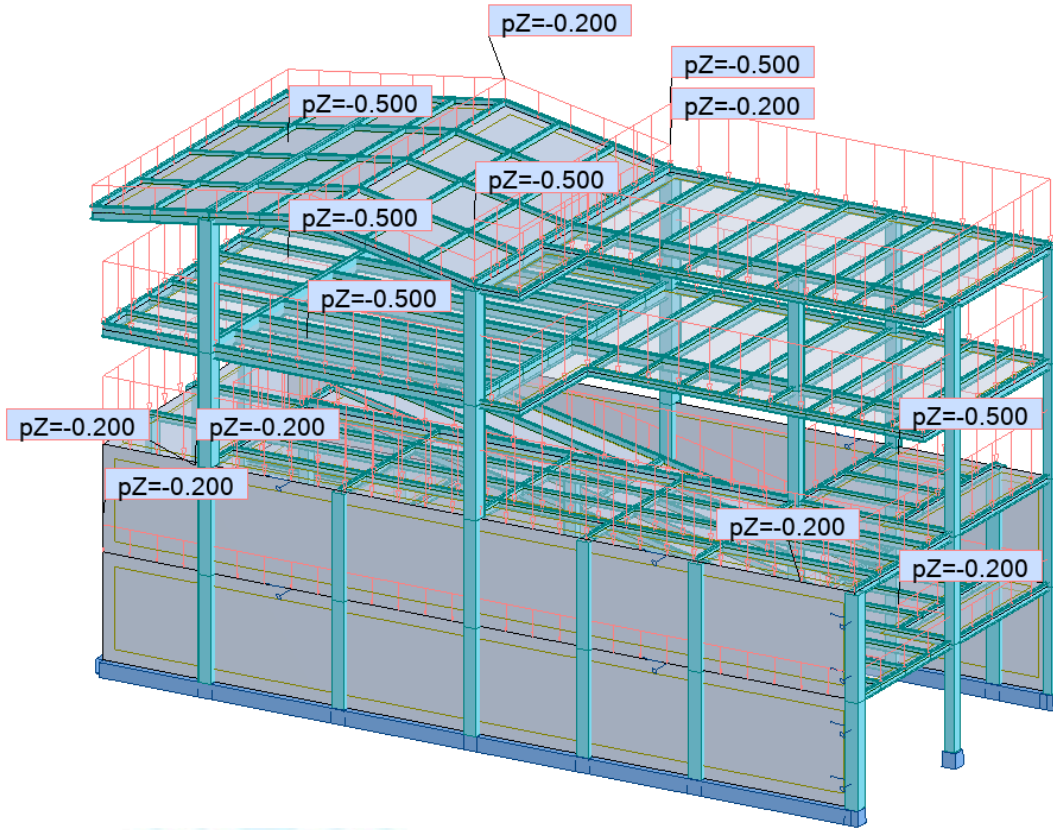
Stores → Upper floors = 3.59Kn/m^2 .

Dining rooms and restaurants = 4.79Kn/m^2 .

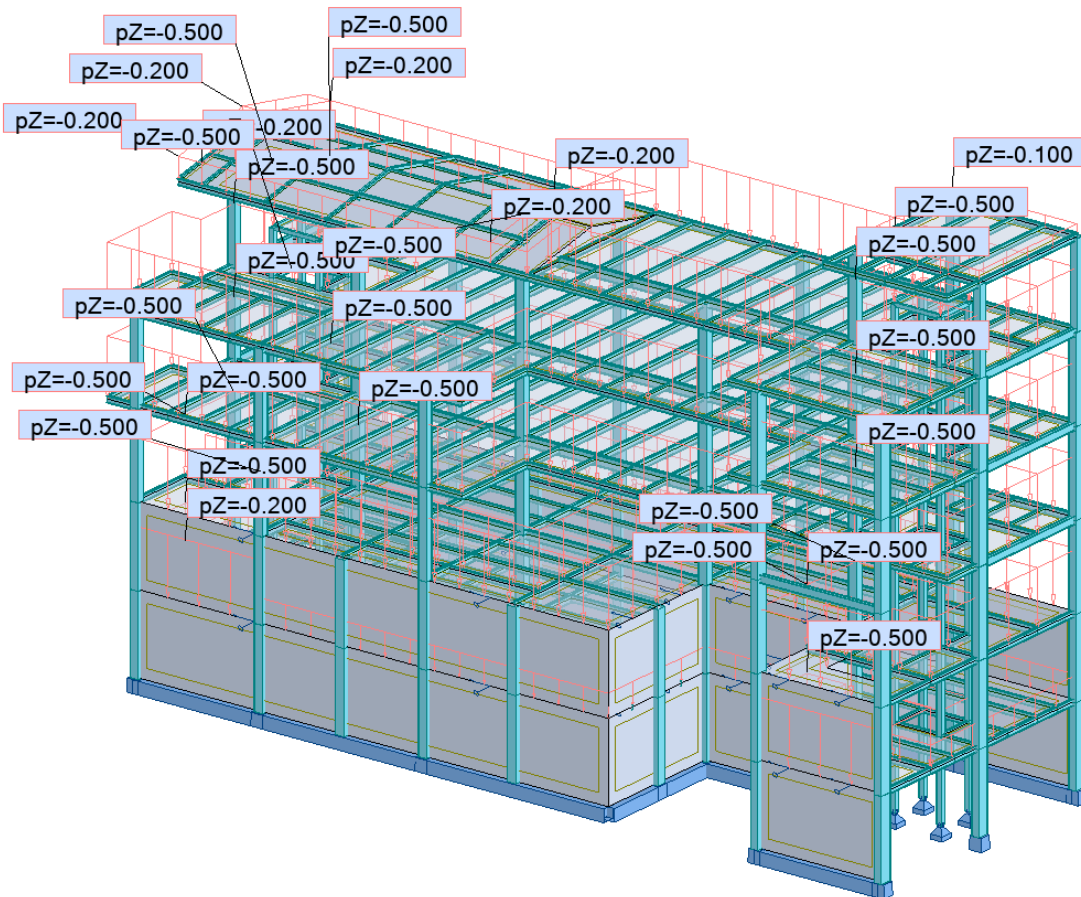
Roofs → occupancy → recreational uses / Public roofs = 4.79Kn/m^2 .

- Restaurantes, 500 kg/m^2 .
- Almacenes, 350 kg/m^2 , se procede a usar 500 kg/m^2 .
- Cubiertas para patios de reunión, 500 kg/m^2 .
- Cubiertas accesibles, 200 kg/m^2 .
- Cubierta inaccesible (pérgola), 100 kg/m^2 .

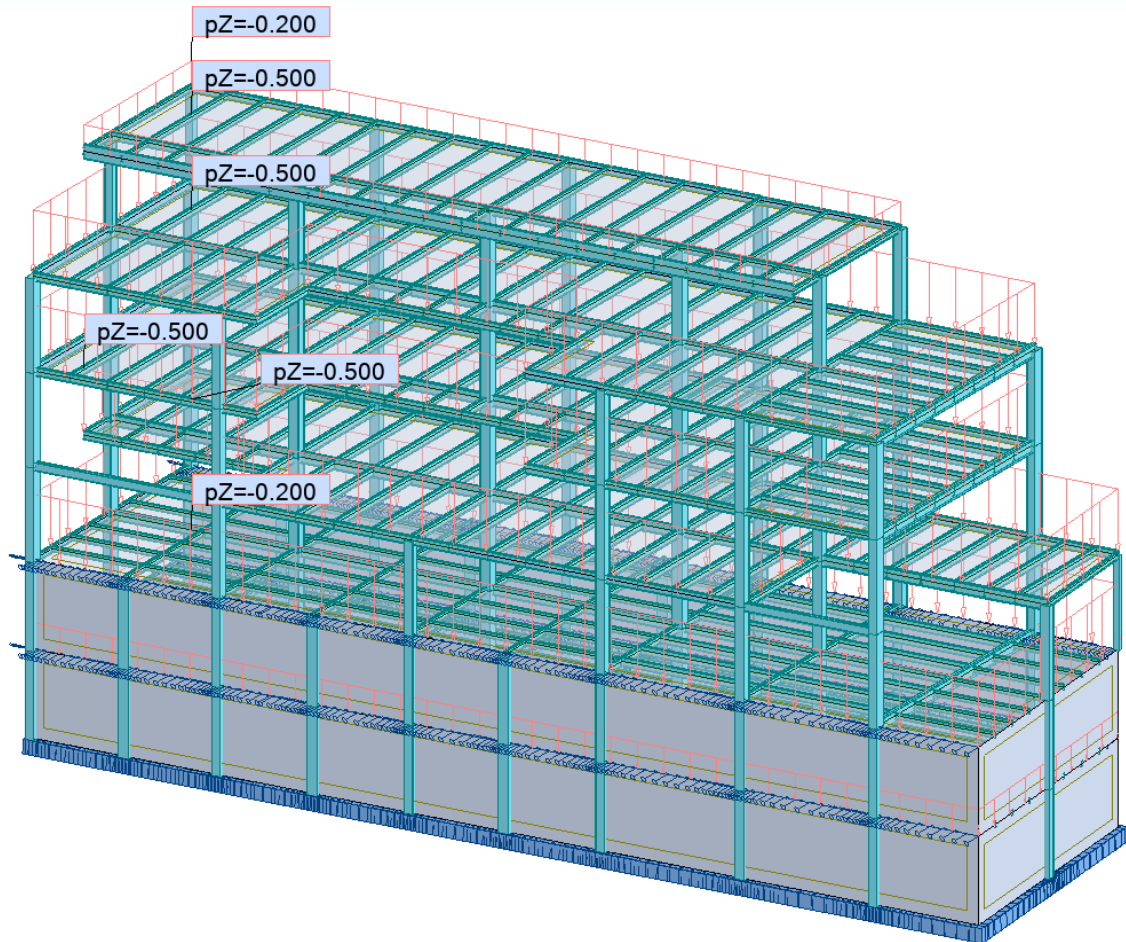
- ET1.



- ET2.



- ET3.



4. Parámetros Sísmicos.

4.1. Caracterización del estrato.

SUELO TIPO C/D. → Varía en el cálculo de corte basal de cada estructura para obtener la mayor demanda sísmica para el periodo en que se encuentra la estructura.

4.2. Aceleraciones espectrales de diseño.

- Pichincha → Quito → Cumbayá → $S_S=2.04$; $S_I=0.82$ → ASCE7-16.
- S_S → MCE_R (Risk-Targered Maximium Considered Earthquake) aceleración espectral de diseño a periodo corto (0.2seg). Para el 1% de probabilidad de colapso en 50 años.

En versiones anteriores a la del ASCE7-10, esta aceleración correspondía a aquella obtenida del 2% de probabilidad de excedencia en 50 años (2475 años como período de retorno).

- S_I → MCE_R (Risk-Targered Maximium Considered Earthquake) aceleración espectral de diseño a periodo largo (1seg). Para el 1% de probabilidad de colapso en 50 años.

En versiones anteriores a la del ASCE7-10, esta aceleración correspondía a aquella obtenida del 2% de probabilidad de excedencia en 50 años (2475 años como período de retorno).

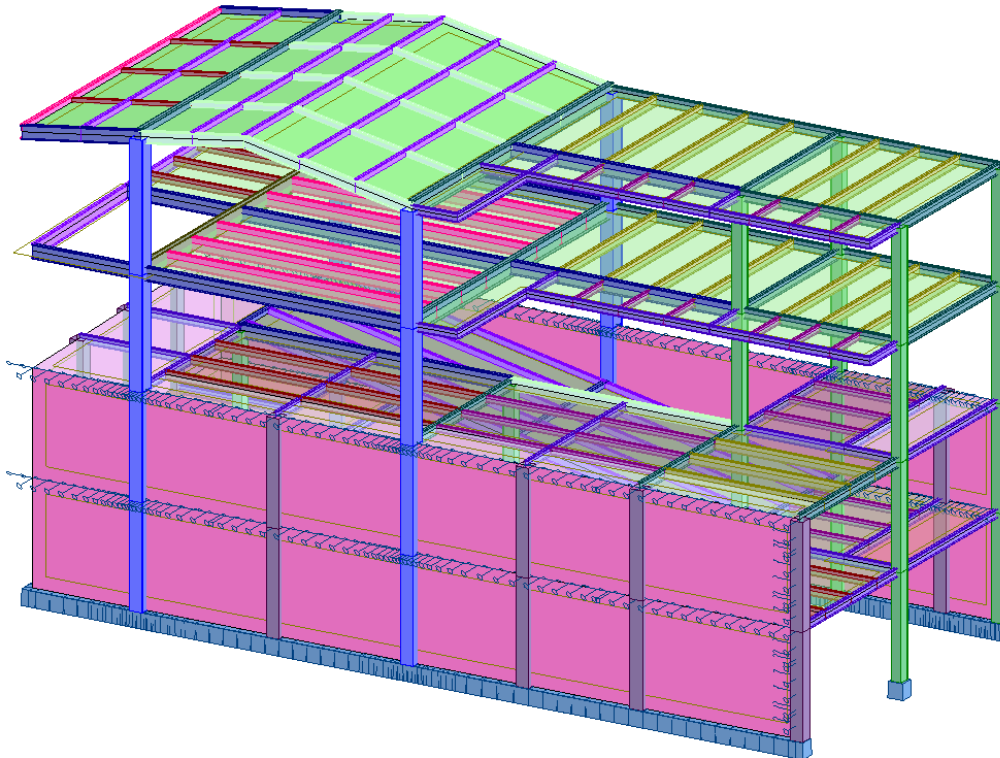
4.3. Factor de Importancia de la estructura.

- Estructura de ocupación normal.
 - $I=1.00$ → ASCE7-16.

5. Parámetros ingresados al programa de análisis estructural.

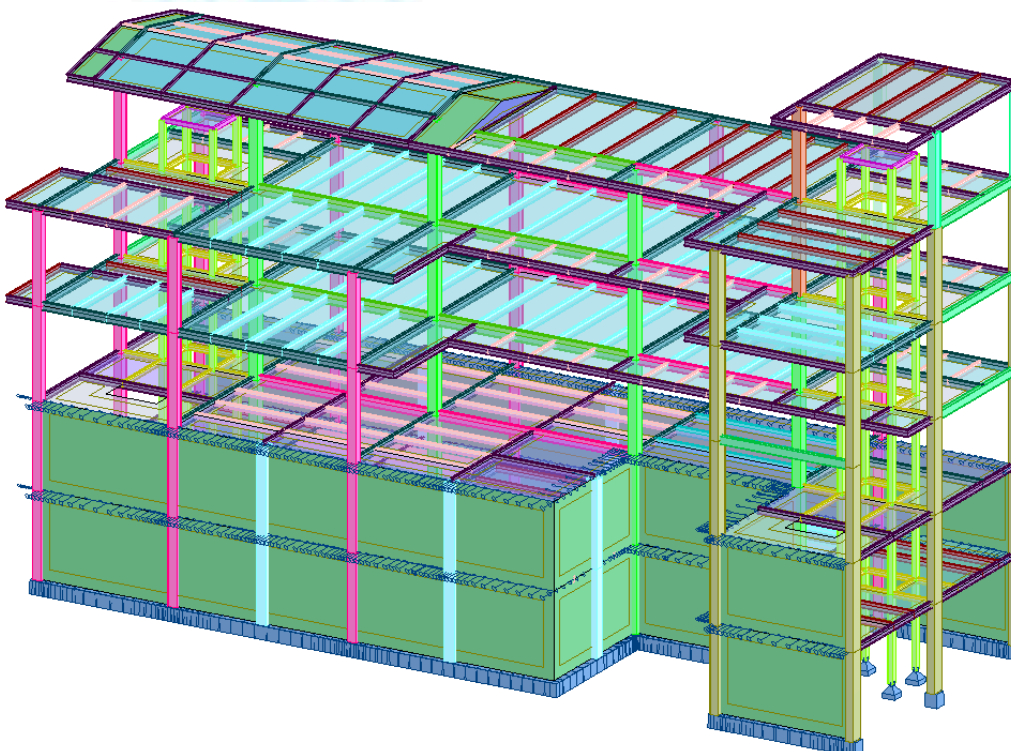
5.1. Modelo con descripción de los elementos.

- ET1



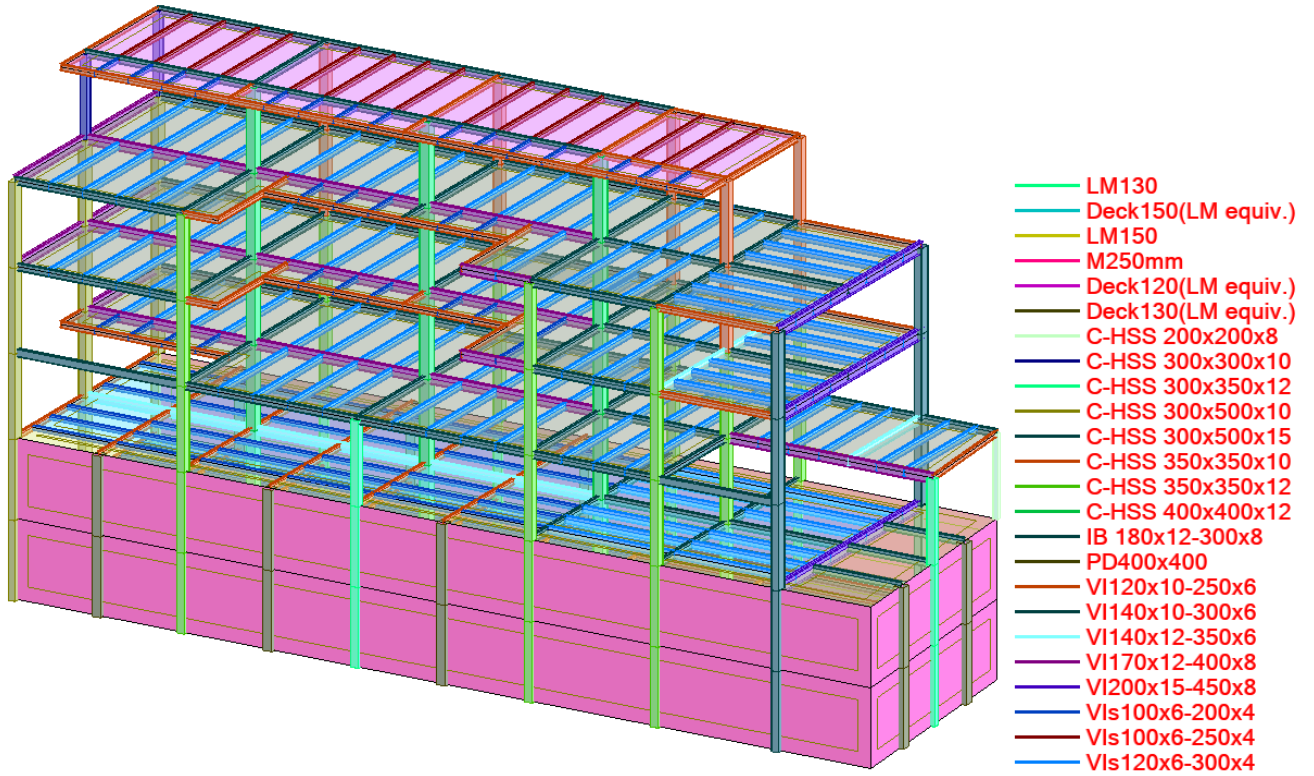
- Deck150(LM equiv.)
- LM130
- LM150
- M250mm
- Deck130(LM equiv.)
- C-HSS 300x350x12
- C-HSS 400x500x15
- IB 200x18-500x12
- PD400x400
- VI120x10-250x6
- VI140x10-300x6
- VI140x12-350x6
- VI170x12-400x8
- VI200x18-500x12
- VIs100x6-200x4
- VIs100x6-250x4
- VIs120x6-300x4
- VIs120x8-400x4
- VIs140x10-400x4

- ET2



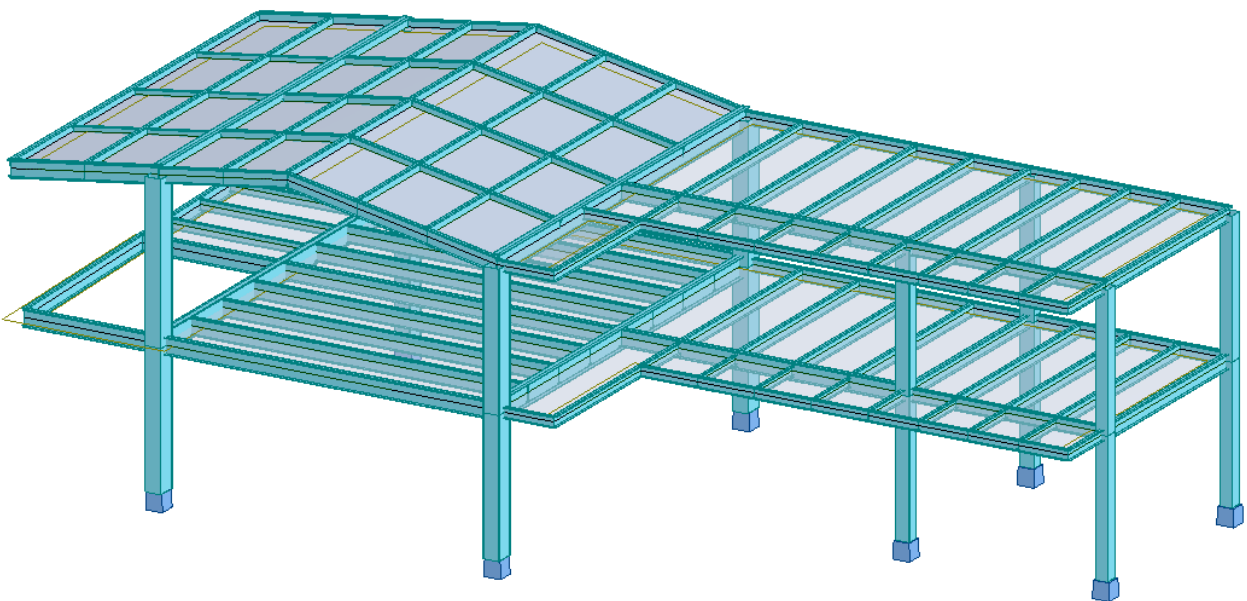
- Deck120(LM equiv.)
- LM13cm
- Deck130(LM equiv.)
- M150mm
- LM15
- M250mm
- Deck150(LM equiv.)
- C-HSS 200x200x8
- C-HSS 300x300x10
- C-HSS 300x350x10
- C-HSS 300x350x12
- C-HSS 300x500x10
- C-HSS 350x350x10
- C-HSS 350x350x12
- IB 180x12-300x8
- PD400x400
- TR100x150x4
- VI100x8-200x4
- VI120x10-250x6
- VI140x10-300x6
- VI140x12-350x6
- VI170x12-400x8
- VIs100x6-200x4
- VIs100x6-250x4
- VIs120x6-300x4
- VIs120x8-300x4

- ET3.

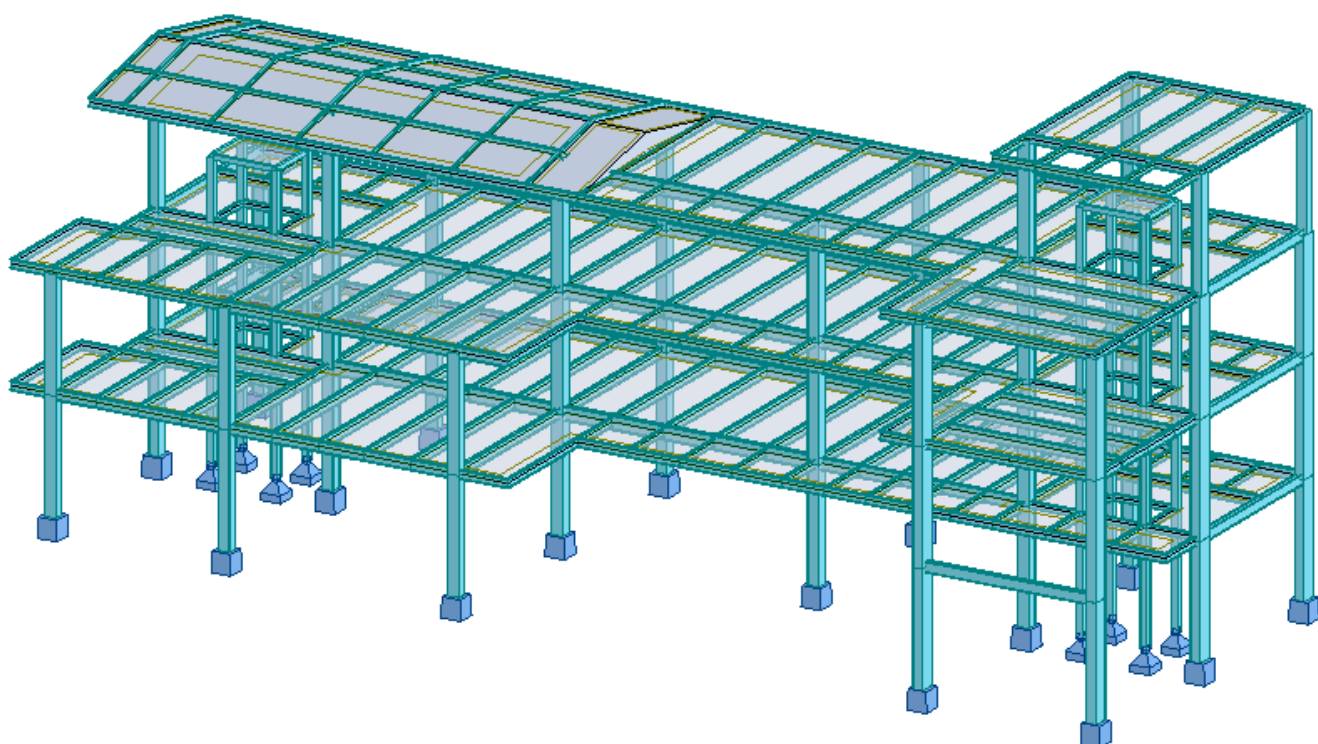


5.1. Modelo de la parte vibrante de la estructura.

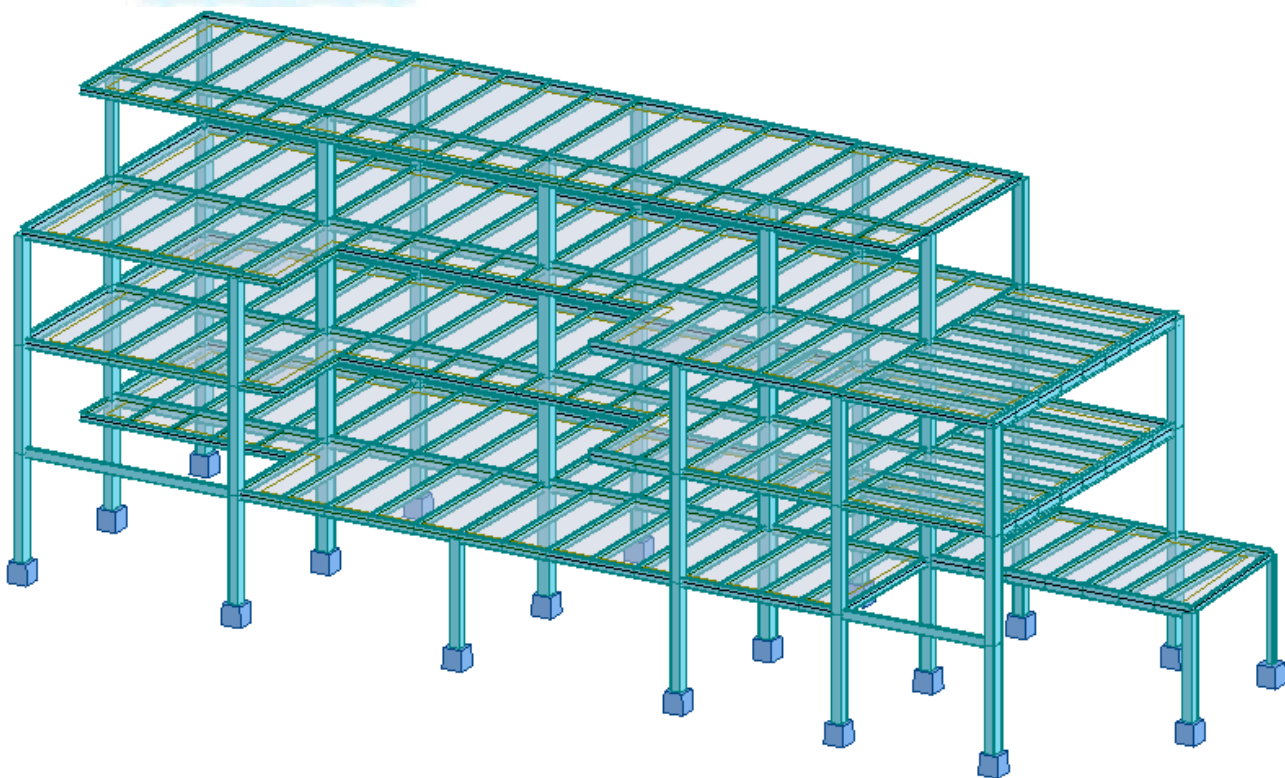
- ET1.



- ET2.



- ET3.



5.2. Descripción y propiedades de materiales estructurales.

Name: Description:

Elasticity		Resistance	
Young modulus, E:	<input type="text" value="253456.58"/> (kG/cm ²)	Characteristic	<input type="text" value="281.228"/> (kG/cm ²)
Poisson ratio, v:	<input type="text" value="0.167"/>	Sample:	<input type="text" value="Cylindrical"/>
Shear modulus, G:	<input type="text" value="108624.25"/> (kG/cm ²)		
Specific weight (unit weight):	<input type="text" value="2.407"/> (T/m ³)		
Thermal expansion coefficient:	<input type="text" value="0.000010"/> (1/°C)		
Damping ratio:	<input type="text" value="0.15"/>		

Name: Description:

Elasticity		Resistance	
Young modulus, E:	<input type="text" value="2038901.7"/> (kG/cm ²)	Characteristic	<input type="text" value="3515.348"/> (kG/cm ²)
Poisson ratio, v:	<input type="text" value="0.3"/>	Reduction factor for shear:	<input type="text" value="1.66"/>
Shear modulus, G:	<input type="text" value="784203.80"/> (kG/cm ²)	Limit strength for tension:	<input type="text" value="4569.952"/> (kG/cm ²)
Specific weight (unit weight):	<input type="text" value="7.849"/> (T/m ³)	<input type="checkbox"/> Annealed steel	
Thermal expansion coefficient:	<input type="text" value="0.000012"/> (1/°C)		
Damping ratio:	<input type="text" value="0.06"/>		

Name: Description:

Elasticity		Resistance	
Young modulus, E:	<input type="text" value="2038901.7"/> (kG/cm ²)	Characteristic	<input type="text" value="3234.120"/> (kG/cm ²)
Poisson ratio, v:	<input type="text" value="0.3"/>	Reduction factor for shear:	<input type="text" value="1.66"/>
Shear modulus, G:	<input type="text" value="784203.80"/> (kG/cm ²)	Limit strength for tension:	<input type="text" value="4077.804"/> (kG/cm ²)
Specific weight (unit weight):	<input type="text" value="7.849"/> (T/m ³)	<input type="checkbox"/> Annealed steel	
Thermal expansion coefficient:	<input type="text" value="0.000012"/> (1/°C)		
Damping ratio:	<input type="text" value="0.06"/>		

5.3. Secciones introducidas al modelo estructural.

Section type: Steel (Deg)
 Material: STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: VI170x12-400x8
 Color: Auto

Dimensions (cm)
 b = 17.000
 hw = 37.600
 tw = 0.800
 tf = 1.200

Elasto-plastic analysis

Section type: Steel (Deg)
 Material: EQQ_STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: C-HSS 300x300x10
 Color: Auto

Dimensions (cm)
 b = 30.000
 h = 30.000
 t = 1.000

Solid
 Elasto-plastic analysis

Section type: Steel (Deg)
 Material: STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: VI140x10-300x6
 Color: Auto

Dimensions (cm)
 b = 14.000
 hw = 28.000
 tw = 0.600
 tf = 1.000

Elasto-plastic analysis

Section type: Steel (Deg)
 Material: EQQ_STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: C-HSS 400x400x12
 Color: Auto

Dimensions (cm)
 b = 40.000
 h = 40.000
 t = 1.200

Solid
 Elasto-plastic analysis

Section type: Steel (Deg)
 Material: STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: IB 180x12-300x8
 Color: Auto

Dimensions (cm)
 b = 18.000
 hw = 27.600
 b1 = 10.400
 tw = 0.800
 tf = 1.200

Elasto-plastic analysis

Section type: Steel (Deg)
 Material: EQQ_STEEL A572-50

Standard Parametric Tapered Compound Special Ax, Iy, Iz ...

Label: C-HSS 300x350x12
 Color: Auto

Dimensions (cm)
 b = 30.000
 h = 35.000
 t = 1.200

Solid
 Elasto-plastic analysis

5.4. Combinaciones de Carga.

ASCE7-16.

- | | | |
|---|--------------------|-----------|
| 1. $1.4D$ | $E_h = \rho Q_E$ | (12.4-3) |
| 2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$ | | |
| 3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$ | $E = E_h - E_v$ | (12.4-2) |
| 4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$ | | |
| 5. $0.9D + 1.0W$ | $E_v = 0.2S_{DS}D$ | (12.4-4a) |
| 6. $1.2D + E_v + E_h + L + 0.2S$ | | |
| 7. $0.9D - E_v + E_h$ | | |

Ya que las cargas aplicadas son menores a 4.79 kN/m² (100psf) (500 kg/m²→valor redondeado para afectos de diseño) y además NO corresponden a garajes ni áreas de ocupación denominadas como “public assembly”, se procede a usar $0.5L$, para las combinaciones 3 y 4 acorde a ASCE7-16 2.4.1.

6. Descripción de resultados del análisis de la estructura.

6.1. Modos de vibración y periodo dinámico de la estructura.

- ET1

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Total mass UX (kg)	Total mass UY (kg)
4/ 1	2.428	0.412	74.635	0.034	74.635	0.034	137552.311	137552.311
4/ 2	3.060	0.327	74.989	65.883	0.353	65.849	137552.311	137552.311
4/ 3	3.389	0.295	75.905	80.463	0.916	14.580	137552.311	137552.311
4/ 4	3.468	0.288	78.070	80.501	2.165	0.038	137552.311	137552.311
4/ 5	4.296	0.233	78.759	80.501	0.689	0.001	137552.311	137552.311
4/ 6	4.865	0.206	78.815	80.503	0.056	0.001	137552.311	137552.311
4/ 7	5.450	0.183	78.930	80.546	0.115	0.043	137552.311	137552.311
4/ 8	5.521	0.181	78.950	80.566	0.020	0.020	137552.311	137552.311
4/ 9	6.377	0.157	79.073	80.568	0.123	0.003	137552.311	137552.311
4/ 10	6.611	0.151	79.194	80.691	0.121	0.123	137552.311	137552.311
4/ 11	6.734	0.149	79.285	80.942	0.091	0.251	137552.311	137552.311
4/ 12	7.555	0.132	79.286	80.961	0.001	0.019	137552.311	137552.311
4/ 13	7.637	0.131	79.382	80.961	0.096	0.000	137552.311	137552.311
4/ 14	8.325	0.120	79.431	80.962	0.049	0.001	137552.311	137552.311
4/ 15	8.606	0.116	79.442	80.962	0.010	0.000	137552.311	137552.311
4/ 16	8.859	0.113	79.448	80.969	0.007	0.008	137552.311	137552.311
4/ 17	8.879	0.113	79.450	80.971	0.001	0.002	137552.311	137552.311
4/ 18	9.003	0.111	79.450	80.971	0.000	0.000	137552.311	137552.311
4/ 19	9.088	0.110	79.450	80.976	0.000	0.005	137552.311	137552.311
4/ 20	9.531	0.105	79.966	80.980	0.516	0.004	137552.311	137552.311
4/ 21	10.321	0.097	80.089	80.980	0.123	0.000	137552.311	137552.311
4/ 22	10.760	0.093	89.121	80.981	9.032	0.000	137552.311	137552.311
4/ 23	11.538	0.087	89.218	80.991	0.098	0.010	137552.311	137552.311
4/ 24	11.552	0.087	89.747	81.000	0.529	0.009	137552.311	137552.311
4/ 25	11.578	0.086	90.487	81.280	0.740	0.280	137552.311	137552.311
4/ 26	11.826	0.085	95.443	81.547	4.956	0.266	137552.311	137552.311
4/ 27	12.336	0.081	95.491	81.581	0.048	0.035	137552.311	137552.311
4/ 28	12.497	0.080	95.756	83.687	0.264	2.106	137552.311	137552.311
4/ 29	13.155	0.076	98.080	83.956	2.324	0.269	137552.311	137552.311
4/ 30	13.292	0.075	98.081	84.593	0.001	0.637	137552.311	137552.311
4/ 31	13.584	0.074	98.115	96.977	0.034	12.384	137552.311	137552.311
4/ 32	14.406	0.069	98.122	97.666	0.007	0.689	137552.311	137552.311
4/ 33	14.822	0.067	98.134	97.769	0.013	0.103	137552.311	137552.311
4/ 34	15.165	0.066	98.233	97.776	0.098	0.007	137552.311	137552.311
4/ 35	15.228	0.066	98.237	97.790	0.004	0.015	137552.311	137552.311

Se aprecia que la participación de masas es mayor al 90%.

- ET2

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Total mass UX (kg)	Total mass UY (kg)
4/ 1	1.501	0.666	0.255	77.818	0.255	77.818	450637.114	450637.114
4/ 2	1.526	0.655	77.646	78.187	77.391	0.369	450637.114	450637.114
4/ 3	1.808	0.553	78.763	78.969	1.117	0.783	450637.114	450637.114
4/ 4	4.553	0.220	89.124	79.232	10.361	0.263	450637.114	450637.114
4/ 5	4.823	0.207	89.377	92.149	0.254	12.916	450637.114	450637.114
4/ 6	5.241	0.191	89.377	92.175	0.000	0.026	450637.114	450637.114
4/ 7	5.745	0.174	89.393	92.175	0.016	0.000	450637.114	450637.114
4/ 8	5.896	0.170	89.403	92.176	0.010	0.001	450637.114	450637.114
4/ 9	5.934	0.169	89.409	92.176	0.006	0.000	450637.114	450637.114
4/ 10	6.010	0.166	89.436	92.178	0.027	0.002	450637.114	450637.114
4/ 11	6.108	0.164	89.437	92.198	0.001	0.021	450637.114	450637.114
4/ 12	6.369	0.157	90.869	92.199	1.431	0.001	450637.114	450637.114
4/ 13	6.399	0.156	90.969	92.201	0.100	0.002	450637.114	450637.114
4/ 14	6.424	0.156	91.172	92.212	0.203	0.010	450637.114	450637.114
4/ 15	6.557	0.152	91.172	92.216	0.000	0.004	450637.114	450637.114

Se aprecia que la participación de masas es mayor al 90%.

- ET3

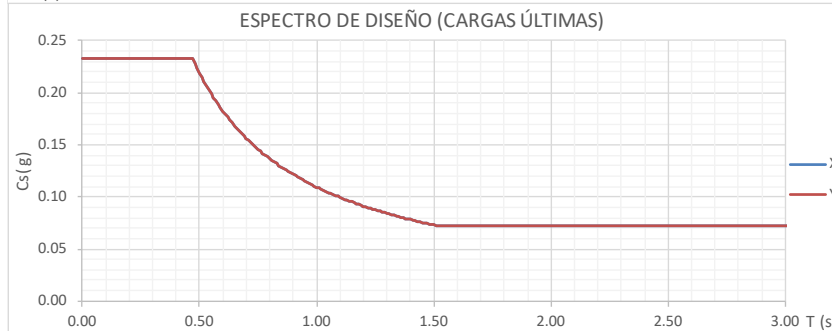
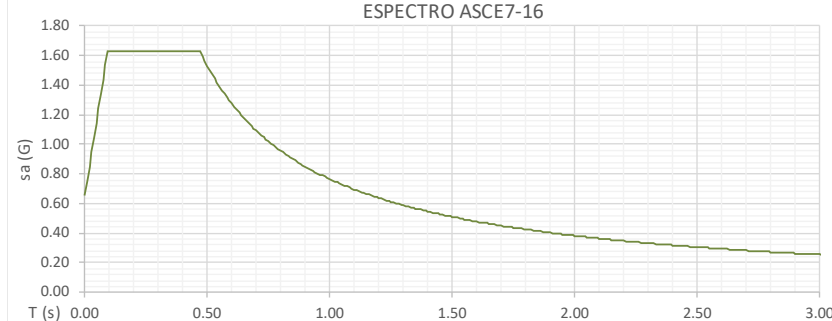
Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Total mass UX (kg)	Total mass UY (kg)
4/ 1	1.238	0.808	75.030	0.048	75.030	0.048	566987.956	566987.956
4/ 2	1.294	0.773	75.114	74.730	0.084	74.682	566987.956	566987.956
4/ 3	1.719	0.582	75.574	75.659	0.460	0.929	566987.956	566987.956
4/ 4	3.880	0.258	75.718	88.164	0.144	12.505	566987.956	566987.956
4/ 5	3.920	0.255	88.900	88.297	13.182	0.134	566987.956	566987.956
4/ 6	4.638	0.216	88.903	88.301	0.003	0.004	566987.956	566987.956
4/ 7	4.665	0.214	89.010	88.334	0.107	0.033	566987.956	566987.956
4/ 8	4.866	0.205	89.010	88.334	0.000	0.001	566987.956	566987.956
4/ 9	5.113	0.196	89.011	88.346	0.000	0.011	566987.956	566987.956
4/ 10	5.670	0.176	89.011	88.346	0.000	0.000	566987.956	566987.956
4/ 11	5.788	0.173	89.011	88.346	0.001	0.000	566987.956	566987.956
4/ 12	5.851	0.171	89.014	88.346	0.003	0.000	566987.956	566987.956
4/ 13	5.990	0.167	89.019	88.347	0.005	0.001	566987.956	566987.956
4/ 14	6.007	0.166	89.025	88.347	0.006	0.000	566987.956	566987.956
4/ 15	6.053	0.165	89.026	88.348	0.001	0.000	566987.956	566987.956
4/ 16	6.084	0.164	89.029	88.348	0.003	0.000	566987.956	566987.956
4/ 17	6.176	0.162	89.032	88.348	0.003	0.000	566987.956	566987.956
4/ 18	6.224	0.161	89.032	88.386	0.000	0.038	566987.956	566987.956
4/ 19	6.239	0.160	89.033	88.458	0.001	0.072	566987.956	566987.956
4/ 20	6.317	0.158	89.034	88.468	0.001	0.010	566987.956	566987.956
4/ 21	6.325	0.158	89.038	88.469	0.004	0.002	566987.956	566987.956
4/ 22	6.347	0.158	89.041	88.471	0.003	0.002	566987.956	566987.956
4/ 23	6.353	0.157	89.050	88.472	0.009	0.001	566987.956	566987.956
4/ 24	6.373	0.157	89.051	88.479	0.001	0.007	566987.956	566987.956
4/ 25	6.417	0.156	89.056	88.507	0.005	0.027	566987.956	566987.956
4/ 26	6.460	0.155	89.056	88.639	0.000	0.132	566987.956	566987.956
4/ 27	6.467	0.155	89.056	88.918	0.000	0.279	566987.956	566987.956
4/ 28	6.671	0.150	89.088	88.957	0.032	0.038	566987.956	566987.956
4/ 29	6.923	0.144	89.212	88.974	0.124	0.018	566987.956	566987.956
4/ 30	7.146	0.140	89.628	88.995	0.416	0.021	566987.956	566987.956
4/ 31	7.189	0.139	91.413	89.047	1.786	0.052	566987.956	566987.956
4/ 32	7.610	0.131	94.272	90.431	2.858	1.384	566987.956	566987.956
4/ 33	7.690	0.130	95.791	94.406	1.520	3.975	566987.956	566987.956
4/ 34	8.055	0.124	95.801	94.495	0.009	0.089	566987.956	566987.956
4/ 35	8.113	0.123	95.814	95.171	0.014	0.677	566987.956	566987.956

Se aprecia que la participación de masas es mayor al 90%.

6.2. Carga sísmica de diseño ASCE7-16.

- ET1.

ASCE7 - 16 ESPECTRO			
$S_S = 2.04$	$F_a = 1.20$	Site Class →	C
$S_1 = 0.82$	$F_v = 1.40$	$T_L =$	4 s
$S_{MS} = 2.448$	$S_{DS} = 1.632$	$T_o =$	0.094
$S_{M1} = 1.148$	$S_{D1} = 0.765$	$T_s =$	0.469



CÁLCULO CORTE BASAL DE DISEÑO			
SENTIDO X		SENTIDO Y	
$hn = 6.80 \text{ m}$	$C_u = 1.40$	$hn = 6.80 \text{ m}$	$C_u = 1.40$
Steel moment resisting frame		Steel moment resisting frame	
$C_t = 0.0724$	$x = 0.8$	$C_t = 0.0724$	$x = 0.8$
$T_a = 0.336 \text{ s}$	$T_a = C_t h_n^x$	$T_a = 0.336 \text{ s}$	$T_a = C_t h_n^x$
$T_{ASCE7} = 0.470 \text{ s}$	→ $C_u \times T_a$	$T_{ASCE7} = 0.470 \text{ s}$	→ $C_u \times T_a$
$T_{MODAL} = 0.410$		$T_{MODAL} = 0.330$	
T = 0.41 s		T = 0.33 s	
$I_e = 1$		Ocupación Normal R.C → I & II	
$S_D = 1.632$	$R = 7$	$S_D = 1.632$	$R = 7$
$S_a/(R \cdot I_e) = 0.233$		$S_a/(R \cdot I_e) = 0.233$	
$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.233$ (12.8-2) (Rango de aceleración)	$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.233$ (12.8-2) (Rango de aceleración)
$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.267$ (12.8-3) (Rango de velocidad)	$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.331$ (12.8-3) (Rango de velocidad)
$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = \text{NA}$ (12.8-4) (Rango desplazamiento)	$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = \text{NA}$ (12.8-4) (Rango desplazamiento)
$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.072$ (12.8-5)	$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.072$ (12.8-5)
$C_s = 0.5 S_1 / (R / I_e)$	$C_{SMIN} = 0.059$ (12.8-5)	$C_s = 0.5 S_1 / (R / I_e)$	$C_{SMIN} = 0.059$ (12.8-5)
Para $S_1 \geq 0.6$		Para $S_1 \geq 0.6$	
Cs = 0.233 g		Cs = 0.233 g	

Table 11.4-1 Short-Period Site Coefficient, F_a						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _a) Spectral Response Acceleration Parameter at Short Period						
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1$	$S_s = 1.25$	$S_s \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

Note: Use straight-line interpolation for intermediate values of S_s .

Table 11.4-2 Long-Period Site Coefficient, F_v						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _a) Spectral Response Acceleration Parameter at 1-s Period						
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

Note: Use straight-line interpolation for intermediate values of S_1 .

Table 12.8-2 Values of Approximate Period Parameters C_t and x		
Structure type	C_t	x
Steel moment resisting frame	0.0724	0.8
Concrete moment resisting frame	0.0466	0.9
Steel eccentrically braced frames	0.0731	0.75
Steel buckling-restrained braced frames	0.0731	0.75
All other structural systems	0.0488	0.75

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficiente C_u
≥ 0.40	1.4
0.30	1.4
0.20	1.5
0.15	1.6
≤ 0.10	1.7

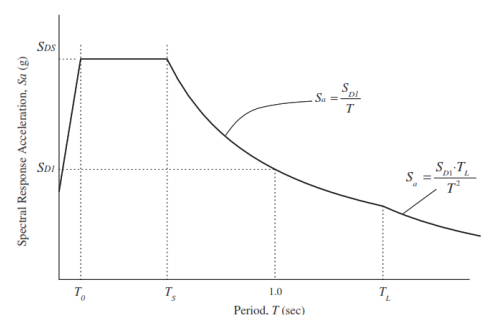
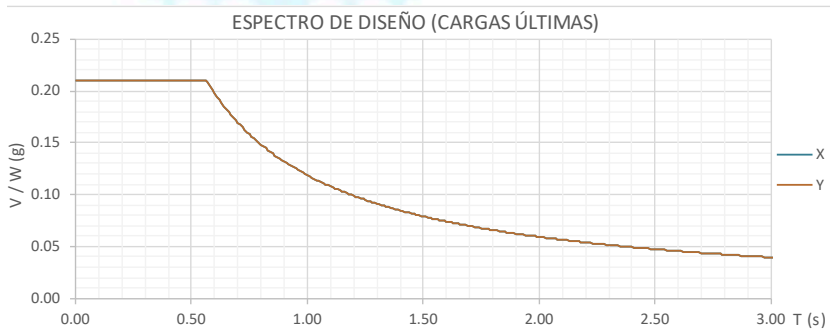
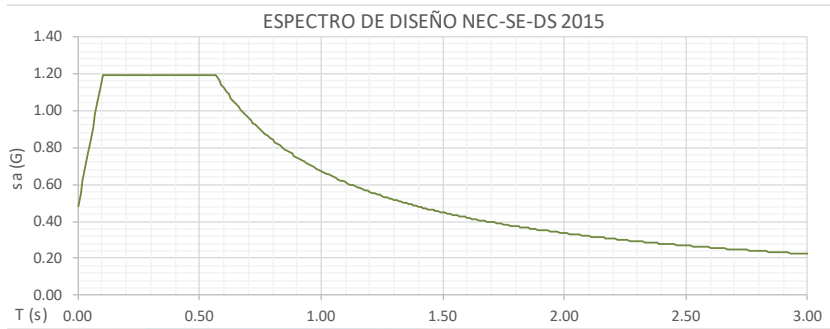


FIGURE 11.4-1 Design Response Spectrum

NEC-SE-DS ESPECTRO					
Z = 0.4	V	F _a = 1.20	η = 2.48	Provincias de la Sierra	
Tipo Perfil	C	F _d = 1.11	r = 1.0		
Suelo		F _s = 1.11			
T ₀ = 0.103 s	T _C = 0.565 s	T _L = 2.664 s	Z.F _a = 0.480 g	ηZ.F _a = 1.190 g	



CÁLCULO CORTE BASAL DE DISEÑO					
SENTIDO X			SENTIDO Y		
hn	6.80 m		hn	6.80 m	
Estructura de ACERO SIN arriostramientos			Estructura de ACERO SIN arriostramientos		
C _t	0.072	α = 0.80	C _t	0.072	x = 0.80
T _a	0.334 s	T _a = C _t h _n ^x	T _a	0.334 s	T _a = C _t h _n ^x
T _{NEC-SE-DS}	0.434 s	→ 1.30 x T _a	T _{NEC-SE-DS}	0.434 s	→ 1.30 x T _a
T _{MODAL}	0.410		T _{MODAL}	0.330	
T	0.41 s		T	0.33 s	
I _e = 1 Ocupación Normal					
S _a	1.190	R = 7	S _a	1.190	R = 7
φ _p	0.9	φ _E = 0.9	φ _p	0.9	φ _E = 0.9
$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$			$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$		
V	0.210 W		V	0.210 W	
0.85 V	= 0.178 W		0.85 V	= 0.178 W	
Estructuras irregulares			Estructuras irregulares		

TABLA 3: Tipo de suelo y factores de sitio F _a						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.40	1.30	1.25	1.23	1.20	1.12
D	1.60	1.40	1.30	1.25	1.20	1.12
E	1.80	1.40	1.25	1.10	1.00	0.85
F	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.5.4

TABLA 3: Tipo de suelo y factores de sitio F _d						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.36	1.28	1.19	1.15	1.11	1.06
D	1.62	1.45	1.36	1.28	1.19	1.11
E	2.10	1.75	1.70	1.65	1.60	1.50
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

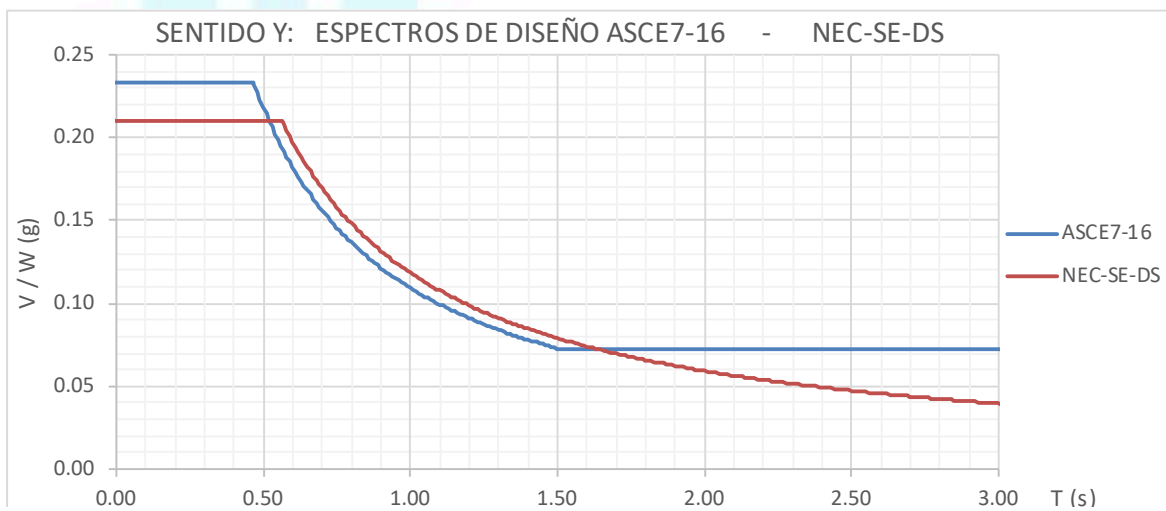
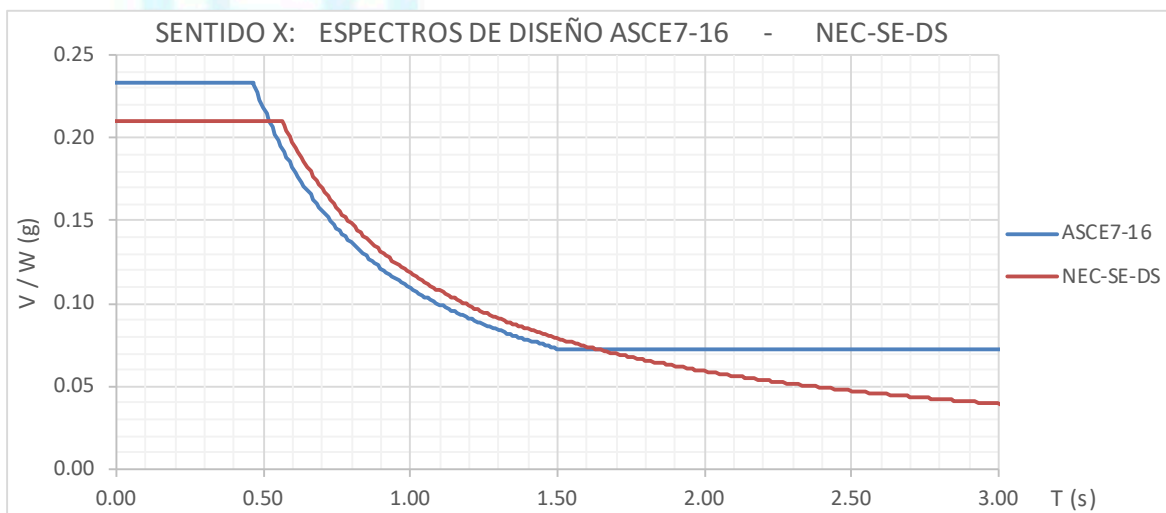
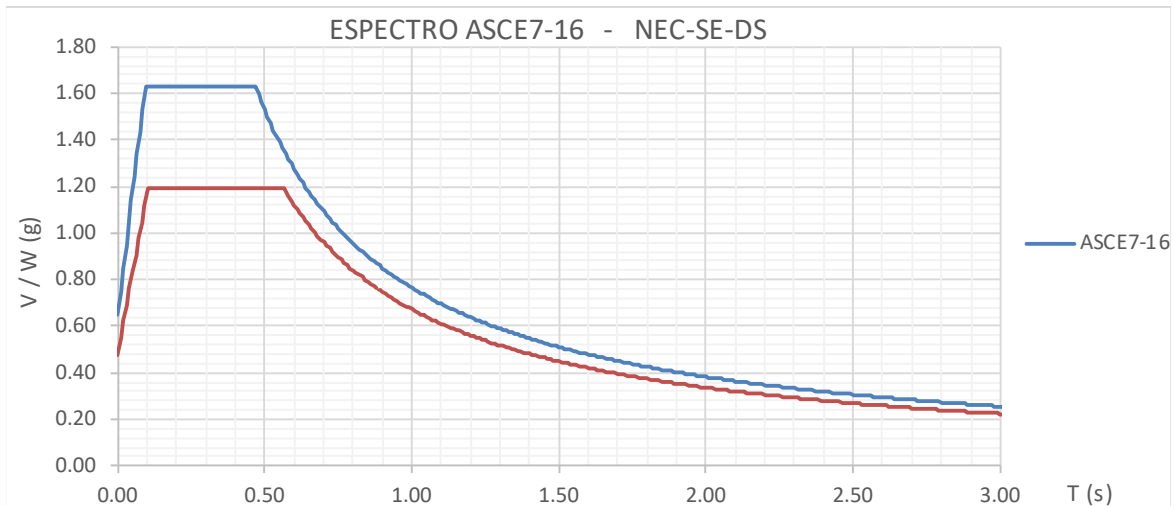
TABLA 3: Tipo de suelo y factores de sitio F _s						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.75	0.75	0.75	0.75	0.75	0.75
B	0.75	0.75	0.75	0.75	0.75	0.75
C	0.85	0.94	1.02	1.06	1.11	1.23
D	1.02	1.06	1.11	1.19	1.28	1.40
E	1.50	1.60	1.70	1.80	1.90	2.00
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

Razón entre S _a y PGA para periodo de retorno seleccionado	Factor usado espectro diseño elástico
DESCRIPCIÓN	η
DESCRIPCIÓN	r
Provincia de Galápagos	2.48
Provincias de la Costa (-E)	1.80
Provincia de Esmeraldas	2.48
Provincias de la Sierra	2.48
Provincias del Oriente	2.60

TIPO DE ESTRUCTURA	C _t	α
Estructura de ACERO SIN arriostramientos	0.072	0.80
Estructura de ACERO CON arriostramientos	0.073	0.75
Pórtico E.-HORMIGÓN A.-SIN Muros E./Diagonales R.	0.055	0.90
Pórtico E.-HORMIGÓN A.-CON Muros E./Diagonales R.	0.055	0.75
Estructuras basadas en MUROS E./MAMPOSTERÍA E.	0.055	0.75

COMPARACIÓN ESPECTROS DE DISEÑO: ASCE7-16 - NEC-SE-DS



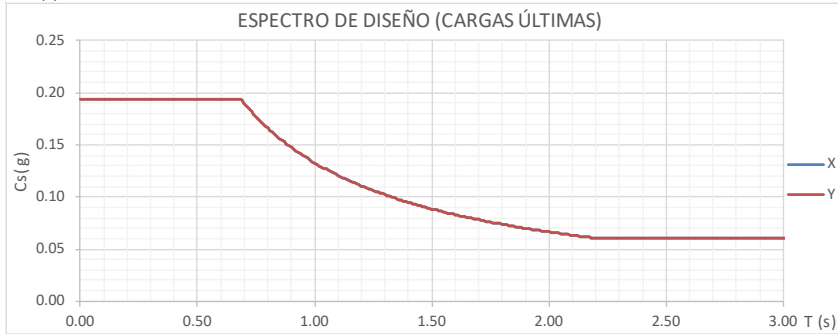
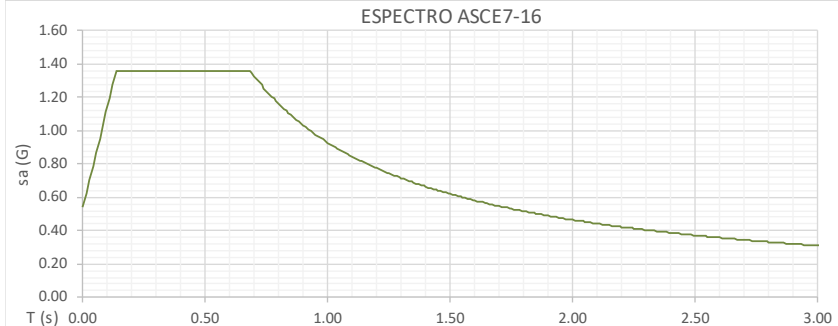
ASCE7-16 NEC-SE-DS

0.233 > 0.178

→ Se procede a usar el espectro ASCE7-16.

• ET2.

ASCE7 - 16 ESPECTRO		
$S_S = 2.04$	$F_a = 1.00$	Site Class \rightarrow D
$S_1 = 0.82$	$F_v = 1.70$	$T_L = 4$ s
$S_{MS} = 2.04$	$S_{DS} = 1.36$	$T_0 = 0.137$
$S_{M1} = 1.394$	$S_{D1} = 0.929$	$T_s = 0.683$



CÁLCULO CORTE BASAL DE DISEÑO			
SENTIDO X		SENTIDO Y	
$h_n = 10.20$ m	$C_u = 1.40$	$h_n = 10.20$ m	$C_u = 1.40$
Steel moment resisting frame		Steel moment resisting frame	
$C_t = 0.0724$	$\chi = 0.8$	$C_t = 0.0724$	$\chi = 0.8$
$T_a = 0.464$ s	$T_a = C_t h_n^x$	$T_a = 0.464$ s	$T_a = C_t h_n^x$
$T_{ASCE7} = 0.650$ s	$\rightarrow C_u \times T_a$	$T_{ASCE7} = 0.650$ s	$\rightarrow C_u \times T_a$
$T_{MODAL} = 0.660$ s		$T_{MODAL} = 0.670$ s	
T = 0.65 s		T = 0.65 s	
Ocupación Normal R.C \rightarrow I & II			
$S_a = 1.360$	$R = 7$	$S_a = 1.360$	$R = 7$
$S_a/(R.I) = 0.194$		$S_a/(R.I) = 0.194$	
$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.194$ (12.8-2) (Rango de aceleración)	$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.194$ (12.8-2) (Rango de aceleración)
$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.204$ (12.8-3) (Rango de velocidad)	$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.204$ (12.8-3) (Rango de velocidad)
$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = NA$ (12.8-4) (Rango desplazamiento)	$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = NA$ (12.8-4) (Rango desplazamiento)
$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.060$ (12.8-5)	$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.060$ (12.8-5)
$C_s = 0.5 S_1 / (R/I_e)$	$C_{SMIN} = 0.059$ (12.8-5)	$C_s = 0.5 S_1 / (R/I_e)$	$C_{SMIN} = 0.059$ (12.8-5)
Para $S_1 \geq 0.6$		Para $S_1 \geq 0.6$	
Cs = 0.194 g		Cs = 0.194 g	

Table 11.4-1 Short-Period Site Coefficient, F_a						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at Short Period						
Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

Note: Use straight-line interpolation for intermediate values of S_S .

Table 11.4-2 Long-Period Site Coefficient, F_v						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at 1-s Period						
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

a \rightarrow *Also, see requirements for site-specific ground motions in Section 11.4.8.
Note: Use straight-line interpolation for intermediate values of S_1 .

Table 12.8-2 Values of Approximate Period Parameters C_t and χ		
Structure type	C_t	χ
Steel moment resisting frame	0.0724	0.8
Concrete moment resisting frame	0.0466	0.9
Steel eccentrically braced frames	0.0731	0.75
Steel buckling-restrained braced frames	0.0731	0.75
All other structural systems	0.0488	0.75

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficiente C_u
≥ 0.40	1.4
0.30	1.4
0.20	1.5
0.15	1.6
≤ 0.10	1.7

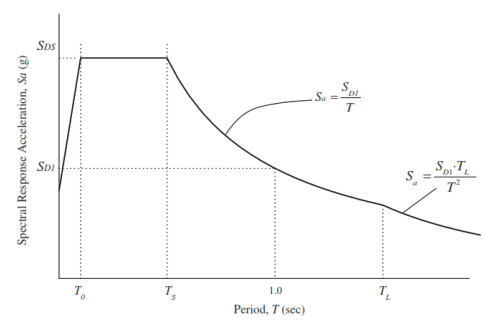
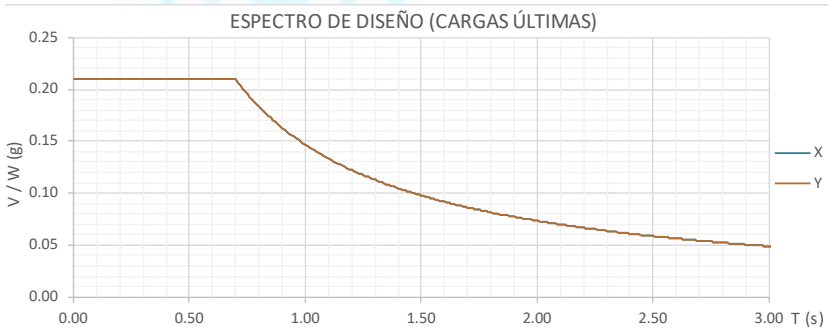
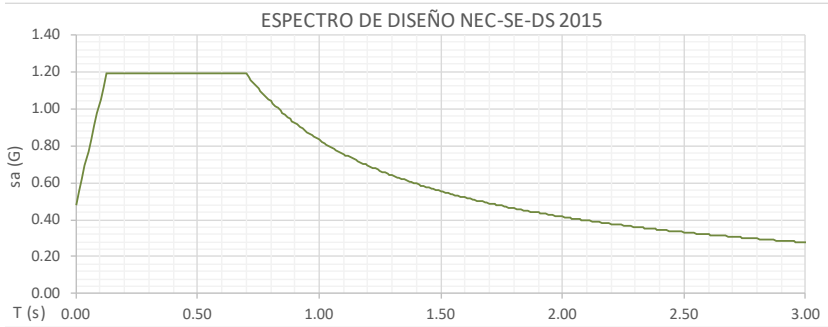


FIGURE 11.4-1 Design Response Spectrum

NEC-SE-DS ESPECTRO					
Z = 0.4	V	F _a = 1.20	η = 2.48	Provincias de la Sierra	
Tipo Perfil	D	F _d = 1.19	r = 1.0		
Suelo		F _s = 1.28			
T ₀ = 0.127 s	T _C = 0.698 s	T _L = 2.856 s	Z.F _a = 0.480 g	ηZ.F _a = 1.190 g	



CÁLCULO CORTE BASAL DE DISEÑO					
SENTIDO X			SENTIDO Y		
hn	10.20 m		hn	10.20 m	
Estructura de ACERO SIN arriostramientos			Estructura de ACERO SIN arriostramientos		
C _t	0.072	α = 0.80	C _t	0.072	x = 0.80
T _a	0.462 s	T _a = C _t h _n ^x	T _a	0.462 s	T _a = C _t h _n ^x
T _{NEC-SE-DS}	0.600 s	→ 1.30 x T _a	T _{NEC-SE-DS}	0.600 s	→ 1.30 x T _a
T _{MODAL}	0.660		T _{MODAL}	0.670	
T = 0.60 s			T = 0.60 s		
I _e = 1 Ocupación Normal					
S _a	1.190	R = 7	S _a	1.190	R = 7
φ _p	0.9	φ _E = 0.9	φ _p	0.9	φ _E = 0.9
$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$			$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$		
V = 0.210 W			V = 0.210 W		
0.85 V = 0.178 W			0.85 V = 0.178 W		
Estructuras irregulares			Estructuras irregulares		

TABLA 3: Tipo de suelo y factores de sitio F _a						
Tipo	Zona sísmica y factor Z					
Perfil	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.40	1.30	1.25	1.23	1.20	1.12
D	1.60	1.40	1.30	1.25	1.20	1.12
E	1.80	1.40	1.25	1.10	1.00	0.85
F	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.5.4

TABLA 3: Tipo de suelo y factores de sitio F _d						
Tipo	Zona sísmica y factor Z					
Perfil	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.36	1.28	1.19	1.15	1.11	1.06
D	1.62	1.45	1.36	1.28	1.19	1.11
E	2.10	1.75	1.70	1.65	1.60	1.50
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

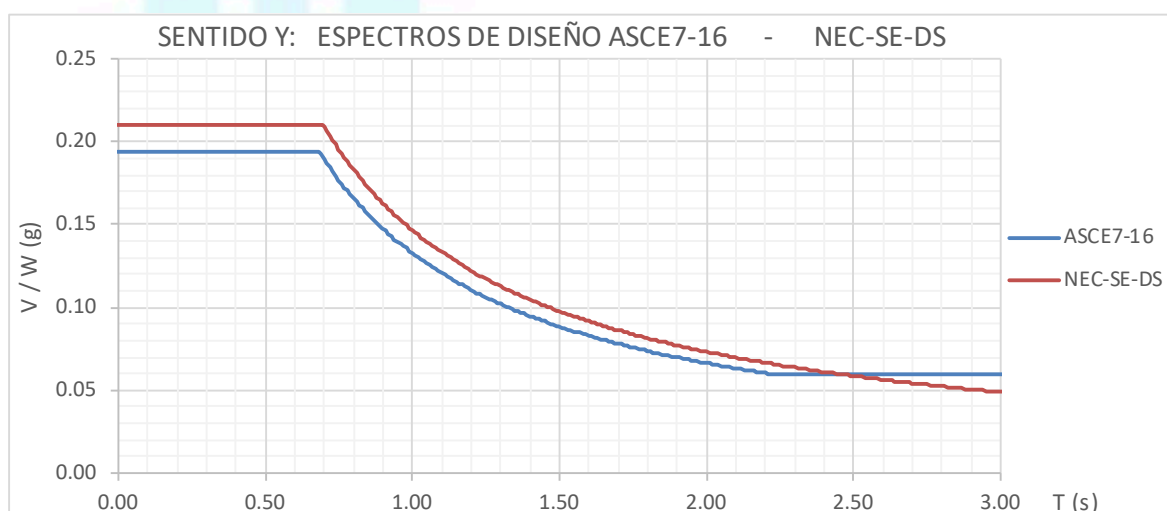
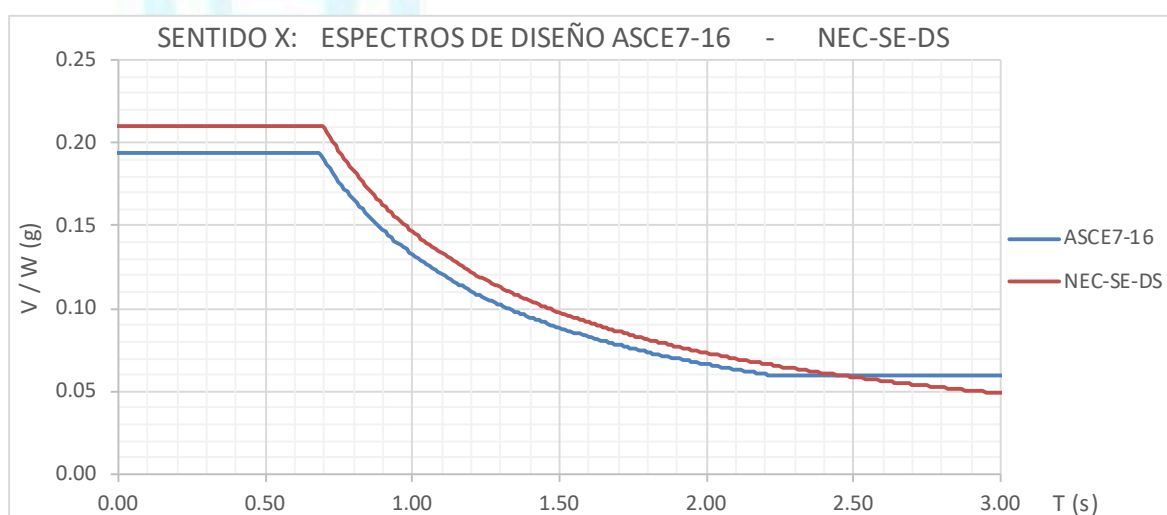
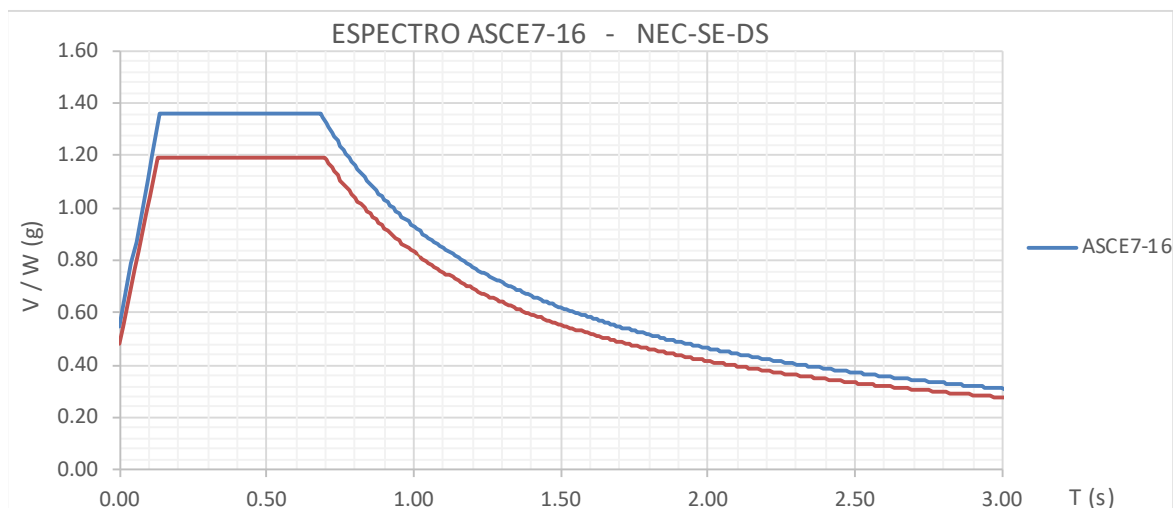
TABLA 3: Tipo de suelo y factores de sitio F _s						
Tipo	Zona sísmica y factor Z					
Perfil	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.75	0.75	0.75	0.75	0.75	0.75
B	0.75	0.75	0.75	0.75	0.75	0.75
C	0.85	0.94	1.02	1.06	1.11	1.23
D	1.02	1.06	1.11	1.19	1.28	1.40
E	1.50	1.60	1.70	1.80	1.90	2.00
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

Razón entre S _a y PGA para periodo de retorno seleccionado		Factor usado espectro diseño elástico	
DESCRIPCIÓN	η	DESCRIPCIÓN	r
Provincia de Galápagos	2.48	Suelos A-B-C-D	1
Provincias de la Costa (-E)	1.80	Suelo tipo E	1.5
Provincia de Esmeraldas	2.48		
Provincias de la Sierra	2.48		
Provincias del Oriente	2.60		

TIPO DE ESTRUCTURA	C _t	α
Estructura de ACERO SIN arriostramientos	0.072	0.80
Estructura de ACERO CON arriostramientos	0.073	0.75
Pórtico E.-HORMIGÓN A.-SIN Muros E./Diagonales R.	0.055	0.90
Pórtico E.-HORMIGÓN A.-CON Muros E./Diagonales R.	0.055	0.75
Estructuras basadas en MUROSE./MAMPOSTERÍA E.	0.055	0.75

COMPARACIÓN ESPECTROS DE DISEÑO: ASCE7-16 - NEC-SE-DS



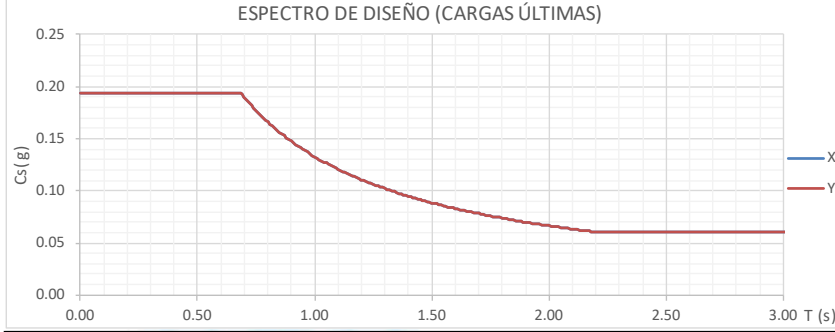
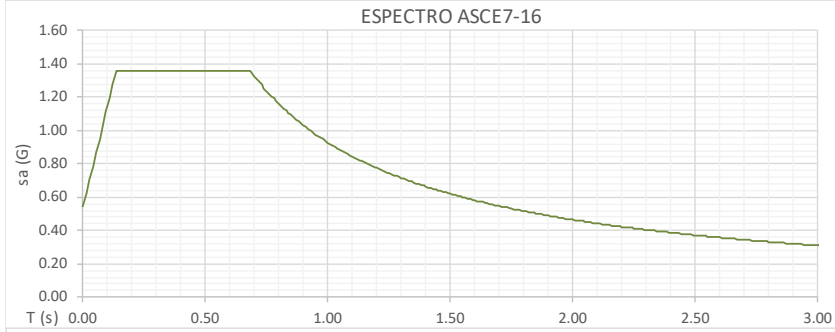
ASCE7-16 NEC-SE-DS

0.194 > 0.178

→ Se procede a usar el espectro ASCE7-16.

• ET3.

ASCE7 - 16 ESPECTRO			
$S_s = 2.04$	$F_a = 1.00$	Site Class →	D
$S_1 = 0.82$	$F_v = 1.70$	$T_L =$	4 s
$S_{MS} = 2.04$	$S_{DS} = 1.36$	$T_0 =$	0.137
$S_{M1} = 1.394$	$S_{D1} = 0.929$	$T_s =$	0.683



CÁLCULO CORTE BASAL DE DISEÑO			
SENTIDO X		SENTIDO Y	
hn= 13.60 m	$C_u = 1.40$	hn= 13.60 m	$C_u = 1.40$
Steel moment resisting frame		Steel moment resisting frame	
$C_t = 0.0724$	x= 0.8	$C_t = 0.0724$	x= 0.8
$T_a = 0.584$ s	$T_a = C_t h_n^x$	$T_a = 0.584$ s	$T_a = C_t h_n^x$
$T_{ASCE7} = 0.818$ s	→ $C_u \times T_a$	$T_{ASCE7} = 0.818$ s	→ $C_u \times T_a$
$T_{MODAL} = 0.810$ s		$T_{MODAL} = 0.770$ s	
T = 0.81 s		T = 0.77 s	
Ocupación Normal R.C → I & II			
$S_a = 1.147$	R = 7	$S_a = 1.207$	R = 7
$S_a/(R.I) = 0.164$		$S_a/(R.I) = 0.172$	
$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.194$ (12.8-2) (Rango de aceleración)	$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	$C_s = 0.194$ (12.8-2) (Rango de aceleración)
$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.164$ (12.8-3) (Rango de velocidad)	$T \leq T_L$ $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = 0.172$ (12.8-3) (Rango de velocidad)
$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = NA$ (12.8-4) (Rango desplazamiento)	$T > T_L$ $C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)}$	$C_{SMAX} = NA$ (12.8-4) (Rango desplazamiento)
$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.060$ (12.8-5)	$C_s = 0.044 S_{DS} I_e \geq 0.01$	$C_{SMIN} = 0.060$ (12.8-5)
$C_s = 0.5 S_1 / (R/I_e)$	$C_{SMIN} = 0.059$ (12.8-5)	$C_s = 0.5 S_1 / (R/I_e)$	$C_{SMIN} = 0.059$ (12.8-5)
Para $S_1 \geq 0.6$		Para $S_1 \geq 0.6$	
Cs = 0.164 g		Cs = 0.172 g	

Table 11.4-1 Short-Period Site Coefficient, F_a						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at Short Period						
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1$	$S_s = 1.25$	$S_s \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

Note: Use straight-line interpolation for intermediate values of S_s .

Table 11.4-2 Long-Period Site Coefficient, F_v						
Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at 1-s Period						
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8
F	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8	11.4.8

a → "Also, see requirements for site-specific ground motions in Section 11.4.8.
Note: Use straight-line interpolation for intermediate values of S_1 .

Table 12.8-2 Values of Approximate Period Parameters C_t and x		
Structure type	C_t	x
Steel moment resisting frame	0.0724	0.8
Concrete moment resisting frame	0.0466	0.9
Steel eccentrically braced frames	0.0731	0.75
Steel buckling-restrained braced frames	0.0731	0.75
All other structural systems	0.0488	0.75

Design Spectral Response Acceleration Parameter at 1s, S_{D1}	Coefficiente C_u
≥ 0.40	1.4
0.30	1.4
0.20	1.5
0.15	1.6
≤ 0.10	1.7

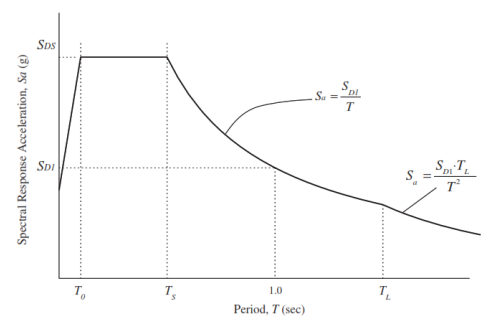
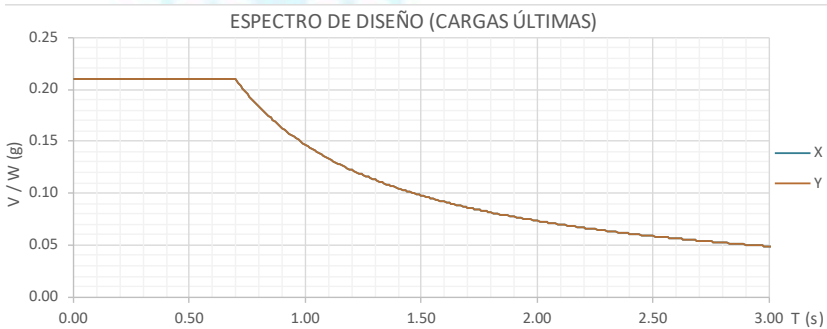
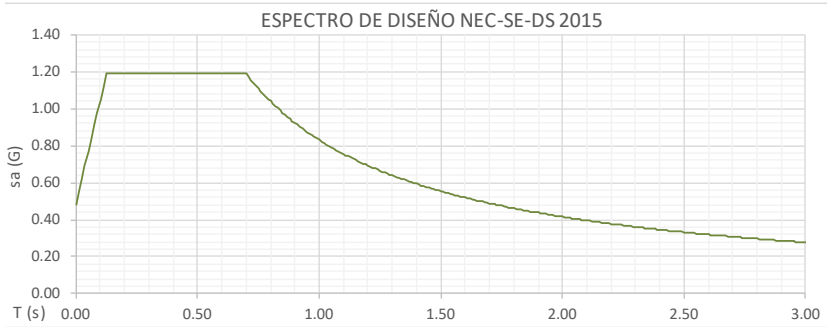


FIGURE 11.4-1 Design Response Spectrum

NEC-SE-DS ESPECTRO					
Z = 0.4	V	F _a = 1.20	η = 2.48	Provincias de la Sierra	
Tipo Perfil	D	F _d = 1.19	r = 1.0		
Suelo		F _s = 1.28			
T ₀ = 0.127 s	T _C = 0.698 s	T _L = 2.856 s	Z.F _a = 0.480 g	ηZ.F _a = 1.190 g	



CÁLCULO CORTE BASAL DE DISEÑO					
SENTIDO X			SENTIDO Y		
hn	13.60 m		hn	13.60 m	
Estructura de ACERO SIN arriostramientos			Estructura de ACERO SIN arriostramientos		
C _t	0.072	α = 0.80	C _t	0.072	x = 0.80
T _a	0.581 s	T _a = C _t h _n ^x	T _a	0.581 s	T _a = C _t h _n ^x
T _{NEC-SE-DS}	0.755 s	→ 1.30 x T _a	T _{NEC-SE-DS}	0.755 s	→ 1.30 x T _a
T _{MODAL}	0.810		T _{MODAL}	0.770	
T	0.75 s		T	0.75 s	
I _e = 1 Ocupación Normal					
S _a	1.108	R = 7	S _a	1.108	R = 7
φ _p	0.9	φ _E = 0.9	φ _p	0.9	φ _E = 0.9
$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$			$V = \frac{I \cdot S_a(T_a)}{R \cdot \phi_p \cdot \phi_E} \cdot W$		
V	0.195 W		V	0.195 W	
0.85 V	0.166 W		0.85 V	0.166 W	
Estructuras irregulares			Estructuras irregulares		

TABLA 3: Tipo de suelo y factores de sitio F _a						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.40	1.30	1.25	1.23	1.20	1.12
D	1.60	1.40	1.30	1.25	1.20	1.12
E	1.80	1.40	1.25	1.10	1.00	0.85
F	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4	10.5.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.5.4

TABLA 3: Tipo de suelo y factores de sitio F _d						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.90	0.90	0.90	0.90	0.90	0.90
B	1.00	1.00	1.00	1.00	1.00	1.00
C	1.36	1.28	1.19	1.15	1.11	1.06
D	1.62	1.45	1.36	1.28	1.19	1.11
E	2.10	1.75	1.70	1.65	1.60	1.50
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

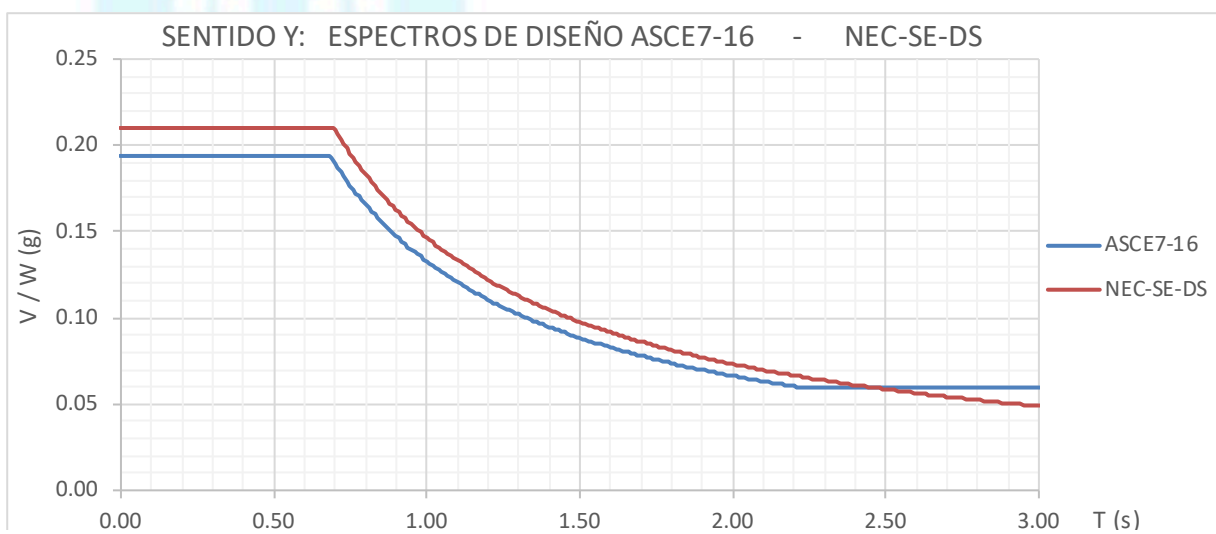
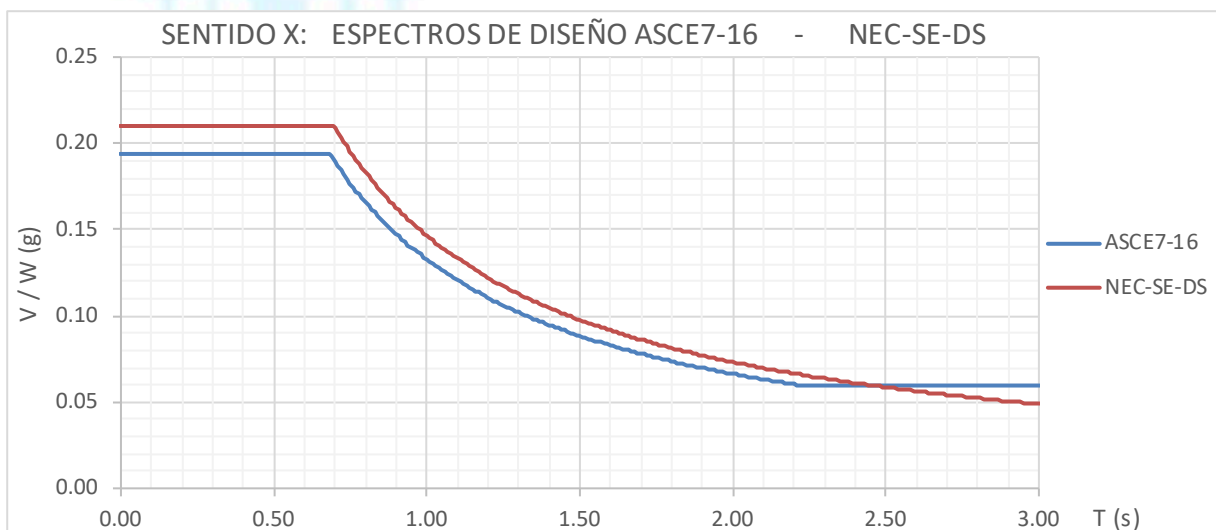
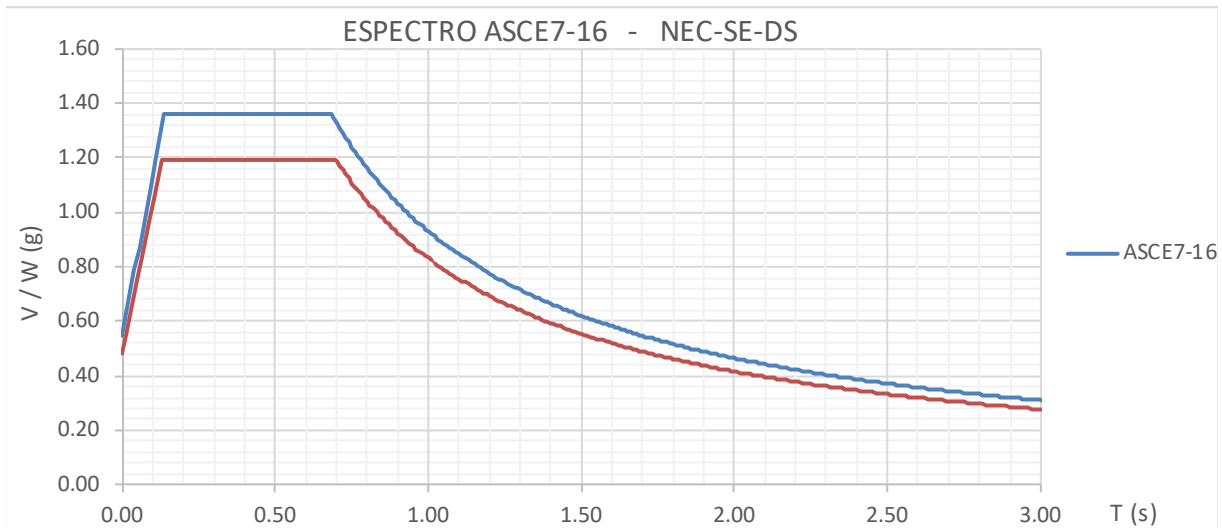
TABLA 3: Tipo de suelo y factores de sitio F _s						
Tipo Perfil	Zona sísmica y factor Z					
	I	II	III	IV	V	VI
Subsuelo	=	=	=	=	=	≥
	0.15	0.25	0.3	0.35	0.4	0.5
A	0.75	0.75	0.75	0.75	0.75	0.75
B	0.75	0.75	0.75	0.75	0.75	0.75
C	0.85	0.94	1.02	1.06	1.11	1.23
D	1.02	1.06	1.11	1.19	1.28	1.40
E	1.50	1.60	1.70	1.80	1.90	2.00
F	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4	10.6.4

Ver tabla 2: Clasificación de perfiles de suelo y sección 10.6.4

Razón entre S _a y PGA para periodo de retorno seleccionado	Factor usado espectro diseño elástico		
	DESCRIPCIÓN	η	r
Provincia de Galápagos	2.48	Suelos A-B-C-D	1
Provincias de la Costa (-E)	1.80	Suelo tipo E	1.5
Provincia de Esmeraldas	2.48		
Provincias de la Sierra	2.48		
Provincias del Oriente	2.60		

TIPO DE ESTRUCTURA	C _t	α
Estructura de ACERO SIN arriostramientos	0.072	0.80
Estructura de ACERO CON arriostramientos	0.073	0.75
Pórtico E.-HORMIGÓN A.-SIN Muros E./Diagonales R.	0.055	0.90
Pórtico E.-HORMIGÓN A.-CON Muros E./Diagonales R.	0.055	0.75
Estructuras basadas en MUROSE./MAMPOSTERÍA E.	0.055	0.75

COMPARACIÓN ESPECTROS DE DISEÑO: ASCE7-16 - NEC-SE-DS



ASCE7-16 NEC-SE-DS

0.172 > 0.166

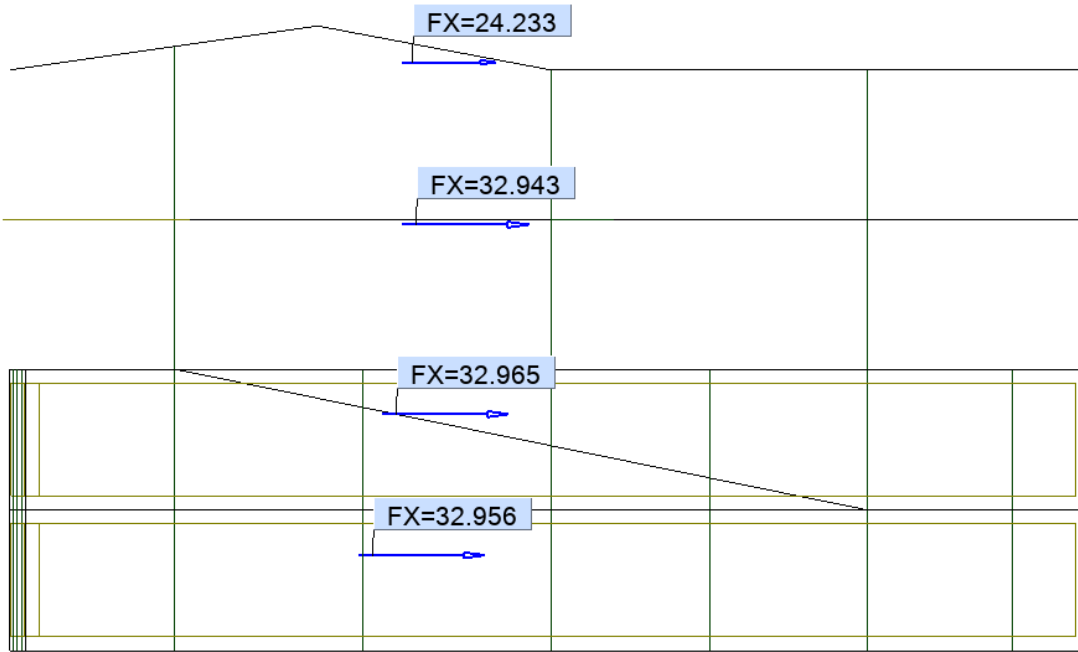
→ Se procede a usar el espectro ASCE7-16.

6.3. Fuerzas horizontales sísmicas (Análisis modal espectral).

6.3.1. Sismo X, introducido en el modelo matemático (Análisis modal espectral).

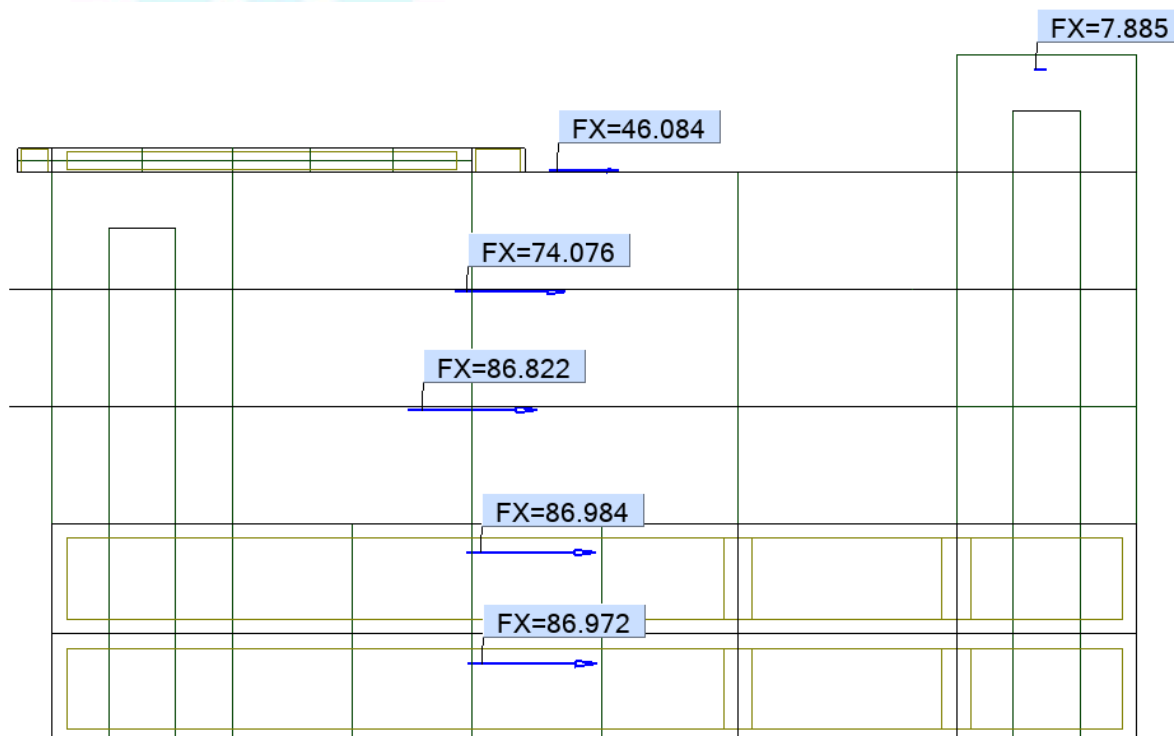
- ET1.

$W=143 \text{ t}$ $V= 0.233 \times 143 \rightarrow V=33.32 \text{ t}$



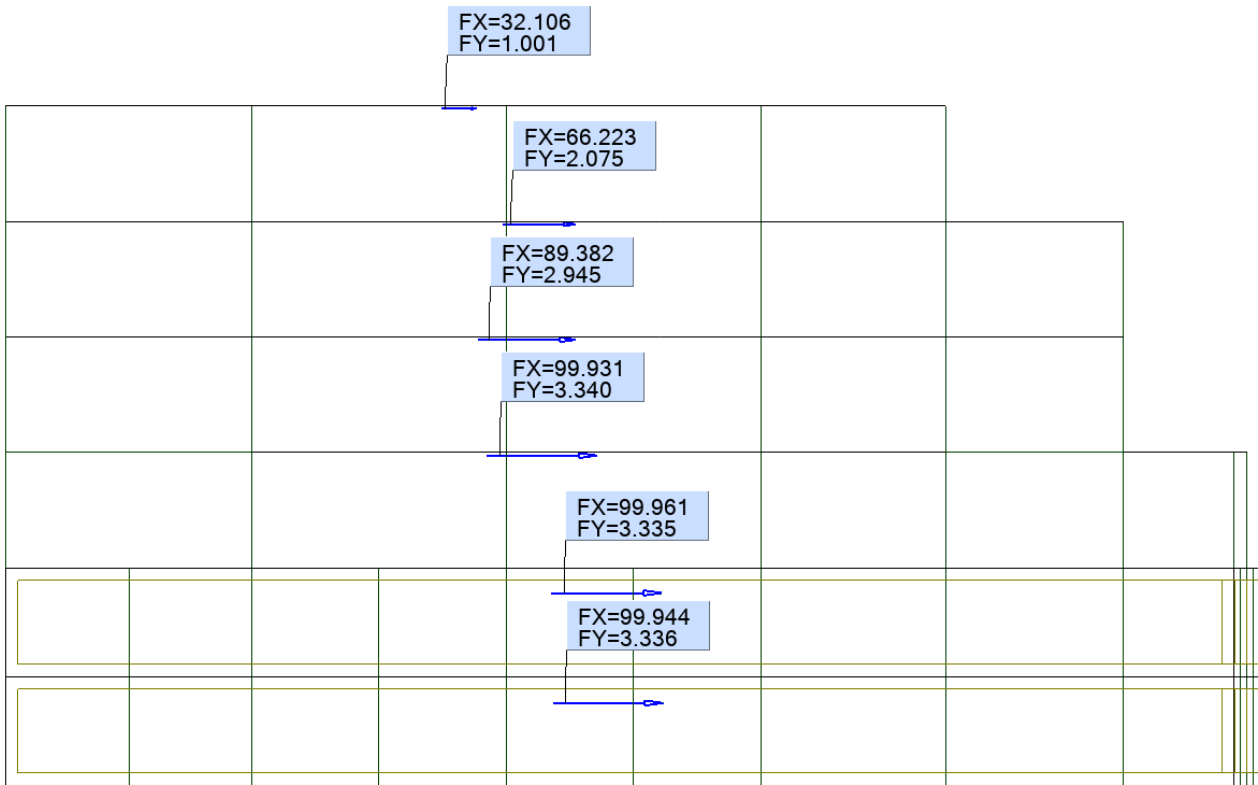
- ET2.

$W=446 \text{ t}$ $V= 0.194 \times 446 \rightarrow V=86.5 \text{ t}$



- ET3.

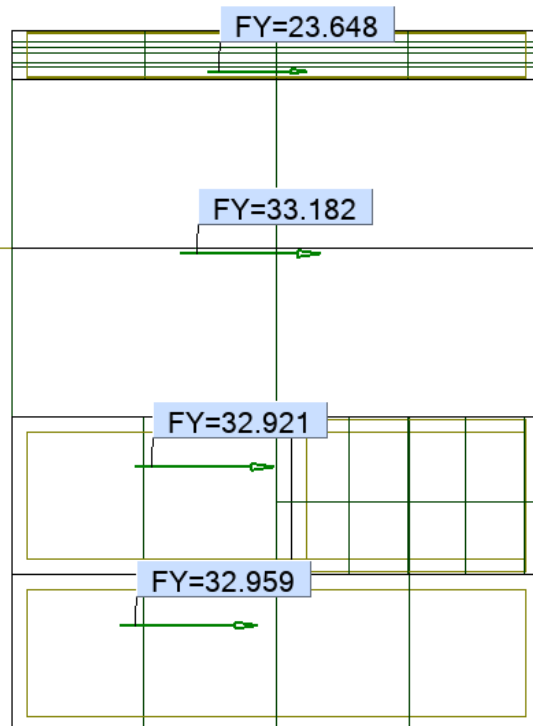
W=577 t V= 0.172 x 577 → V=99.3 t



6.3.2. Sismo Y, introducido en el modelo matemático (Análisis modal espectral).

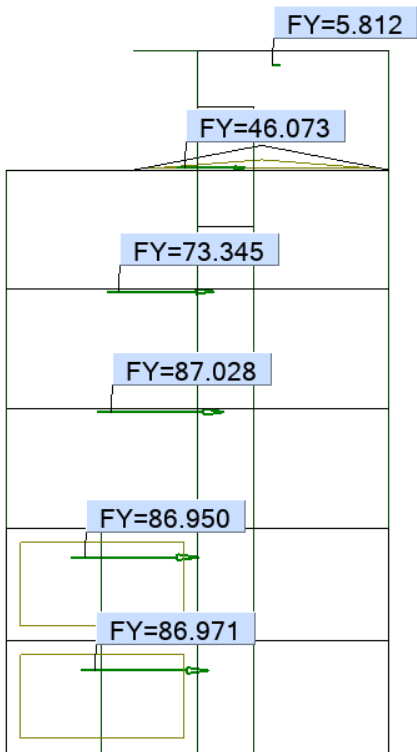
- ET1.

W=143 t V= 0.233 x 143 → V=33.32 t



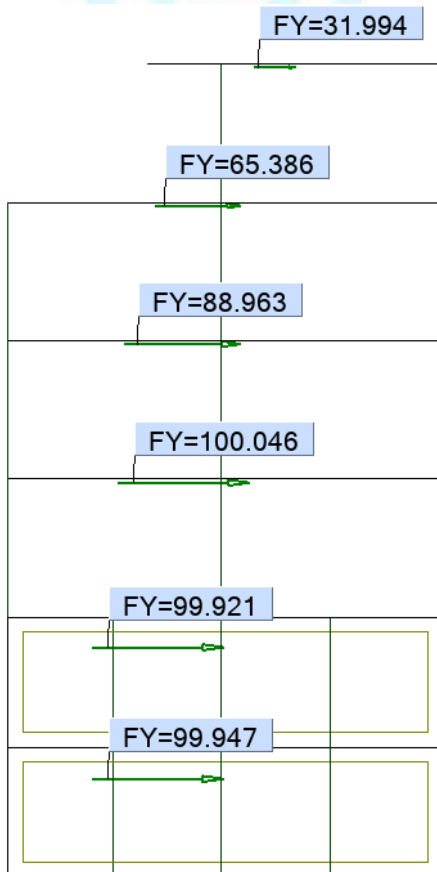
- ET2.

$W=446 \text{ t}$ $V=0.194 \times 446 \rightarrow V=86.5 \text{ t}$



- ET3.

$W=577 \text{ t}$ $V=0.172 \times 577 \rightarrow V=99.3 \text{ t}$



6.4. Evaluación de irregularidad torsional.

$C_d = 5.5$ → Factor de amplificación de desplazamientos, de elásticos a inelásticos (introducido e implícito en el programa de análisis estructural, y resultados presentes en la memoria).

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (12.8-15)$$

6.4.1. Con Sismo X.

ET1.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO X										CONTROL TORSION	cm	CONTROL TORS EXT.	
	ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$				1.2 $\Delta u MED$
2	7.60	6.40	7.60	6.40										
					5.00	4.20	4.20	5.00	4.60	5.52	Ok	6.44	Ok	
1	2.60	2.20	2.60	2.20										
					2.60	2.20	2.20	2.60	2.40	2.88	Ok	3.36	Ok	

CIMENTACIÓN / SUBSUELOS

Los valores de las tablas corresponden a desplazamientos inelásticos.

- ET2.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO X										CONTROL TORSION	cm	CONTROL TORS EXT.
	ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$			
3	12.90	12.20	12.90	12.20									
					4.50	4.60	4.50	4.60	4.55	5.46	Ok	6.37	Ok
2	8.40	7.60	8.40	7.60									
					5.00	4.50	4.50	5.00	4.75	5.70	Ok	6.65	Ok
1	3.40	3.10	3.40	3.10									
					3.40	3.10	3.10	3.40	3.25	3.90	Ok	4.55	Ok

CIMENTACIÓN / SUBSUELOS

Los valores de las tablas corresponden a desplazamientos inelásticos.

- ET3.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO X										CONTROL TORSION	cm	CONTROL TORS EXT.
	ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$			
4	21.60	20.70	21.60	20.70									
					5.00	5.40	5.00	5.40	5.20	6.24	Ok	7.28	Ok
3	16.60	15.30	16.60	15.30									
					6.20	5.80	5.80	6.20	6.00	7.20	Ok	8.40	Ok
2	10.40	9.50	10.40	9.50									
					6.50	5.80	5.80	6.50	6.15	7.38	Ok	8.61	Ok
1	3.90	3.70	3.90	3.70									
					3.90	3.70	3.70	3.90	3.80	4.56	Ok	5.32	Ok

CIMENTACIÓN / SUBSUELOS

6.4.2. Con Sismo Y.

- ET1.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO Y										CONTROL TORSION	CONTROL TORS EXT.	
	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm			
ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$	$1.2\Delta u MED$	cm	cm	
2	4.20	5.60	4.20	5.60									
					2.30	3.40	2.30	3.40	2.85	3.42	Ok	3.99	Ok
1	1.90	2.20	1.90	2.20									
					1.90	2.20	1.90	2.20	2.05	2.46	Ok	2.87	Ok

CIMENTACIÓN / SUBSUELOS

Los valores de las tablas corresponden a desplazamientos inelásticos

- ET2.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO Y										CONTROL TORSION	CONTROL TORS EXT.	
	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm			
ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$	$1.2\Delta u MED$	cm	cm	
3	12.90	13.50	12.90	13.50									
					4.40	5.10	4.40	5.10	4.75	5.70	Ok	6.65	Ok
2	8.50	8.40	8.50	8.40									
					4.90	4.70	4.70	4.90	4.80	5.76	Ok	6.72	Ok
1	3.60	3.70	3.60	3.70									
					3.60	3.70	3.60	3.70	3.65	4.38	Ok	5.11	Ok

CIMENTACIÓN / SUBSUELOS

- ET3.

CÁLCULO DE TORSIÓN

PLANTAS	SISMO Y										CONTROL TORSION	CONTROL TORS EXT.	
	cm	cm	cm	cm	cm	cm	cm	cm	cm	cm			
ALTAS	$\delta_e E1$	$\delta_e E2$	$\delta_u E1$	$\delta_u E2$	$\Delta u E1$	$\Delta u E2$	$\Delta u MIN$	$\Delta u MAX$	$\Delta u MED$	$1.2\Delta u MED$	cm	cm	
4	19.50	22.80	19.50	22.80									
					5.70	5.80	5.70	5.80	5.75	6.90	Ok	8.05	Ok
3	13.80	17.00	13.80	17.00									
					4.90	5.60	4.90	5.60	5.25	6.30	Ok	7.35	Ok
2	8.90	11.40	8.90	11.40									
					5.40	6.50	5.40	6.50	5.95	7.14	Ok	8.33	Ok
1	3.50	4.90	3.50	4.90									
					3.50	4.90	3.50	4.90	4.20	5.04	Ok	5.88	Ok

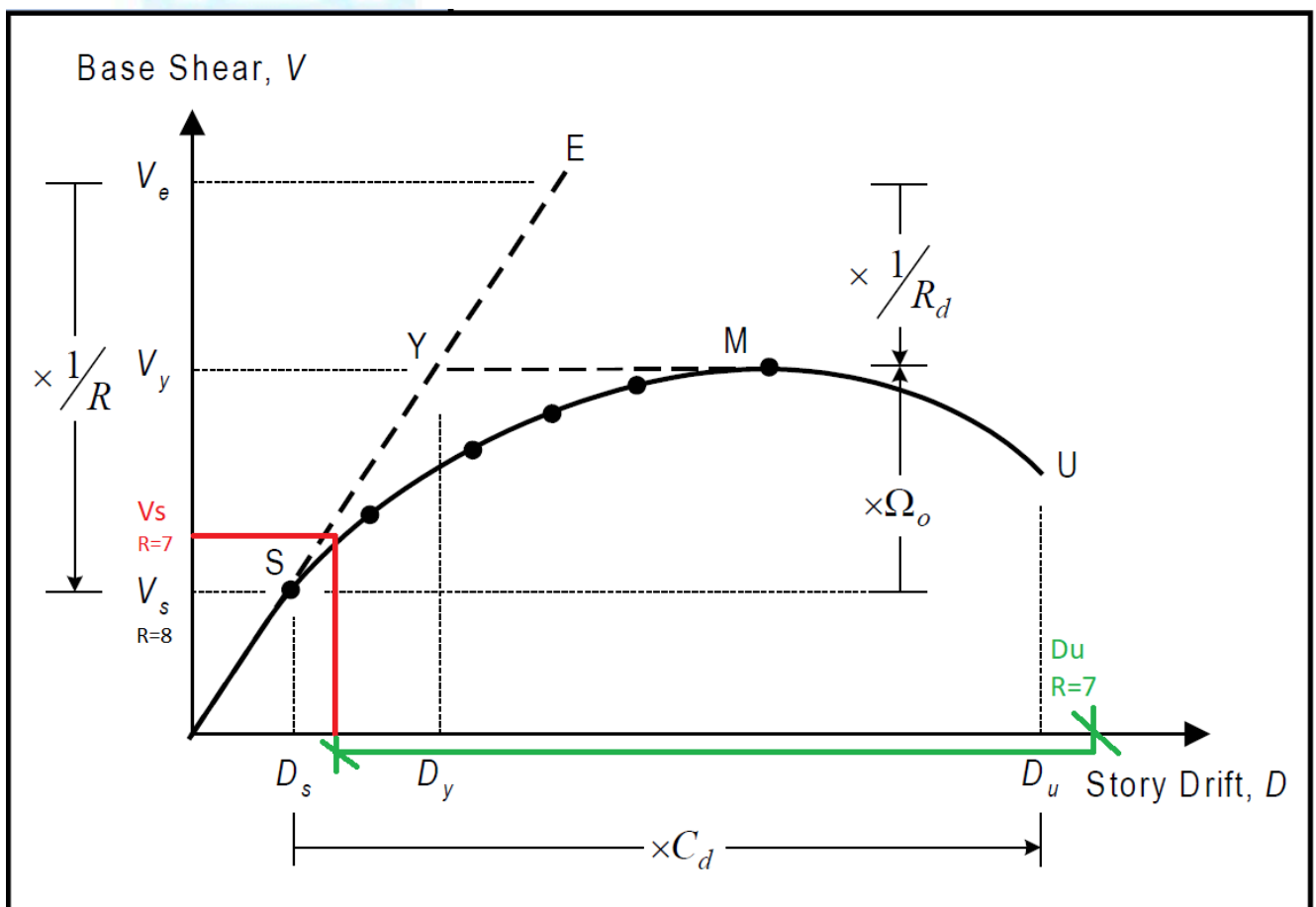
CIMENTACIÓN / SUBSUELOS

6.5. Cálculo de derivas.

$Cd = 5.5$ → Factor de amplificación de desplazamientos, de elásticos a inelásticos (introducido e implícito en el programa de análisis estructural, y resultados presentes en la memoria).

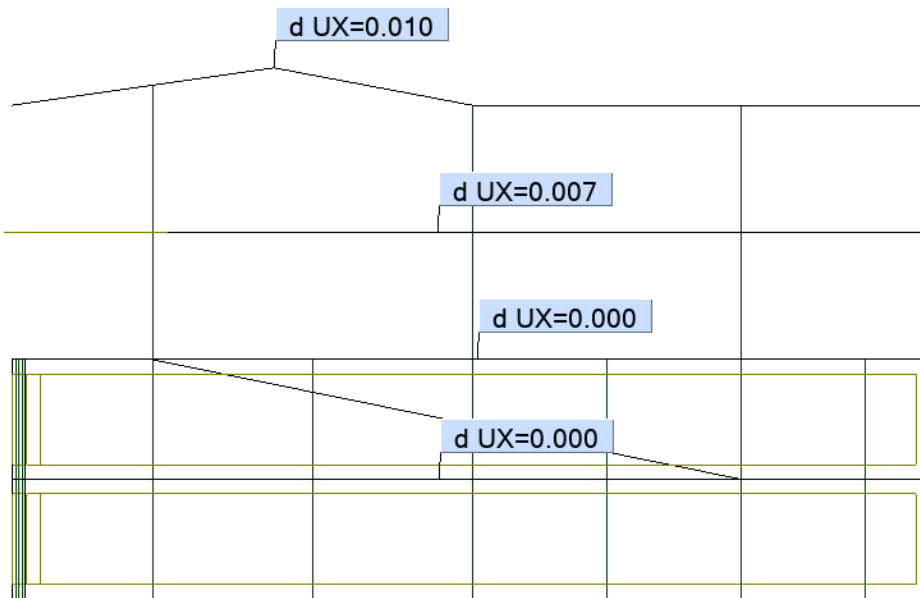
El utilizar un $R=7$ (en lugar de un $R=8$, acorde a normativa) para el cálculo deformaciones laterales, incrementa la deriva de piso en la proporción de la relación $\frac{Cd}{R}$, es decir:

$$\frac{\frac{5.5}{7}}{\frac{5.5}{8}} = 1.14$$



6.5.1. Con Sismo X.

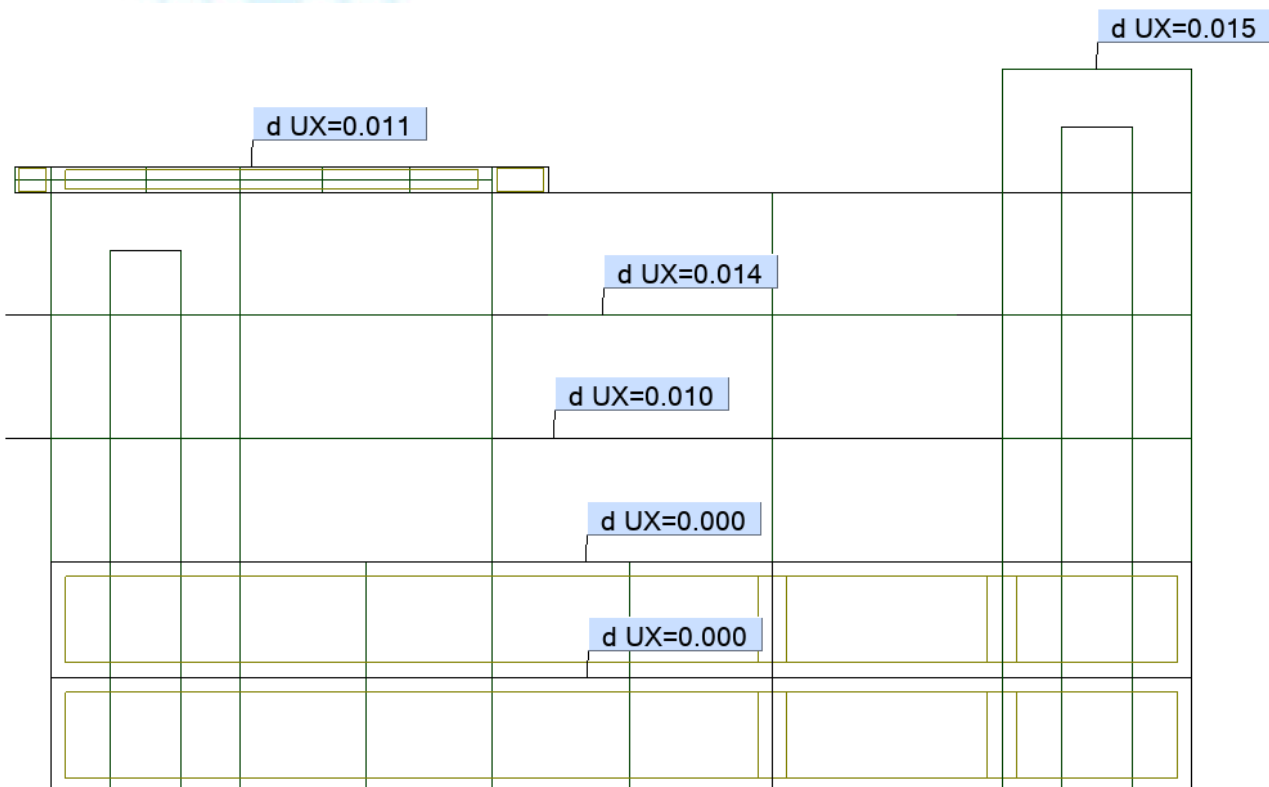
- ET1



DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

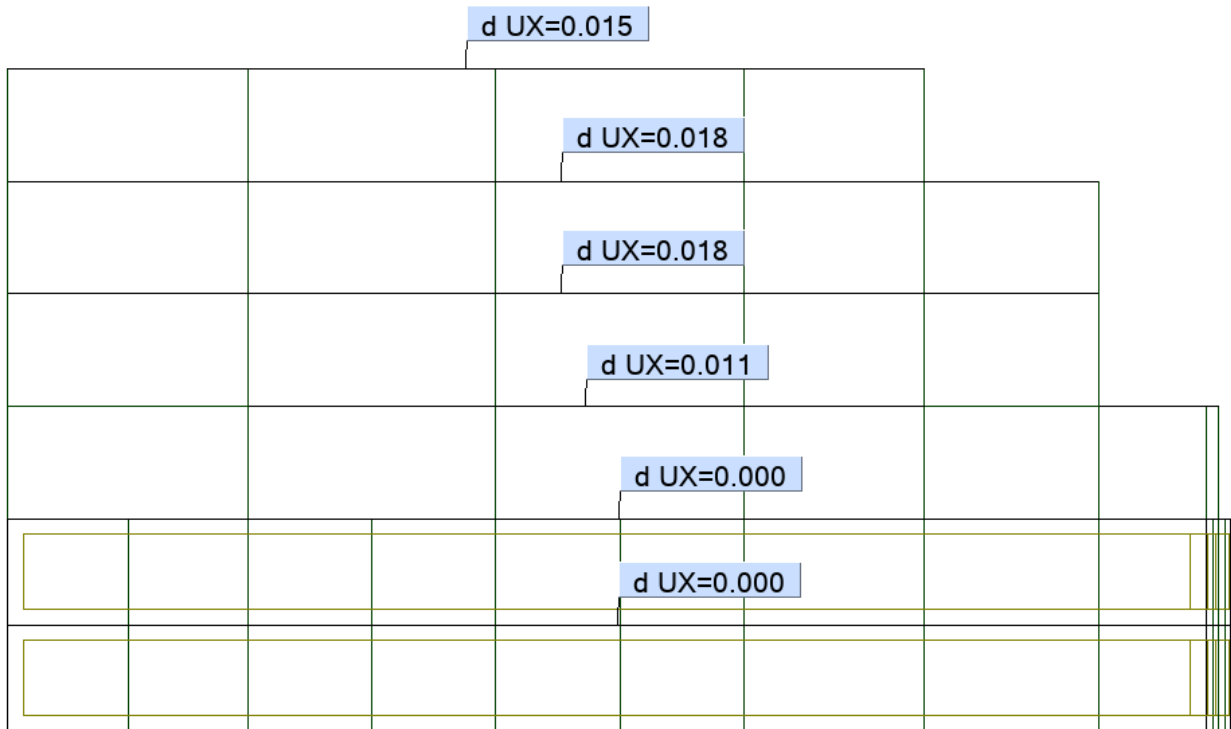
- ET2



DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

- ET3

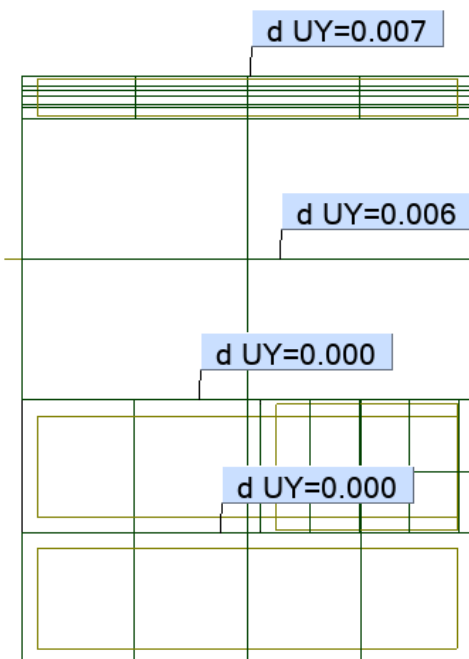


DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

6.1.1. Con Sismo Y.

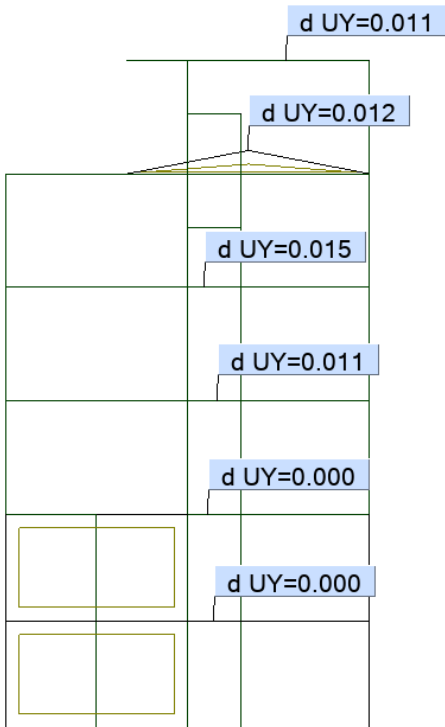
- ET1



DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

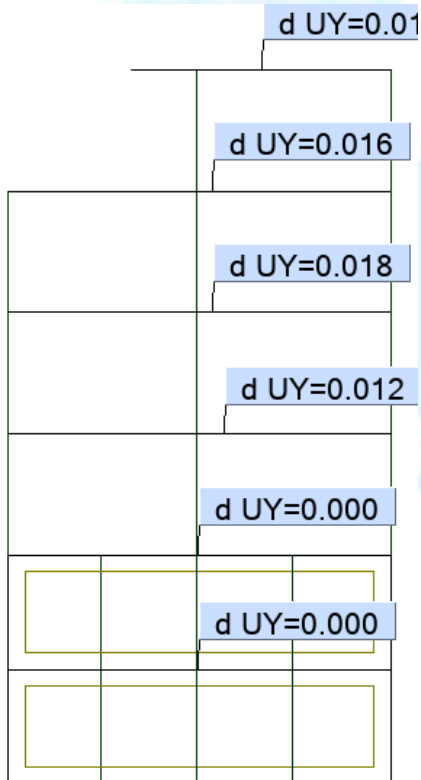
- ET2



DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

- ET3



DERIVA / $h_{\text{entrepiso}} < 0.020 \rightarrow \text{ok!}$

Los valores de las tablas corresponden a desplazamientos inelásticos. ($du \rightarrow$ relación entre deriva y altura de entrepiso)

7. Diseño Estructural.

El diseño de la estructura de hormigón armado ha sido realizado conforme lo dispuesto en las normas NEC-SE-HM y ACI318-19. Para la parte correspondiente al acero estructural se utilizaron las disposiciones de AISC 360-16, AISC 358-16, AISC 341-16 y NEC-SE-AC.

7.1. Vigas.

- Compacidad.

RELACIONES LIMITE PARA VIGAS (SMF) AISC341.16

TABLA I-8-1 SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

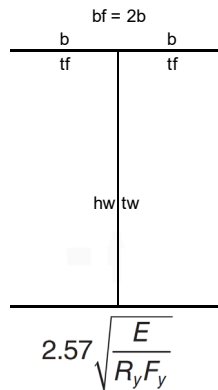
ELEMENTOS NO RIGIDIZADOS (UNSTIFFENED ELEMENTS)
ALAS DE VIGAS

ELEMENTOS RIGIDIZADOS (STIFFENED ELEMENTS)
ALMAS DE VIGAS

E (Ksi) 29,000.00

$$0.32 \sqrt{\frac{E}{R_y F_y}}$$

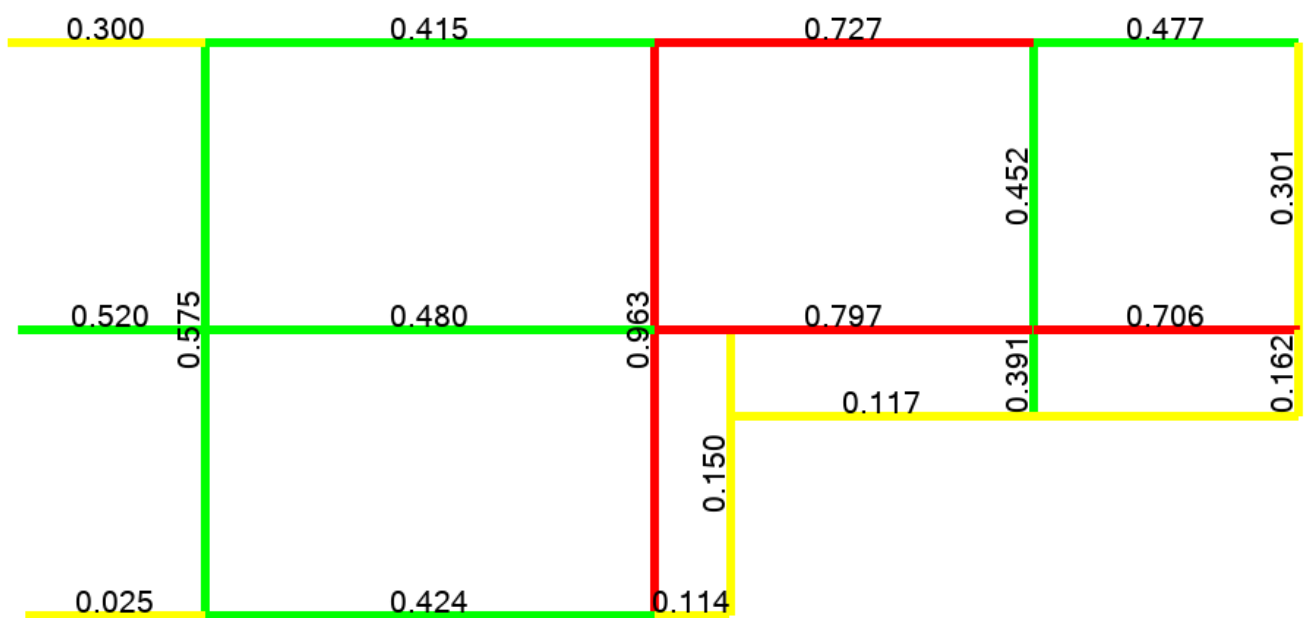
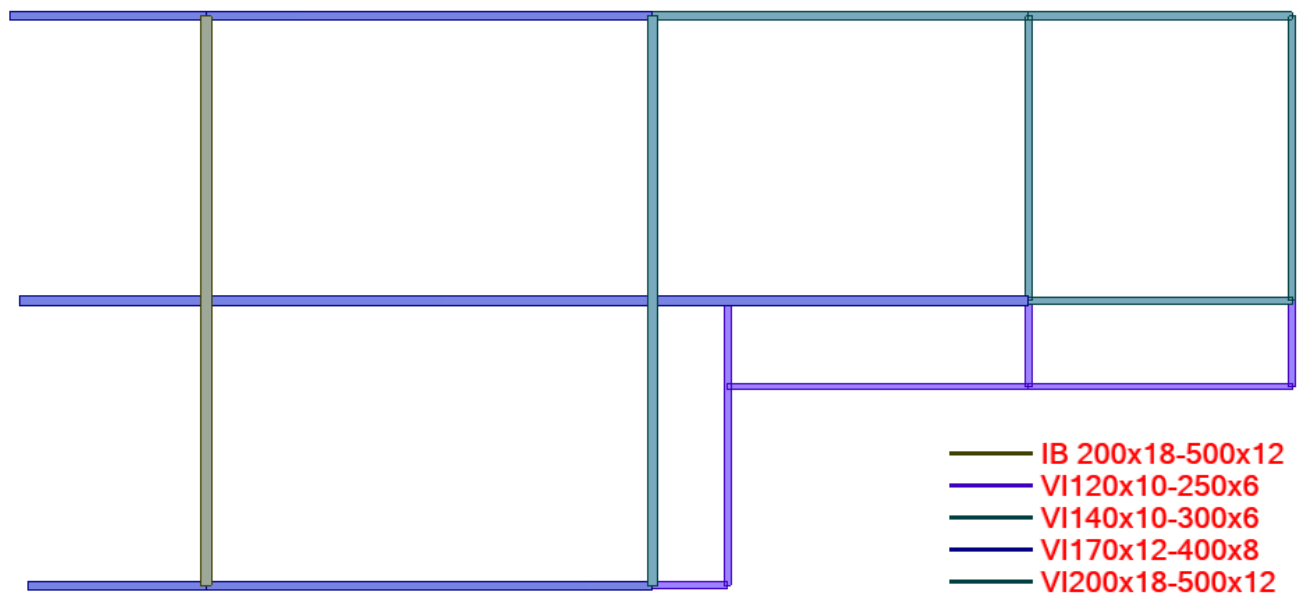
tf (mm)	Ry	
	1.5	1.1
	bf max (mm)	
	Fy(Ksi)	Fy(Ksi)
	36	50
3	44.5	44.1
4	59.3	58.8
5	74.2	73.5
6	89.0	88.2
7	103.8	102.9
8	118.7	117.6
9	133.5	132.3
10	148.3	147.0
11	163.1	161.7
12	178.0	176.4
13	192.8	191.0
14	207.6	205.7
15	222.5	220.4
16	237.3	235.1
17	252.1	249.8
18	267.0	264.5
19	281.8	279.2
20	296.6	293.9
21	311.5	308.6
22	326.3	323.3
23	341.1	338.0
24	356.0	352.7
25	370.8	367.4
26	385.6	382.1
27	400.4	396.8
28	415.3	411.5
29	430.1	426.2
30	444.9	440.9
32	474.6	470.3
35	519.1	514.4
40	593.3	587.8
45	667.4	661.3
50	741.6	734.8



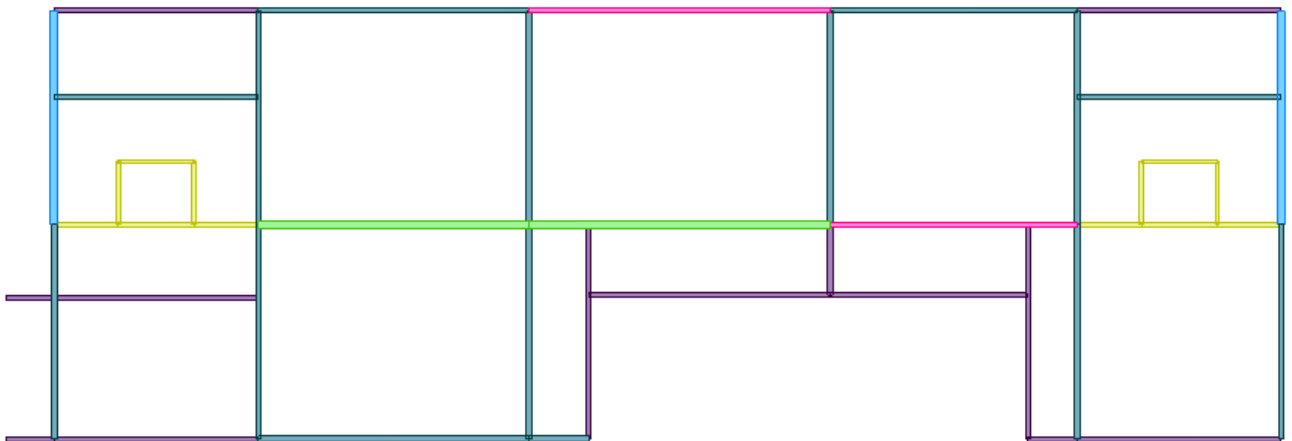
tw (mm)	Ry	
	1.5	1.1
	hw max (mm)	
	Fy(Ksi)	Fy(Ksi)
	36	50
3	178.7	177.0
4	238.2	236.1
5	297.8	295.1
6	357.3	354.1
7	416.9	413.1
8	476.5	472.1
9	536.0	531.1
10	595.6	590.1
11	655.1	649.1
12	714.7	708.2
13	774.2	767.2
14	833.8	826.2
15	893.4	885.2
16	952.9	944.2
17	1,012.5	1,003.2
18	1,072.0	1,062.2
19	1,131.6	1,121.3
20	1,191.1	1,180.3
25	1,488.9	1,475.3

TIPO	DIMENSIONES							
	ALA				ALMA			
	B (mm)	e (mm)	Bmax (mm)	Compacidad símica	H (mm)	e (mm)	Hmax (mm)	Compacidad símica
VI (200-120)x18-500x12	200	18	264.5	0.76 ok	464	12	708.2	0.66 ok
VI (180-100)x12-300x8	180	12	176.4	1.02 ok	276	8	472.1	0.58 ok
VI 200x18-500x12	200	18	264.5	0.76 ok	464	12	708.2	0.66 ok
VI 200x15-450x10	200	15	220.4	0.91 ok	420	10	590.1	0.71 ok
VI 170x12-400x8	170	12	176.4	0.96 ok	376	8	472.1	0.80 ok
VI 140x12-350x6	140	12	176.4	0.79 ok	326	6	354.1	0.92 ok
VI 140x10-300x6	140	10	147	0.95 ok	280	6	354.1	0.79 ok
VI 120x10-250x6	120	10	147	0.82 ok	230	6	354.1	0.65 ok
VI 100x8-200x4	100	8	117.6	0.85 ok	184	4	236.1	0.78 ok

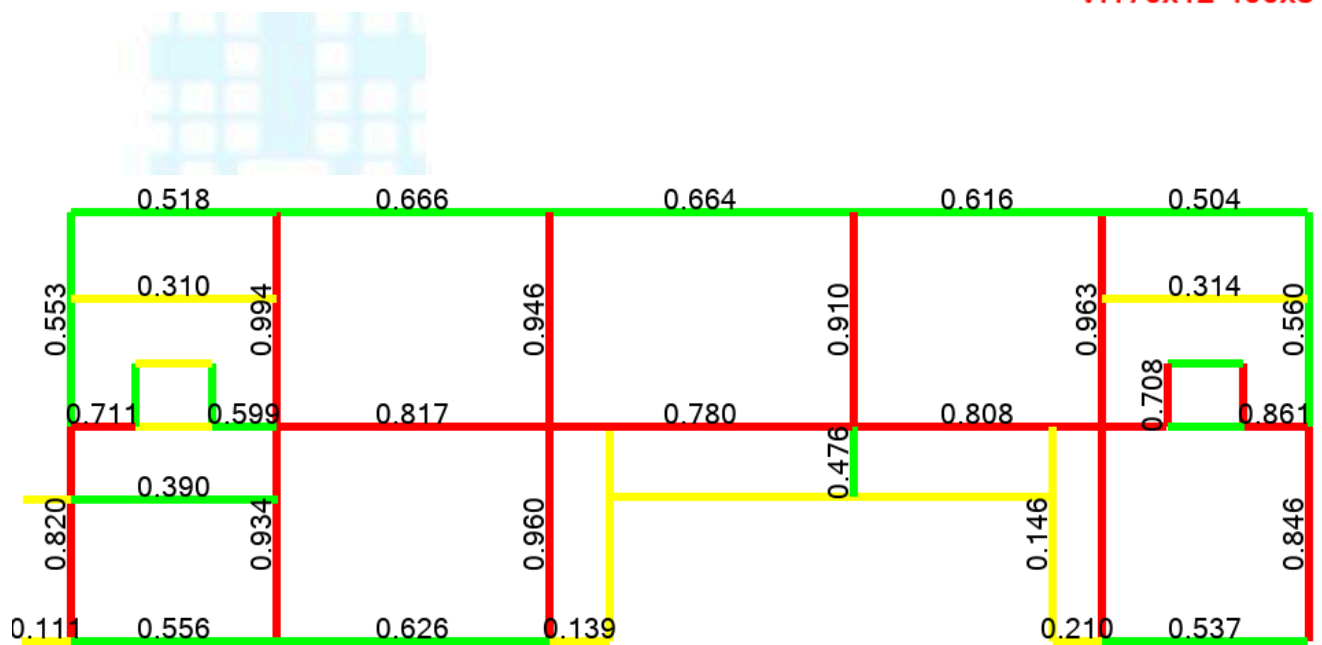
- Utilización vigas Planta alta 1 – ET1.



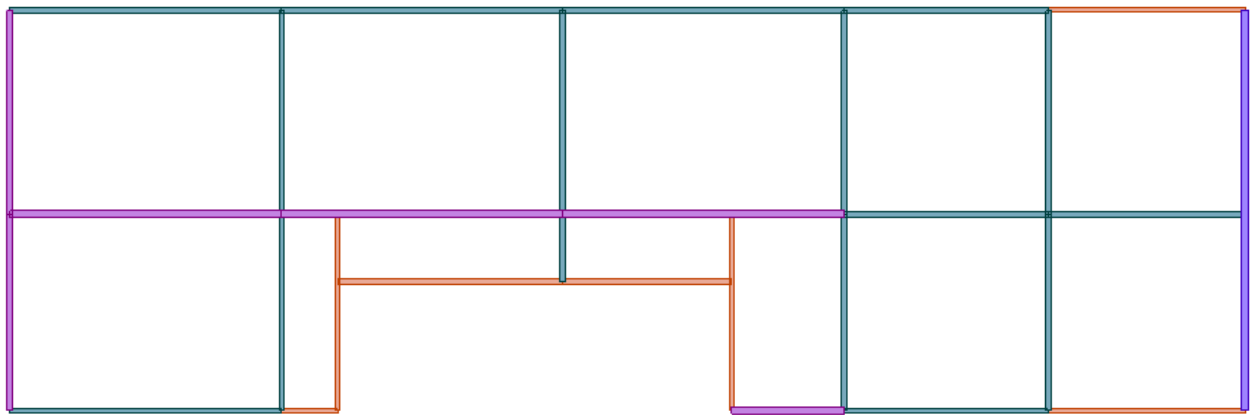
- Utilización vigas Planta alta 2 – ET2.



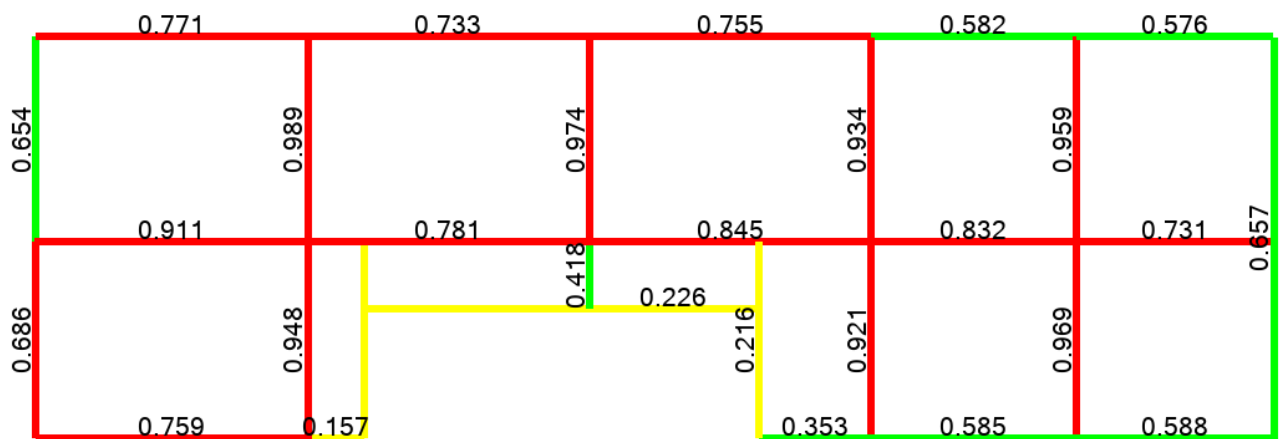
- IB 200x12-300x8
- VI100x8-200x4
- VI120x10-250x6
- VI140x10-300x6
- VI140x12-350x6
- VI170x12-400x8



- Utilización vigas Planta alta 3 – ET3.



- VI120x10-250x6
- VI140x10-300x6
- VI170x12-400x8
- VI200x15-450x8



- **Deflexiones.**

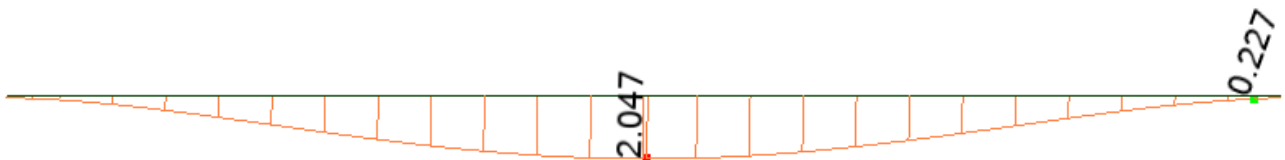
Table 1. Deflection Limits, adapted from IBC Table 1604.4

CONSTRUCTION	LIVE	SNOW OR WIND	DEAD + LIVE
Roof members:			
Supporting plaster ceiling	// 360	// 360	// 240
Supporting nonplaster ceiling	// 240	// 240	// 180
Not supporting ceiling	// 180	// 180	// 120
Roof members supporting metal roofing:	// 150	-	// 60
Floor Members	// 360	-	// 240
Exterior walls and interior partitions:			
With brittle finishes	-	// 240	-
With flexible finishes	-	// 120	-
Secondary wall members supporting metal siding	-	// 90	-

AISC STEEL DESIGN GUIDE 3, Serviceability Design considerations for steel Buildings.

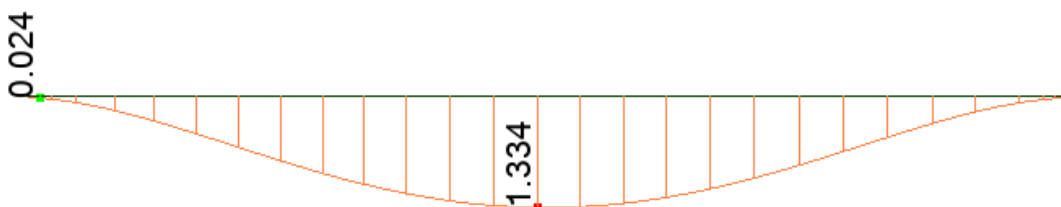
- VIGA EJES B1-B3 → Planta alta 1 – ET1.

Deflexión con carga de servicio D+L



$$\Delta_{MAX} = 1020/240 = 4.25\text{cm} \rightarrow 2.05\text{cm} < 4.25\text{cm} \rightarrow \text{ok!}$$

- VIGA EJES P1 – P3 → ET3 → Planta Baja.

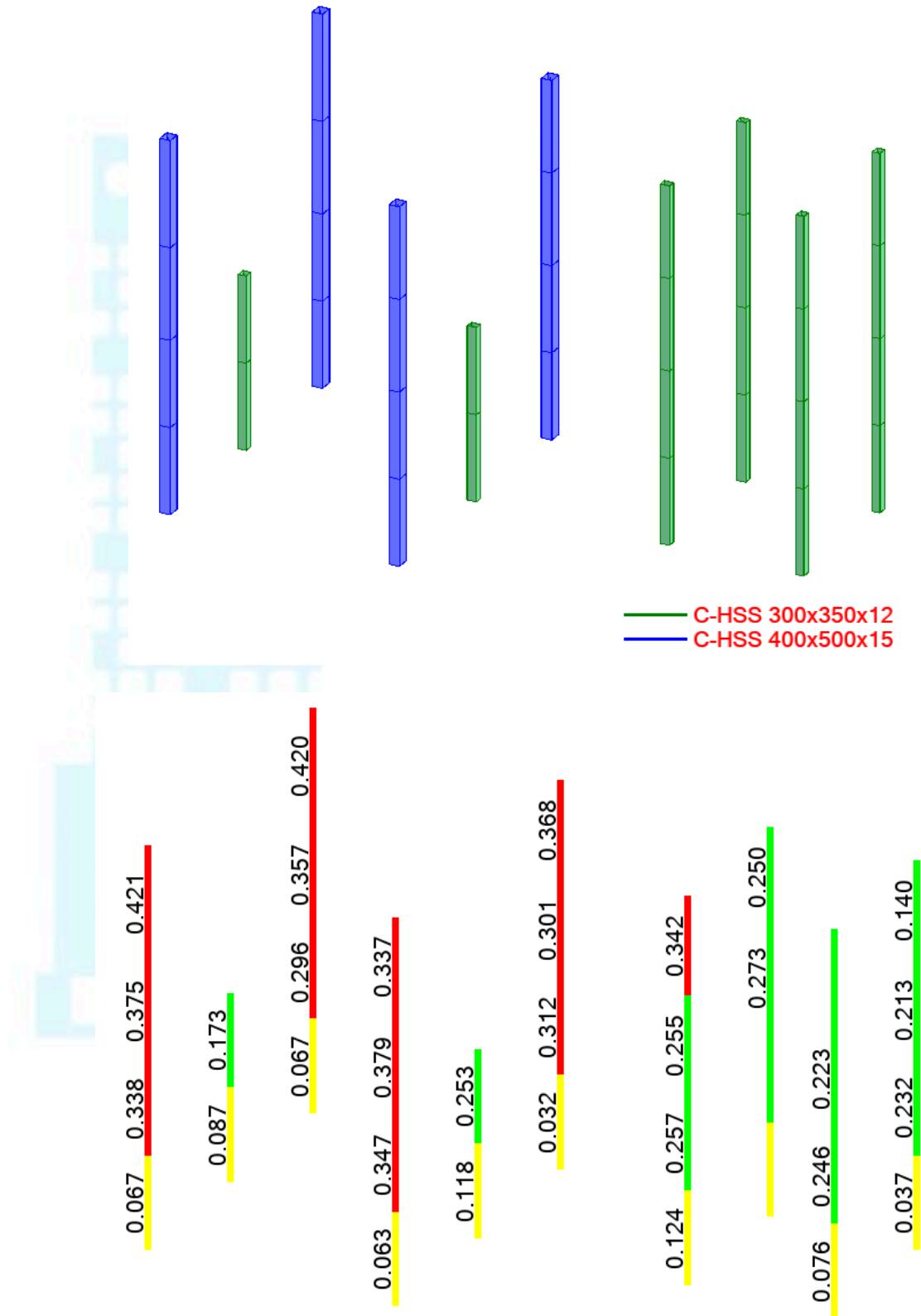


$$\Delta_{MAX} = 985/240 = 4.10\text{cm} \rightarrow 1.4\text{cm} < 4.10\text{cm} \rightarrow \text{ok!}$$

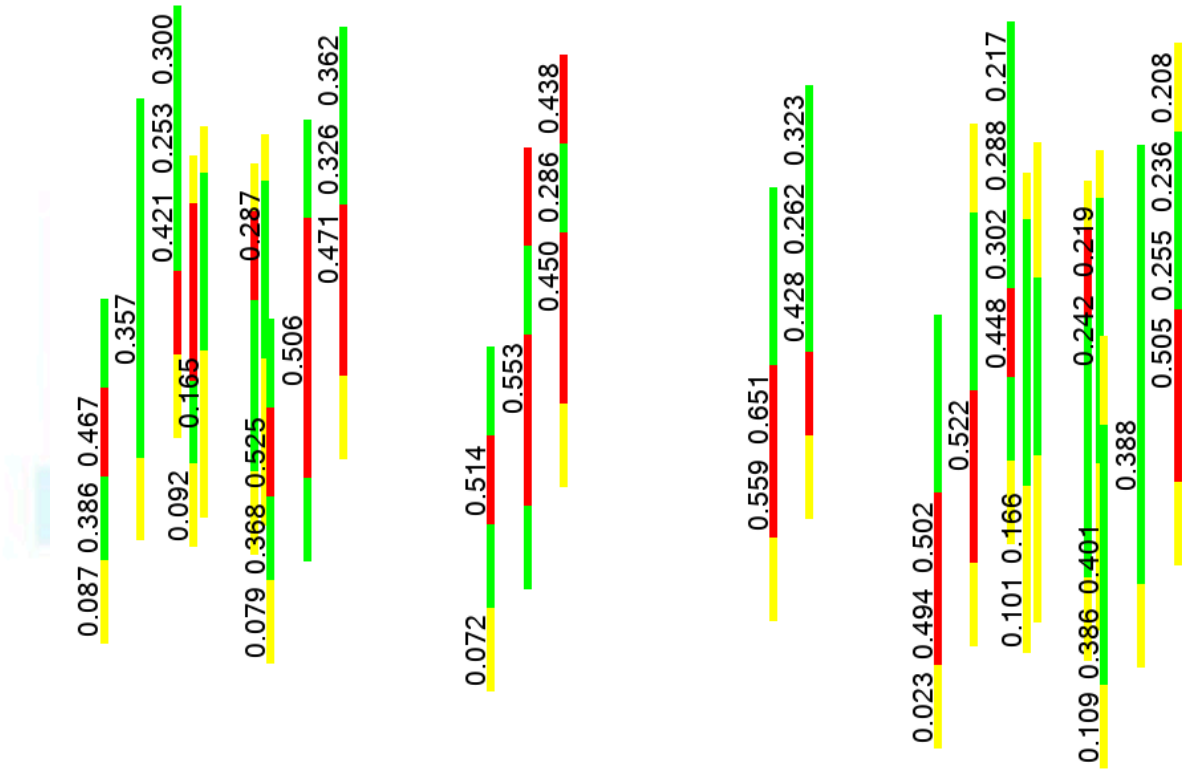
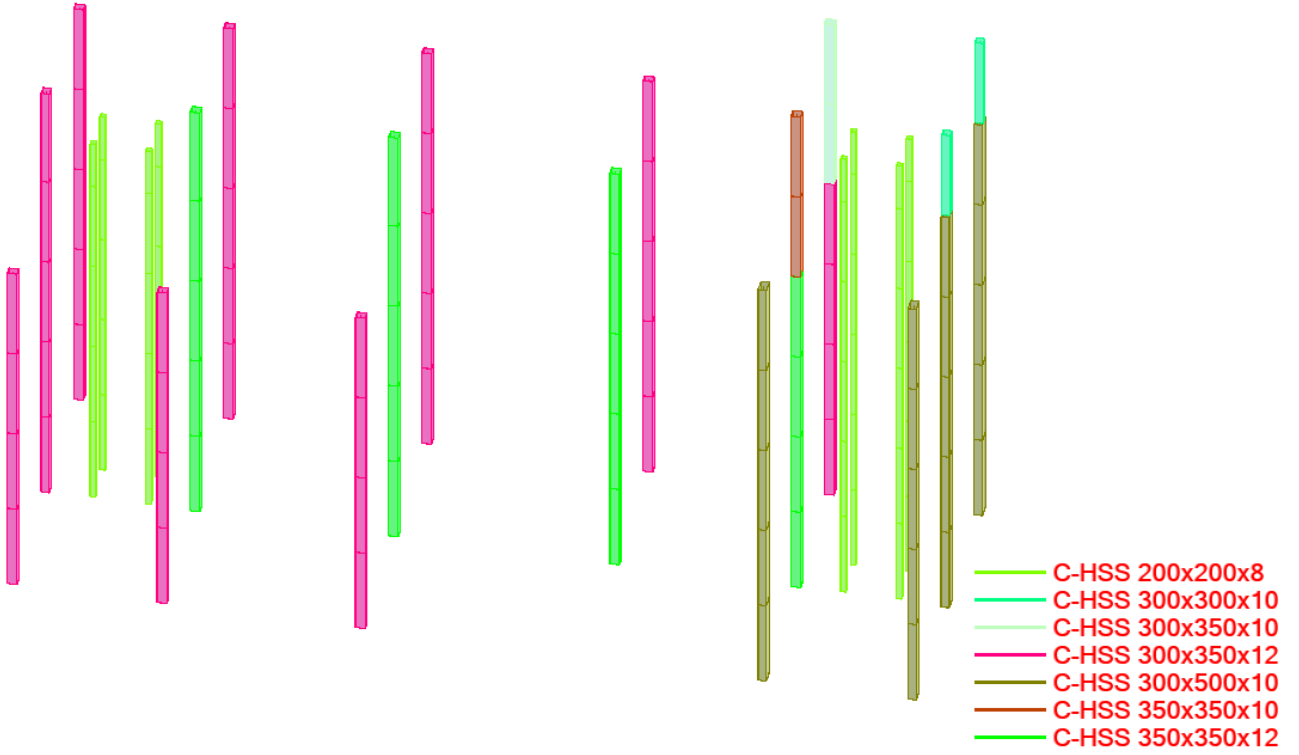
7.2. Columnas.

- Utilización de columnas C-HSS

- ET1

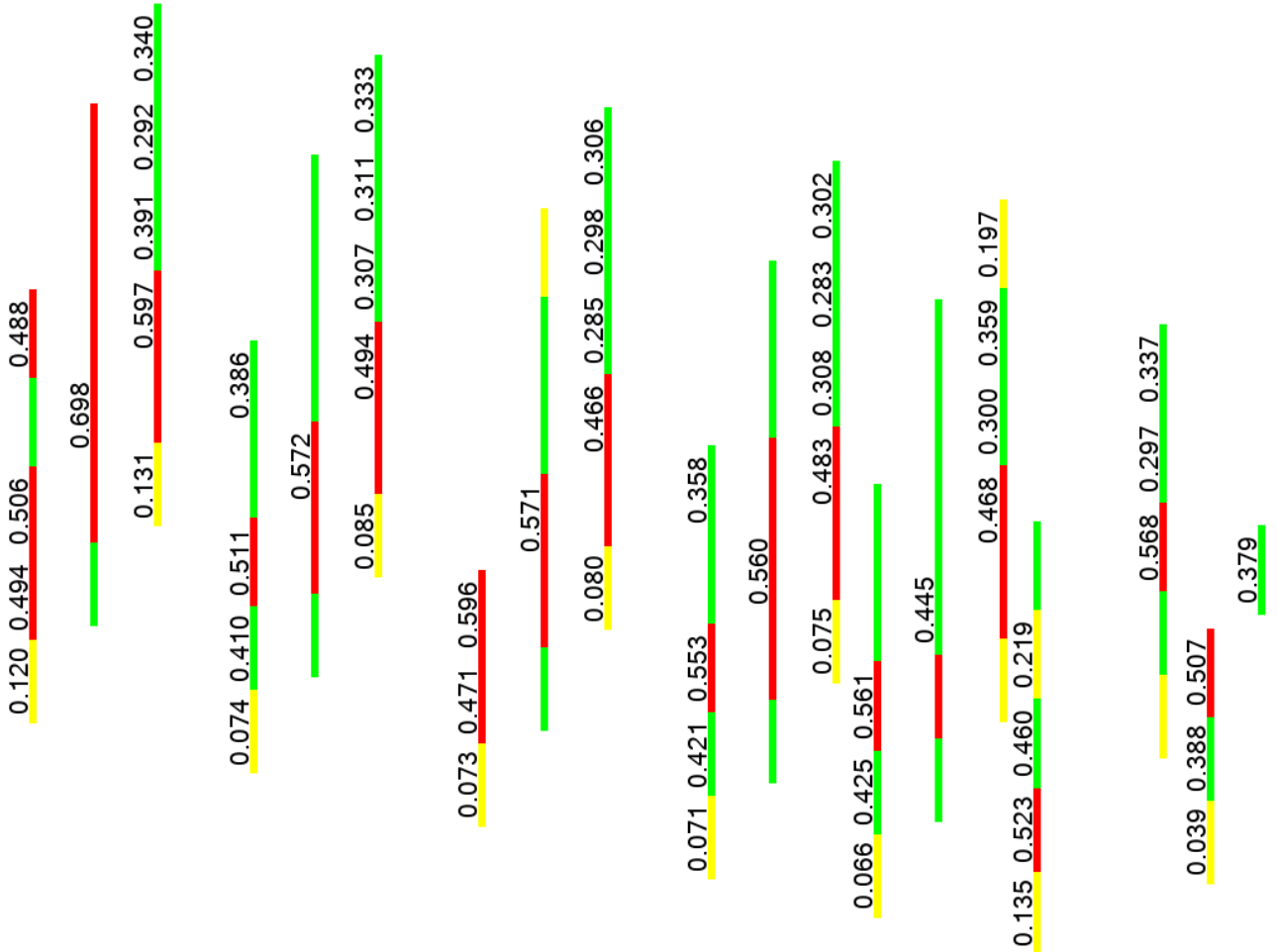
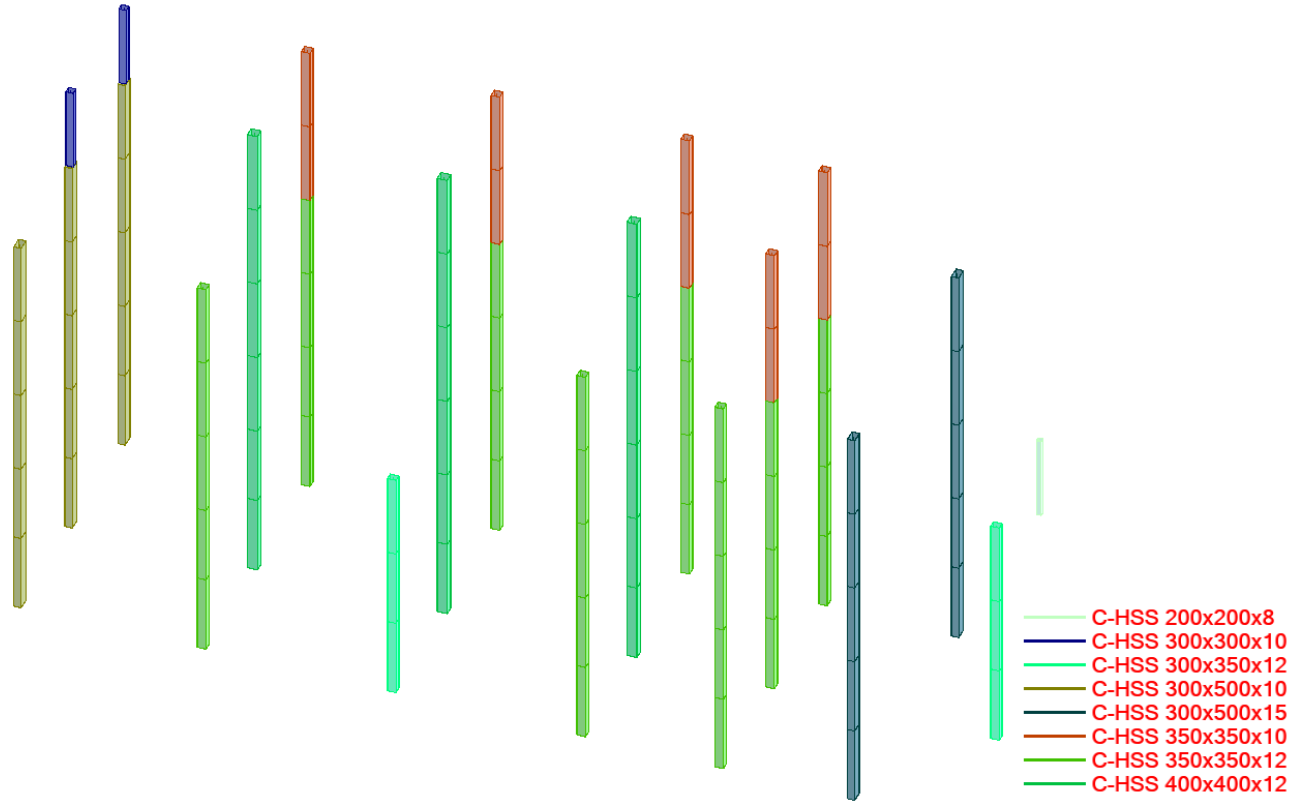


Se comprueba el diseño adecuado.



Se comprueba el diseño adecuado.

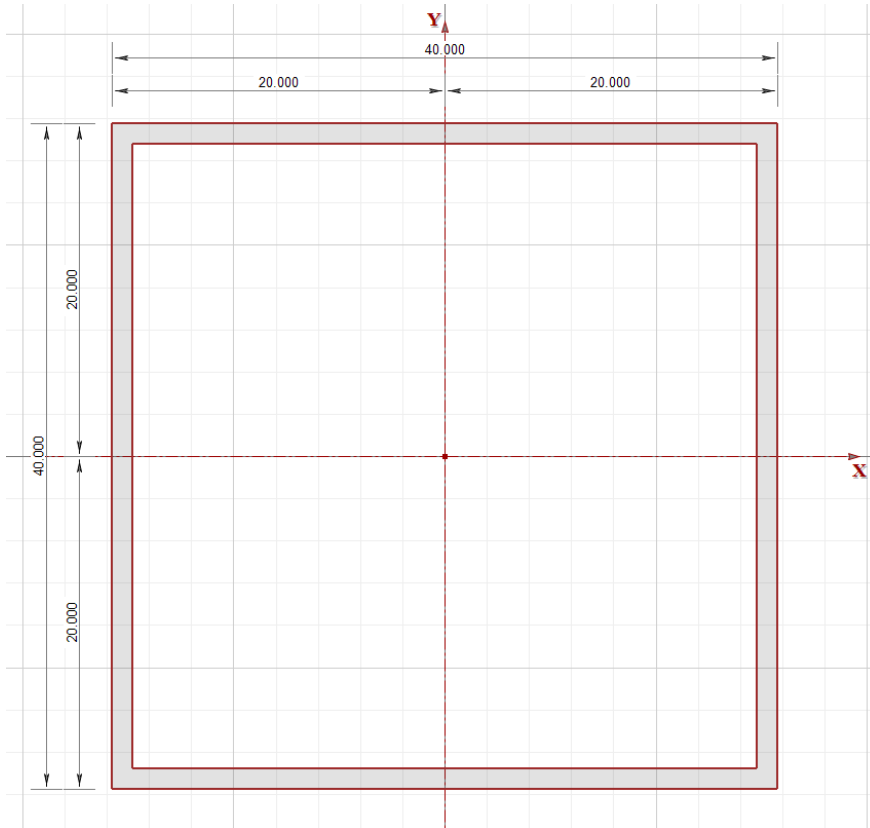
• ET3



Se comprueba el diseño adecuado.

7.3. Conexiones.

- WUF-W_CONEXIÓN + MOMENT RATIO.

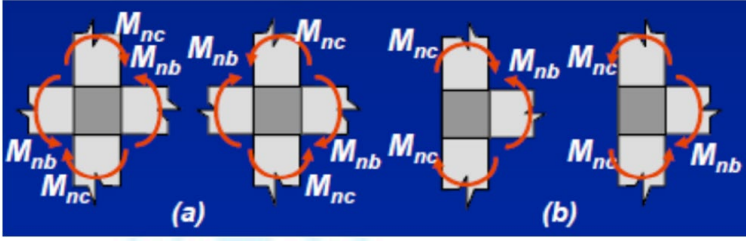


Properties		
Total Width =	40.000	cm
Total Height =	40.000	cm
Centroid, Xo =	0.000	cm
Centroid, Yo =	0.000	cm
X-Bar (Right) =	20.000	cm
X-Bar (Left) =	20.000	cm
Y-Bar (Top) =	20.000	cm
Y-Bar (Bot) =	20.000	cm
Max Thick =	1.200	cm
Area, Ax =	186.24	cm ²
Inertia, Ixx =	4.677E+004	cm ⁴
Inertia, Iyy =	4.677E+004	cm ⁴
Inertia, Ixy =	0.000	cm ⁴
Sx (Top) =	2338.68	cm ³
Sx (Bot) =	2338.68	cm ³
Sy (Left) =	2338.68	cm ³
Sy (Right) =	2338.68	cm ³
rx =	15.848	cm
ry =	15.848	cm
Plastic Zx =	2710.66	cm ³
Plastic Zy =	2710.66	cm ³

(Conservadoramente se desprecia la aportación del hormigón en la Inercia de la columna.)

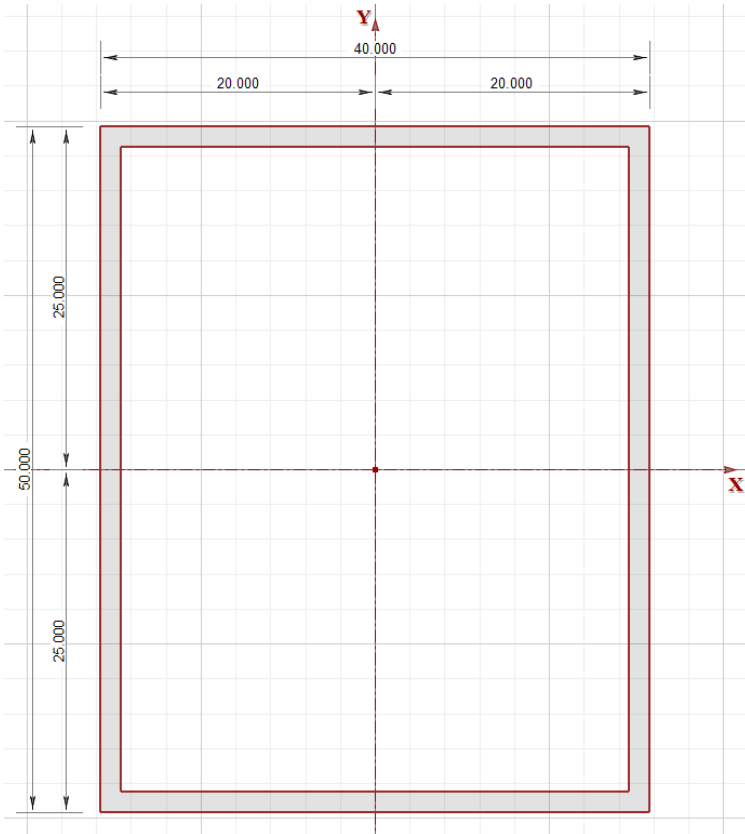


Properties		
Total Width =	17.000	cm
Total Height =	40.000	cm
Centroid, Xo =	0.000	cm
Centroid, Yo =	0.000	cm
X-Bar (Right) =	8.500	cm
X-Bar (Left) =	8.500	cm
Y-Bar (Top) =	20.000	cm
Y-Bar (Bot) =	20.000	cm
Max Thick =	1.200	cm
Area, Ax =	70.880	cm ²
Inertia, Ixx =	1.890E+004	cm ⁴
Inertia, Iyy =	984.20	cm ⁴
Inertia, Ixy =	0.000	cm ⁴
Sx (Top) =	945.21	cm ³
Sx (Bot) =	945.21	cm ³
Sy (Left) =	115.79	cm ³
Sy (Right) =	115.79	cm ³
rx =	16.331	cm
ry =	3.726	cm
Plastic Zx =	1074.27	cm ³
Plastic Zy =	179.42	cm ³

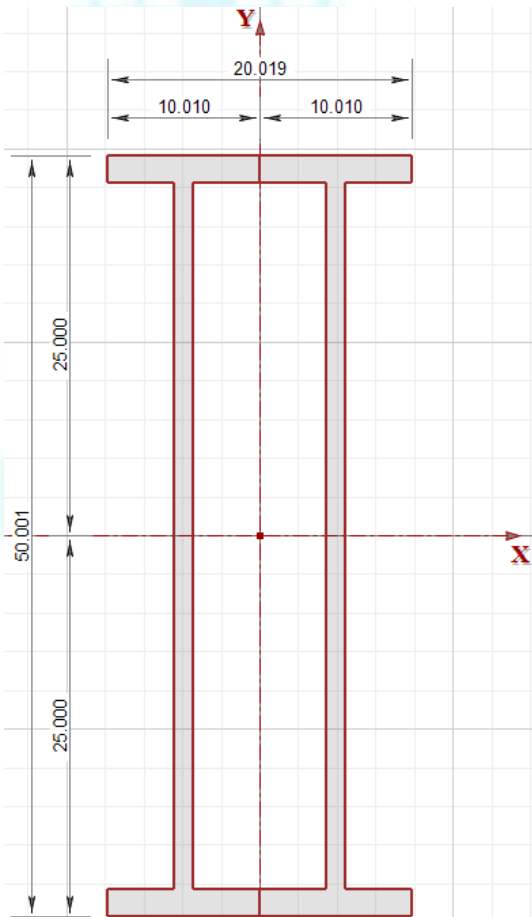
WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION - FR			
COLUMNA			
		C-HSS 400x400x12	
$\Sigma M_{pc}^* = \Sigma Z_c (F_{yc} - \alpha_s P_r / A_g)$			
$\alpha_s =$	1		→ Factor para ajustar la magnitud de fuerza (1.0 LRFD, 1.5 ASD).
$Z_c =$	2711 cm^3	= 165.44 in^3	→ Módulo plástico de la sección de la columna respecto al eje de flexión.
$F_{yc} =$	3515.3 kg/cm^2	= 50 ksi	→ Tensión mínima de fluencia de la columna.
$F_{uc} =$	4570.0 kg/cm^2	= 65 ksi	→ Resistencia a tracción mínima especificada de la columna.
$A_g =$	186 cm^2	= 28.83 in^2	→ Área bruta de la columna.
$P_r =$	190 T	= 418.87 $kips$	→ Resistencia axial requerida de acuerdo a ASCE41-16.D1.4a.
$M_{pc} =$	5868.15 $kip.in$	= 67.61 $T.m$	
$\Sigma M_{pc} =$	11736.29 $kip.in$	= 135.22 $T.m$	
VIGAS			
$\Sigma (M_{pr} + M_{uv})$		AISC358-16→8.4	$M_{pr} = C_{pr} R_y F_y Z_e$ AISC358-16→2.4.3-1
		Excepciones: Columnas de último piso de edificios de varias plantas. AISC341.16 E3.4.a.a.1.	
$\alpha_s =$	1		→ Factor para ajustar la magnitud de fuerza (1.0 LRFD, 1.5 ASD).
$C_{pr} =$	1.40	WUF-W AISC358-16→8.7.1	→ Factor para considerar la resistencia máxima de la conexión.
User Note: The C_{pr} value of 1.4 for WUF-W moment connections is based on experimental data that shows a high degree of strain hardening.			
$R_y =$	1.1		→ Relación entre esfuerzo de fluencia esperado y mínimo esfuerzo de fluencia.
$F_{yb} =$	3515.3 kg/cm^2	= 50 ksi	→ Tensión mínima de fluencia de la viga.
$F_{ub} =$	4570.0 kg/cm^2	= 65 ksi	→ Resistencia a tracción mínima de la viga.
$dc \rightarrow$	40 cm	= 15.75 in	→ Lado de la columna en el sentido de flexión (depth)
$Sh =$	0 cm	= 0.00 in	→ La localización de la rótula plástica se considerará en la cara de la columna, es decir $sh=0$ → AISC358-19 8.7.Step2.
VIGA 1		VIGA 2	
VL170x12-400x8		VL170x12-400x8	
$Z_e = Z_x =$	1074 cm^3	= 65.54 in^3	$Z_e = Z_x =$ 1074 cm^3 = 65.54 in^3
$Z_e = Z_x =$ Módulo plástico efectivo de la sección (o conexión) en la ubicación de la rótula plástica, para WUF-W → $Z_e = Z_x$ → AISC 358-16 8.			
$M_{pr1} =$	5046.54 $kip.in$	= 58.14 $T.m$	$M_{pr2} =$ 5046.54 $kip.in$ = 58.14 $T.m$
$ln1 =$	710 cm	= 279.53 in	$ln2 =$ 710 cm = 279.53 in
$V_{G1} =$	21.0 T	= 46.30 $kips$	$V_{G2} =$ 21.0 T = 46.30 $kips$
$V_{h1} =$	82.40 kip	= 37.38 T	$V_{h2} =$ 82.40 kip = 37.38 T
$M_{uv1} =$	648.85 $kip.in$	= 7.48 $T.m$	$M_{uv2} =$ 648.85 $kip.in$ = 7.48 $T.m$

		<p>WUF-W (AISC358-16 8.4)</p> $M_{uv} = V_h \left(\frac{d_c}{2} \right)$
$V_G = W_U \ell_n / 2$ $V_E = (M_{pr1} + M_{pr2}) / \ell_n$		
$\Sigma M_{prb} = 11390.79 \text{ kip.in} = 131.24 \text{ T.m}$		
$\frac{\Sigma M_{pc}^*}{\Sigma M_{pb}^*} > 1.0$ $\frac{135.22}{131.24} > 1$		
<div style="border: 1px solid black; padding: 2px; display: inline-block;"> 1.03 > 1 OK </div>		
RESISTENCIA DE DISEÑO A CORTE DE LA VIGA		
$V_u = V_h = 82.40 \text{ kips} = 37.37 \text{ T}$ $\phi V_n = 60.8 \text{ T ok!}$		
RESISTENCIA DE DISEÑO DE PLACA DE CORTE Y SOLDADURA		
$t_p = 12 \text{ mm} = 0.472 \text{ in}$	→ Espesor de la placa.	$\geq t \text{ beam web}$
$h_p = 280 \text{ mm} = 11.024 \text{ in}$	→ Altura de la placa.	
$F_{yp} = 3515.35 \text{ kg/cm}^2 = 50 \text{ ksi}$	→ Tensión mínima de fluencia de la placa.	
$R_y = 1.1$	→ Relación: esfuerzo de fluencia esperado y mínimo esfuerzo de fluencia → placa	
$t_c = 12 \text{ mm} = 0.472 \text{ in}$	→ Espesor de la cara de la columna para la conexión.	
$t_{bw} = 8 \text{ mm} = 0.315 \text{ in}$	→ Espesor del alma de la viga para la conexión.	
$F_{EXX} = 4921.49 \text{ ksi} = 70$	→ Grado del electrodo.	
$t_w = 10 \text{ mm} = 0.394 \text{ in}$	→ Espesor de soldadura de filete.	
$l_{w1} = 340 \text{ mm} = 13.386 \text{ in}$	→ Longitud de suelda (dimensión mayor).	
$l_{w2} = 280 \text{ mm} = 11.024 \text{ in}$	→ Longitud de suelda (dimensión menor).	
Resistencia de la placa → fluencia → $t_{pmin} = \frac{V_u}{\phi h_p (0.60 R_{yp} F_{yp})} = 0.302 \text{ in} = 7.7 \text{ mm}$ espesor mínimo placa		
REVISIÓN DE SOLDADURA PLACA - COLUMNA		
Resistencia por metal de soldadura → $\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 0.75(0.60 F_{EXX}) t_e l_{w1} = 117.37 \text{ kips} = 53.23 \text{ T}$		
Resistencia metal base → fluencia → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 1(0.60 F_y) t_c l_{w1} = 189.72 \text{ kips} = 86.04 \text{ T}$		
Resistencia metal base → ruptura → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 0.75(0.60 F_u) t_c l_{w1} = 184.98 \text{ kips} = 83.89 \text{ T}$		
53.23 T ok!		
REVISIÓN DE SOLDADURA PLACA - VIGA		
Resistencia por metal de soldadura → $\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 0.75(0.60 F_{EXX}) t_e l_{w2} = 96.65 \text{ kips} = 43.83 \text{ T}$		
Resistencia metal base → fluencia → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 1(0.60 F_y) t_{bw} l_{w2} = 104.16 \text{ kips} = 47.24 \text{ T}$		
Resistencia metal base → ruptura → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 0.75(0.60 F_u) t_{bw} l_{w2} = 101.56 \text{ kips} = 46.06 \text{ T}$		
43.83 T ok!		

PLANTA ALTA 1 – NUDO A3 – ET1.



Properties		
Total Width =	40.000	cm
Total Height =	50.000	cm
Centroid, Xo =	0.000	cm
Centroid, Yo =	0.000	cm
X-Bar (Right) =	20.000	cm
X-Bar (Left) =	20.000	cm
Y-Bar (Top) =	25.000	cm
Y-Bar (Bot) =	25.000	cm
Max Thick =	1.500	cm
Area, Ax =	261.00	cm ²
Inertia, lxx =	9.655E+004	cm ⁴
Inertia, lyy =	6.828E+004	cm ⁴
Inertia, lxy =	0.000	cm ⁴
Sx (Top) =	3861.83	cm ³
Sx (Bot) =	3861.83	cm ³
Sy (Left) =	3413.79	cm ³
Sy (Right) =	3413.79	cm ³
rx =	19.233	cm
ry =	16.174	cm
Plastic Zx =	4566.75	cm ³
Plastic Zy =	3914.25	cm ³



Properties		
Total Width =	20.019	cm
Total Height =	50.001	cm
Centroid, Xo =	0.007	cm
Centroid, Yo =	-0.029	cm
X-Bar (Right) =	10.010	cm
X-Bar (Left) =	10.010	cm
Y-Bar (Top) =	25.000	cm
Y-Bar (Bot) =	25.000	cm
Max Thick =	1.800	cm
Area, Ax =	183.36	cm ²
Inertia, lxx =	6.182E+004	cm ⁴
Inertia, lyy =	5215.04	cm ⁴
Inertia, lxy =	-0.452	cm ⁴
Sx (Top) =	2472.64	cm ³
Sx (Bot) =	2472.64	cm ³
Sy (Left) =	521.00	cm ³
Sy (Right) =	521.00	cm ³
rx =	18.361	cm
ry =	5.333	cm
Plastic Zx =	3026.98	cm ³
Plastic Zy =	918.57	cm ³

WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION - FR

COLUMNA

C-HSS 400x500x15

$$\Sigma M_{pc}^* = \Sigma Z_c (F_{yc} - \alpha_s P_r / A_g)$$

$\alpha_s =$	1			→ Factor para ajustar la magnitud de fuerza (1.0 LRFD, 1.5 ASD).
$Z_c =$	4567	cm ³	=	278.70 in ³ → Módulo plástico de la sección de la columna respecto al eje de flexión.
$F_{yc} =$	3515.3	kg/cm ²	=	50 ksi → Tensión mínima de fluencia de la columna.
$F_{uc} =$	4570.0	kg/cm ³	=	65 ksi → Resistencia a tracción mínima especificada de la columna.
$A_g =$	261	cm ²	=	40.46 in ² → Área bruta de la columna.
$P_r =$	80	T	=	176.37 kips → Resistencia axial requerida de acuerdo a ASCE41-16.D1.4a.

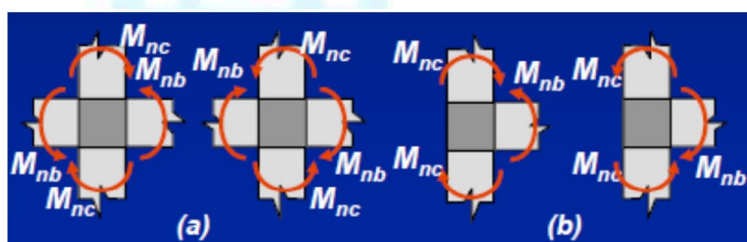
$$M_{pc} = 12719.77 \text{ kip.in} = 146.55 \text{ T.m}$$

$$\Sigma M_{pc} = 25439.54 \text{ kip.in} = \mathbf{293.09 \text{ T.m}}$$

VIGAS

$$\Sigma (M_{pr} + M_{uv}) \quad \text{AISC358-16} \rightarrow 8.4$$

$$M_{pr} = C_{pr} R_y F_y Z_e \quad \text{AISC358-16} \rightarrow 2.4.3-1$$



Excepciones: Columnas de último piso de edificios de varias plantas. AISC341.16 E3.4.a.a.1.

$\alpha_s =$	1			→ Factor para ajustar la magnitud de fuerza (1.0 LRFD, 1.5 ASD).
$C_{pr} =$	1.40	WUF-W AISC358-16	→ 8.7.1	→ Factor para considerar la resistencia máxima de la conexión.

User Note: The C_{pr} value of 1.4 for WUF-W moment connections is based on experimental data that shows a high degree of strain hardening.

$R_y =$	1.1			→ Relación entre esfuerzo de fluencia esperado y mínimo esfuerzo de fluencia.
$F_{yb} =$	3515.3	kg/cm ²	=	50 ksi → Tensión mínima de fluencia de la viga.
$F_{ub} =$	4570.0	kg/cm ³	=	65 ksi → Resistencia a tracción mínima de la viga.
$d_c \rightarrow$	40	cm	=	15.75 in → Lado de la columna en el sentido de flexión (depth)
$Sh =$	0	cm	=	0.00 in → La localización de la rótula plástica se considerará en la cara de la columna, es decir sh=0 → AISC358-19 8.7.Step2.

VIGA 1

VIGA 2

VL(200x120)X18-500x12

NO EXISTE

$$Z_e = Z_x = 3027 \text{ cm}^3 = 184.72 \text{ in}^3$$

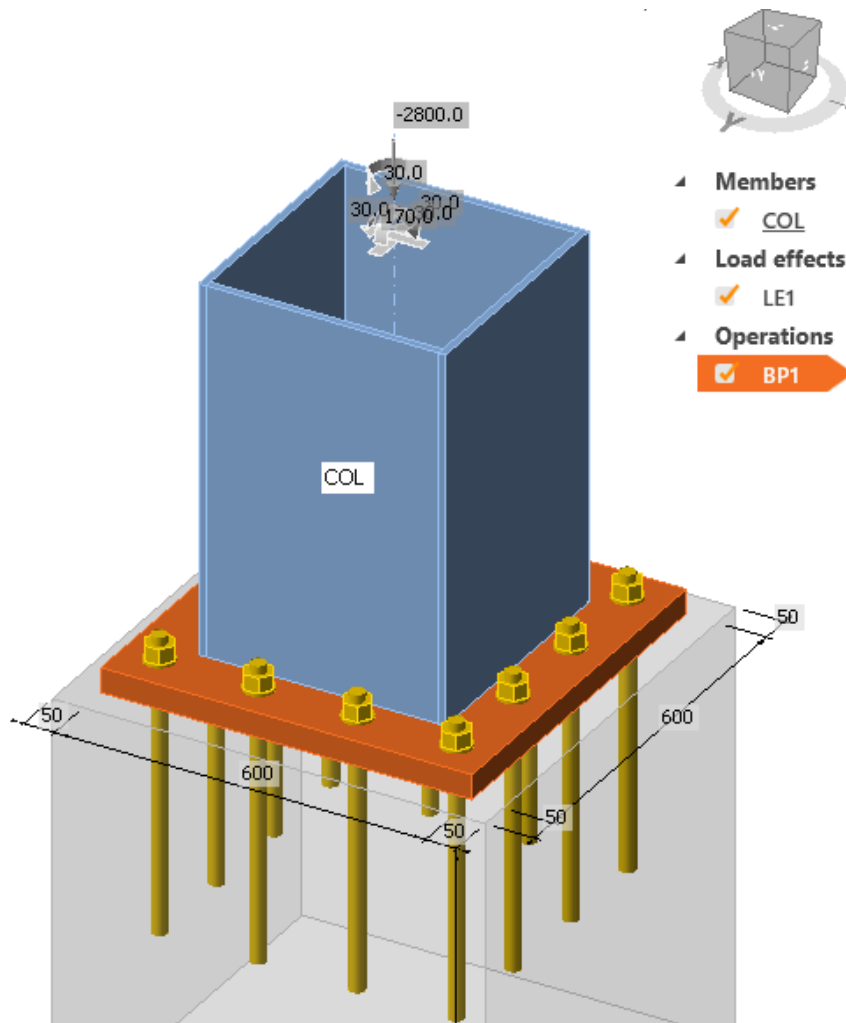
$$Z_e = Z_x = 0 \text{ cm}^3 = 0.00 \text{ in}^3$$

$Z_e = Z_x =$ Módulo plástico efectivo de la sección (o conexión) en la ubicación de la rótula plástica, para WUF-W → $Z_e = Z_x$ → AISC 358-16 8.

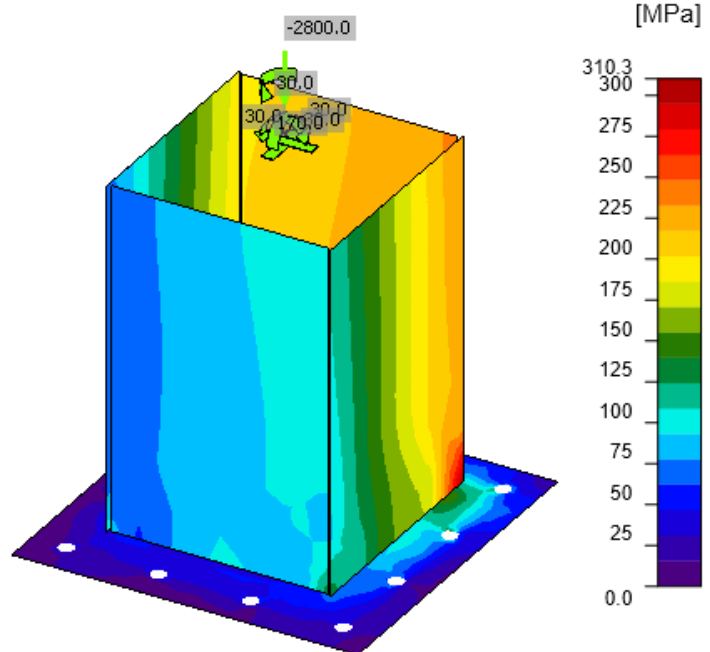
M_{pr1} =	14223.35	kip.in	=	163.87	T.m	M_{pr2} =	0.00	kip.in	=	0.00	T.m
ln1=	1025	cm	=	403.54	in	ln2=	0	cm	=	0.00	in
V _{G1} =	40.0	T	=	88.18	kips	V _{G2} =	0.0	T	=	0.00	kips
V _{h1} =	158.68	kip	=	71.98	T	V _{h2} =	0.00	kip	=	0.00	T
M_{uv1} =	1249.42	kip.in	=	14.39	T.m	M_{uv2} =	0.00	kip.in	=	0.00	T.m

		<p>WUF-W (AISC358-16 8.4)</p> $M_{uv} = V_h \left(\frac{d_c}{2} \right)$
$V_G = W_U \ell_n / 2$ $V_E = (M_{pr1} + M_{pr2}) / \ell_n$		
$\Sigma M_{prb} = 15472.77 \text{ kip.in} = 178.27 \text{ T.m}$		
$\frac{\Sigma M_{pc}^*}{\Sigma M_{pb}^*} > 1.0$ $\frac{293.09}{178.27} > 1$		
$1.64 > 1$ OK		
RESISTENCIA DE DISEÑO A CORTE DE LA VIGA		
$V_u = V_h = 158.68 \text{ kips} = 71.96 \text{ T}$ $\phi V_n = 204 \text{ T ok!}$		
RESISTENCIA DE DISEÑO DE PLACA DE CORTE Y SOLDADURA		
$t_p = 18 \text{ mm} = 0.709 \text{ in}$	→ Espesor de la placa.	$\geq t \text{ beam web}$
$h_p = 350 \text{ mm} = 13.780 \text{ in}$	→ Altura de la placa.	
$F_{yp} = 3515.35 \text{ kg/cm}^2 = 50 \text{ ksi}$	→ Tensión mínima de fluencia de la placa.	
$R_y = 1.1$	→ Relación: esfuerzo de fluencia esperado y mínimo esfuerzo de fluencia → placa	
$t_c = 15 \text{ mm} = 0.591 \text{ in}$	→ Espesor de la cara de la columna para la conexión.	
$t_{bw} = 12 \text{ mm} = 0.472 \text{ in}$	→ Espesor del alma de la viga para la conexión.	
$F_{EXX} = 4921.49 \text{ ksi} = 70$	→ Grado del electrodo.	
$t_w = 15 \text{ mm} = 0.591 \text{ in}$	→ Espesor de soldadura de filete.	
$l_{w1} = 420 \text{ mm} = 16.535 \text{ in}$	→ Longitud de suelda (dimensión mayor).	
$l_{w2} = 350 \text{ mm} = 13.780 \text{ in}$	→ Longitud de suelda (dimensión menor).	
Resistencia de la placa → fluencia → $t_{pmin} = \frac{V_u}{\phi h_p (0.60 R_{yp} F_{yp})} = 0.465 \text{ in} = 11.8 \text{ mm}$ espesor mínimo placa		
REVISIÓN DE SOLDADURA PLACA - COLUMNA		
Resistencia por metal de soldadura → $\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 0.75(0.60 F_{EXX}) t_e l_{w1} = 217.47 \text{ kips} = 98.63 \text{ T}$		
Resistencia metal base → fluencia → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 1(0.60 F_y) t_c l_{w1} = 292.95 \text{ kips} = 132.86 \text{ T}$		
Resistencia metal base → ruptura. → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 0.75(0.60 F_u) t_c l_{w1} = 285.63 \text{ kips} = 129.54 \text{ T}$		
197.25 T ok!		
REVISIÓN DE SOLDADURA PLACA - VIGA		
Resistencia por metal de soldadura → $\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = 0.75(0.60 F_{EXX}) t_e l_{w2} = 181.23 \text{ kips} = 82.19 \text{ T}$		
Resistencia metal base → fluencia → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 1(0.60 F_y) t_{bw} l_{w2} = 195.30 \text{ kips} = 88.57 \text{ T}$		
Resistencia metal base → ruptura. → $\phi R_n = \phi \cdot F_{nBM} \cdot A_{BM} = 0.75(0.60 F_u) t_{bw} l_{w2} = 190.42 \text{ kips} = 86.36 \text{ T}$		
164.38 T ok!		

- CONEXIÓN DE BASE DE COLUMNA HSS 400x400x12



Analysis	✓	100.0%
Plates	✓	0.0 < 5.0%
Anchors	✓	17.6 < 100%
Welds	✓	0.0 < 100%
Concrete block	✓	59.0 < 100%



Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Anchors	17.6 < 100%	OK
Welds	0.0 < 100%	OK
Concrete block	59.0 < 100%	OK
Buckling	Not calculated	

Plates

Name	F_y [MPa]	Thickness [mm]	Loads	σ_{Ed} [MPa]	ϵ_{pI} [%]	σ_{CEd} [MPa]	Check status
COL-tfl 1	344.7	12.0	LE1	261.2	0.0	0.0	OK
COL-bfl 1	344.7	12.0	LE1	289.3	0.0	0.0	OK
COL-w 1	344.7	12.0	LE1	107.1	0.0	0.0	OK
COL-w 2	344.7	12.0	LE1	281.7	0.0	0.0	OK
BP1	344.7	40.0	LE1	159.7	0.0	0.0	OK

Design data

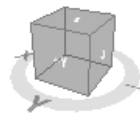
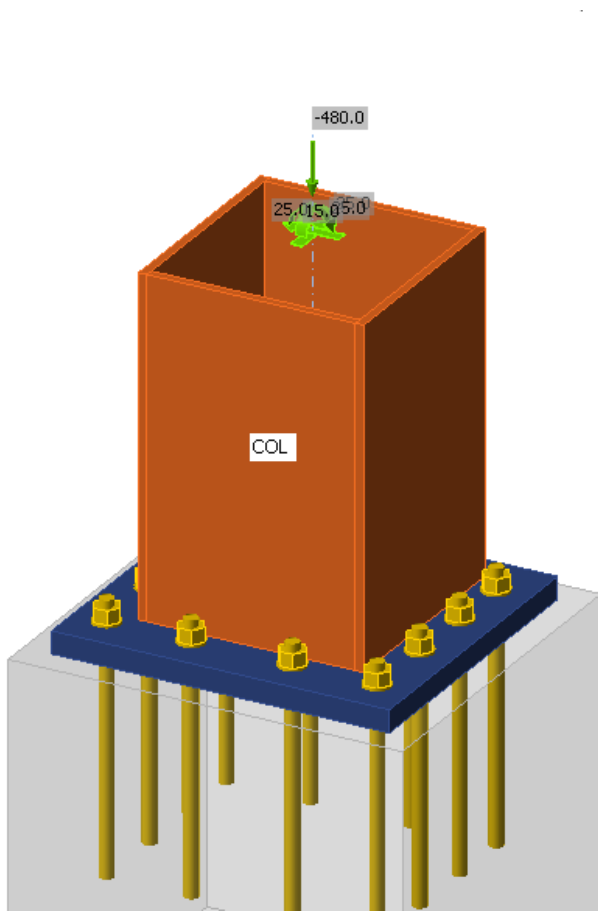
Material	f_y [MPa]	ϵ_{lim} [%]
A572 Gr.50	344.7	5.0



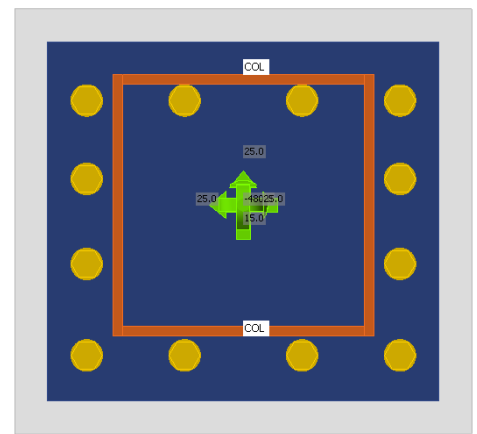
Anchors

Shape	Item	Loads	N_f [kN]	V [kN]	U_{t_t} [%]	U_{t_s} [%]	$U_{t_{ts}}$ [%]	Status
	A17	LE1	0.0	10.2	0.0	13.0	3.3	OK
	A18	LE1	0.0	6.9	0.0	8.7	1.7	OK
	A19	LE1	0.0	4.9	0.0	6.2	1.0	OK
	A20	LE1	0.0	7.3	0.0	9.2	1.9	OK
	A21	LE1	0.0	11.6	0.0	14.7	4.1	OK
	A22	LE1	0.0	5.2	0.0	6.7	1.1	OK
	A23	LE1	0.0	7.1	0.0	9.0	1.8	OK
	A24	LE1	0.0	11.1	0.0	14.0	3.8	OK
	A25	LE1	0.0	11.0	0.0	13.9	3.8	OK
	A26	LE1	0.0	13.8	0.0	17.6	5.5	OK
	A27	LE1	0.0	9.6	0.0	12.2	3.0	OK
	A28	LE1	0.0	10.6	0.0	13.4	3.5	OK

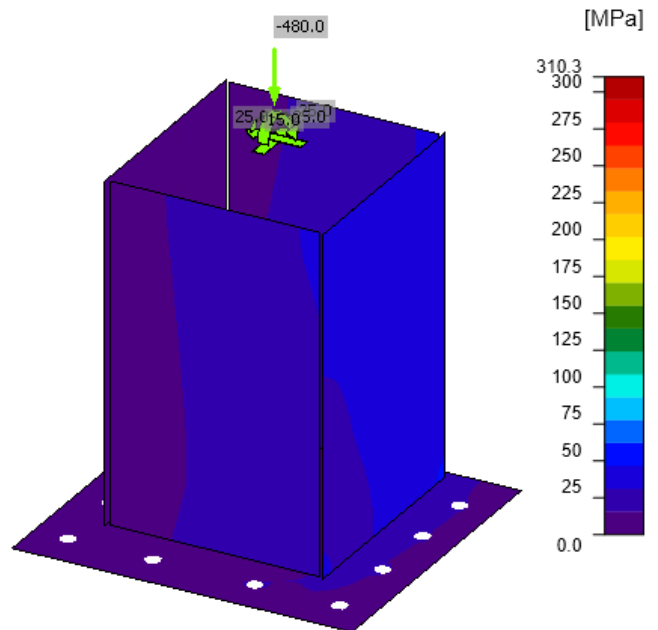
- CONEXIÓN DE BASE DE COLUMNA HSS 350x350x12 – EXCÉNTRICA.



- ▾ Members
 - ✓ COL
- ▾ Load effects
 - ✓ LE1
- ▾ Operations
 - ✓ BP1



Analysis	✓	100.0%
Plates	✓	0.0 < 5.0%
Anchors	✓	4.1 < 100%
Welds	✓	0.0 < 100%
Concrete block	✓	11.0 < 100%
Buckling		Not calculated



Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Anchors	4.1 < 100%	OK
Welds	0.0 < 100%	OK
Concrete block	11.0 < 100%	OK
Buckling	Not calculated	

Plates

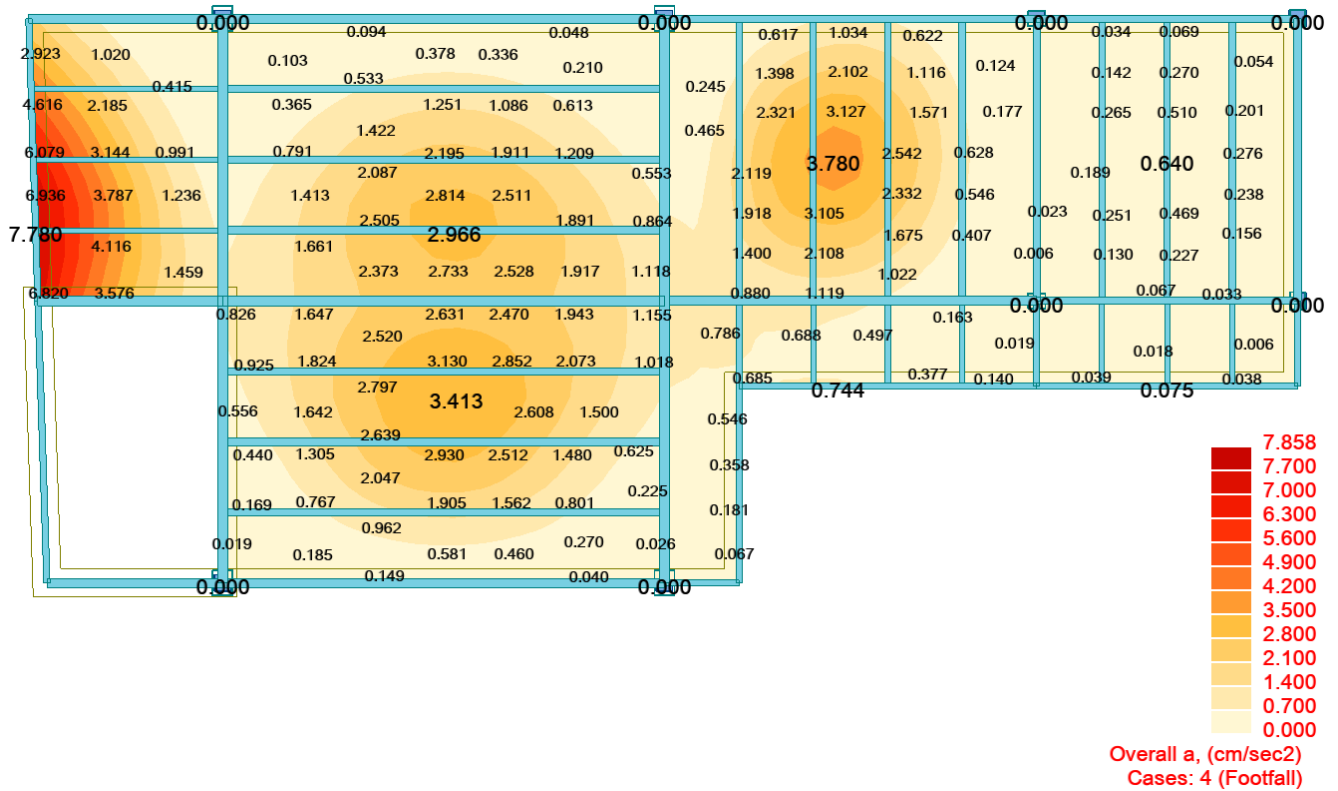
Name	F_y [MPa]	Thickness [mm]	Loads	σ_{Ed} [MPa]	ϵ_{pl} [%]	σ_{CEd} [MPa]	Check status
COL-tfl 1	344.7	15.0	LE1	22.1	0.0	0.0	OK
COL-bfl 1	344.7	15.0	LE1	48.0	0.0	0.0	OK
COL-w 1	344.7	15.0	LE1	32.1	0.0	0.0	OK
COL-w 2	344.7	15.0	LE1	32.2	0.0	0.0	OK
BP1	344.7	40.0	LE1	29.2	0.0	0.0	OK



Anchors

Shape	Item	Loads	N_f [kN]	V [kN]	U_{t_t} [%]	U_{t_s} [%]	$U_{t_{ts}}$ [%]	Status
	A5	LE1	0.0	3.1	0.0	3.9	0.5	OK
	A6	LE1	0.0	2.9	0.0	3.6	0.4	OK
	A7	LE1	0.0	2.8	0.0	3.5	0.4	OK
	A8	LE1	0.0	2.9	0.0	3.6	0.4	OK
	A9	LE1	0.0	3.0	0.0	3.8	0.4	OK
	A10	LE1	0.0	2.8	0.0	3.5	0.4	OK
	A11	LE1	0.0	2.7	0.0	3.4	0.4	OK
	A12	LE1	0.0	3.2	0.0	4.0	0.5	OK
	A13	LE1	0.0	3.1	0.0	4.0	0.5	OK
	A14	LE1	0.0	3.2	0.0	4.1	0.5	OK
	A15	LE1	0.0	3.0	0.0	3.8	0.4	OK
	A16	LE1	0.0	2.8	0.0	3.6	0.4	OK

7.4. Análisis de vibraciones.



Se utilizan los parámetros correspondientes a “Shopping malls”, por ser el que más se ajusta a las características de la terraza mirador.

$$\beta = 0.02.$$

	Constant Force P_o	Damping Ratio β	Acceleration Limit $a_o/g \times 100\%$
Offices, Residences, Churches	0.29 kN (65 lb)	0.02–0.05*	0.5%
Shopping Malls	0.29 kN (65 lb)	0.02	1.5%
Footbridges—Indoor	0.41 kN (92 lb)	0.01	1.5%
Footbridges—Outdoor	0.41 kN (92 lb)	0.01	5.0%

* 0.02 for floors with few non-structural components (ceilings, ducts, partitions, etc.) as can occur in open work areas and churches.
0.03 for floors with non-structural components and furnishings, but with only small demountable partitions, typical of many modular office areas.
0.05 for full height partitions between floors.

$$a_g = 9.81 \text{ m/s}^2 = 981 \text{ cm/s}^2$$

$$\gg \text{Footbridges-Indoor} \gg \frac{a_o}{g} \times 100\% \leq 1.5\%.$$

$$a_o \leq 0.015 \times 981 = 14.72 \text{ cm/s}^2$$

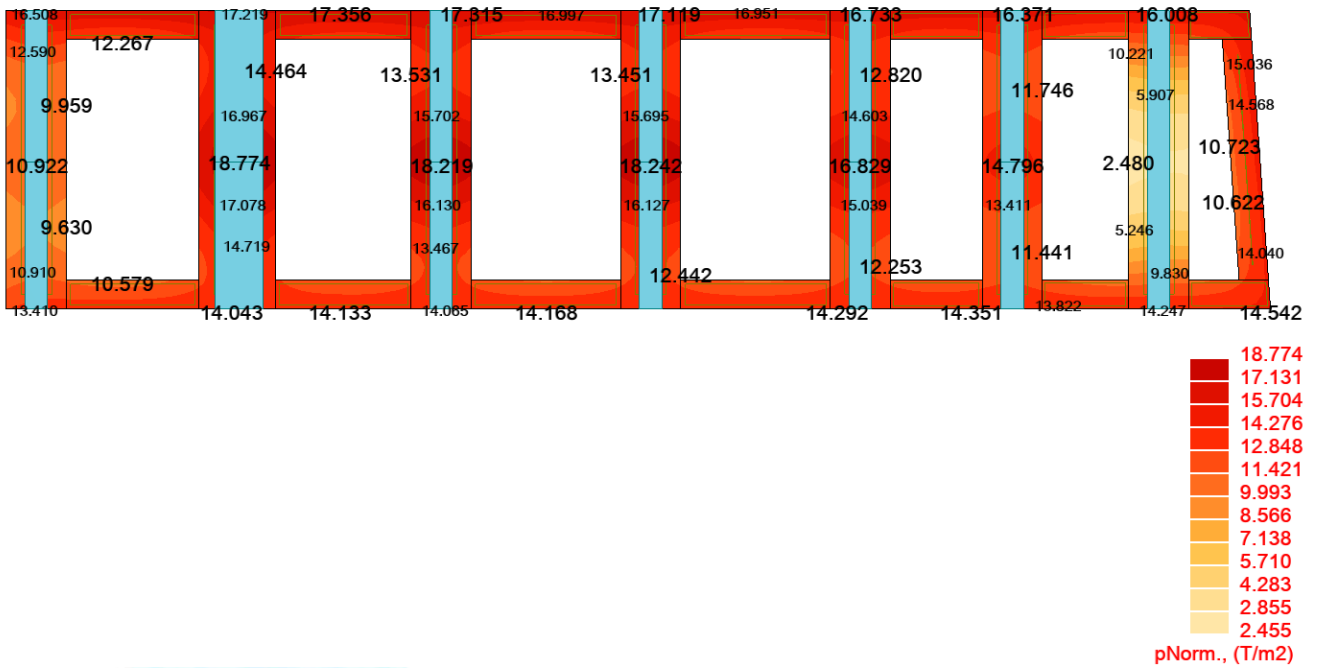
$$a_o = 7.86 \text{ cm/s}^2 < 14.72 \text{ cm/s}^2 \rightarrow \text{ok}$$

7.5. Cimentación.

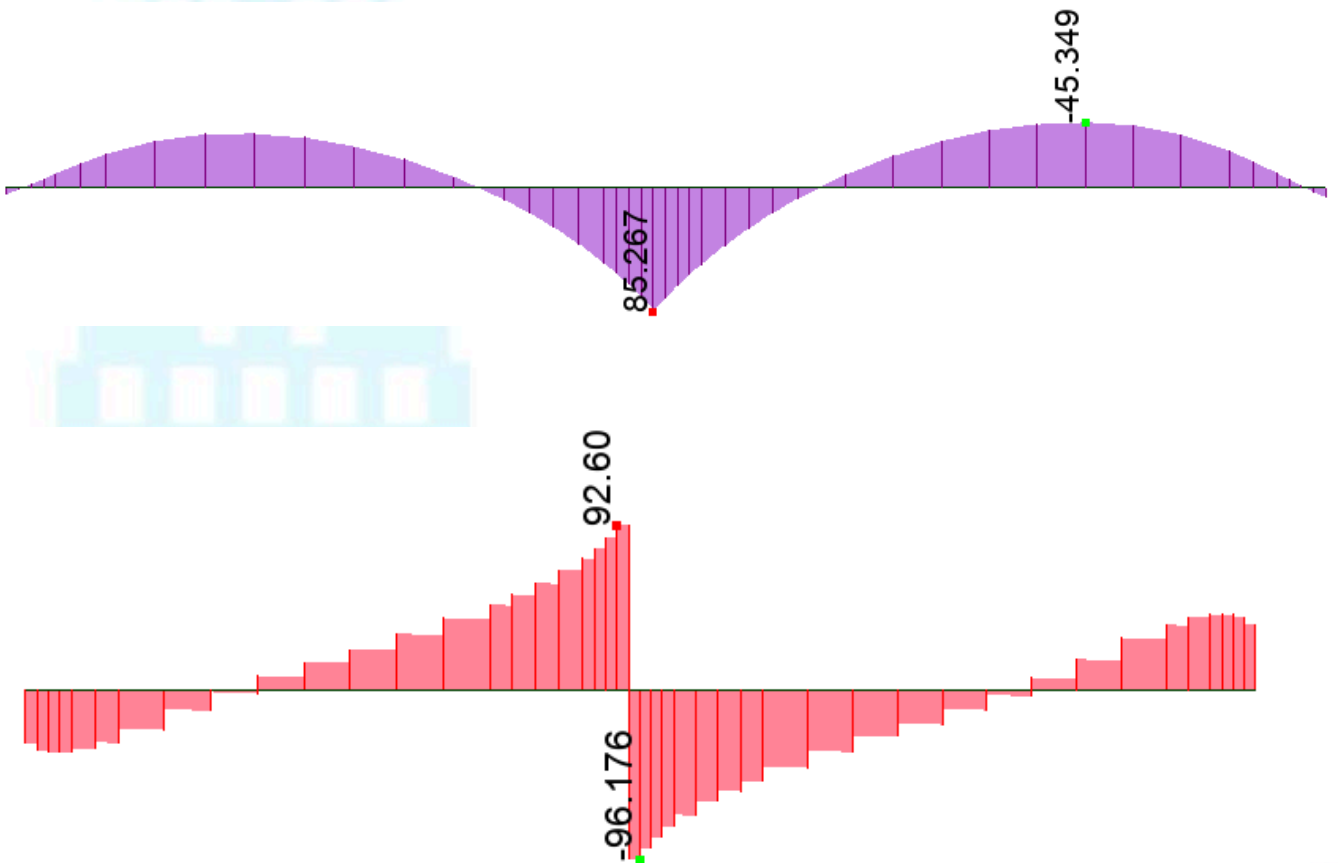
- **Esfuerzos de servicio sobre el suelo:**

$q_a = 25 \text{ T / m}^2$ (Según estudio de suelos).

- ET3.



- **Diseño viga cimentación – Eje M – ET3.**



f_c (Kg/cm ²)	280.00
f_y (Kg/cm ²)	4200.00
H tot (cm)	80.00
bw (cm)	80.00
bf>=bw (cm)	
hf (cm)	

Recub (cm)	7.50
Estrib (mm)	10.00

Ductilidad (3/4 - 1/2 - 3/8) 0.500

ARMAD. TENSION (mm)	1 era capa Núm. Var	2 da capa Núm. Var	3 era capa Núm. Var
8			
10			
12			
14			
16			
18			
20			
22	6	3	
25			
28			
32			

ARMAD. COMPRES. (mm)	1 era capa Núm. Var	2 da capa Núm. Var	3 era capa Núm. Var
8			
10			
12			
14			
16			
18			
20			
22	6	4	
25			
28			
32			

As tot (cm ²)	34.21
d (cm)	68.83
ρ	0.00621

A's tot (cm ²)	38.01
d' (cm)	11.48
ρ'	0.00690

Mu res (tm) 85.87

As min (cm²) 18.49

Se comprueba el diseño adecuado.

REFUERZO TRANSVERSAL ELEMENTOS SOMETIDOS A FLEXIÓN

h = 80 cm

$\phi_L = 22$ mm

r = 7.5 cm

d = 70.4 cm

$\phi_E = 10$ mm

ramas = 4

$f'_c = 280$ kg/cm²

$A_v = 3.14$ cm²

$V_c = 49.95$ T

$b_w = 80$ cm

$f_y = 4200$ kg/cm²

$V_s = 92.89$ T

$\lambda = 1$

$S_{colocado} = 10$ cm

$\phi V_n = 107.13$ T

$\phi V_s = 69.67$ T

$V_n = 142.84$ T

Se comprueba el diseño adecuado.

6.1. Muro de subsuelo.

→ DATOS DE ESTUDIO DE SUELOS – CONSTRUPLOT:

Coefficiente de Balasto (ks) = 5000 T/m³.

Ángulo de fricción (φ) = 25 °

Cohesión (c) = 0.15 kg/cm²

Peso específico del suelo (γ) = 1.70 T/m³

PRESIONES EN MUROS DE SUBSUELO

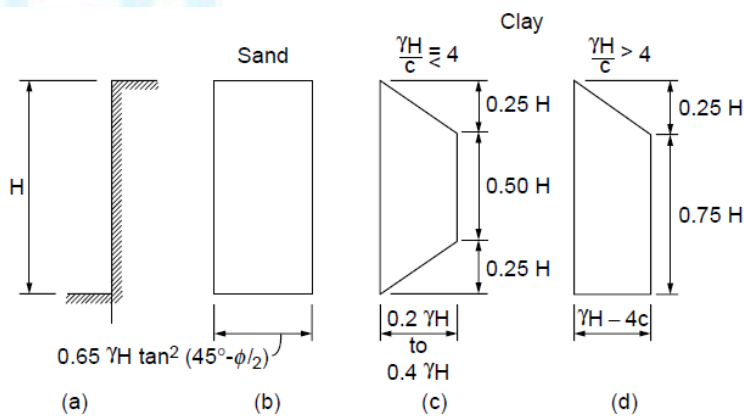


FIGURE 22.8 Lateral pressure distribution for computation of strut loads in braced excavation systems. (After Terzaghi, K., and Peck, R. 1967. *Soil Mechanics in Engineering Practice*. John Wiley & Sons, New York. Copyright © 1967 John Wiley & Sons, Inc. Reprinted by permission of John Wiley & Sons, Inc.)

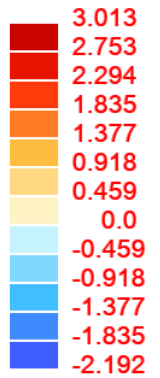
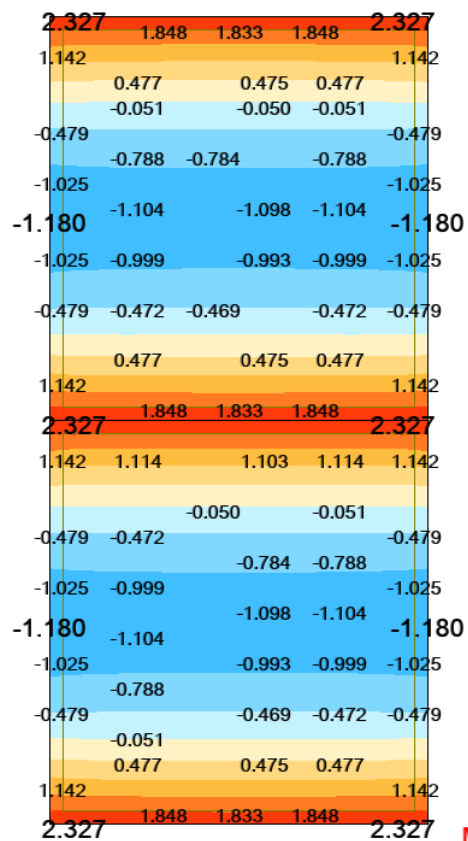
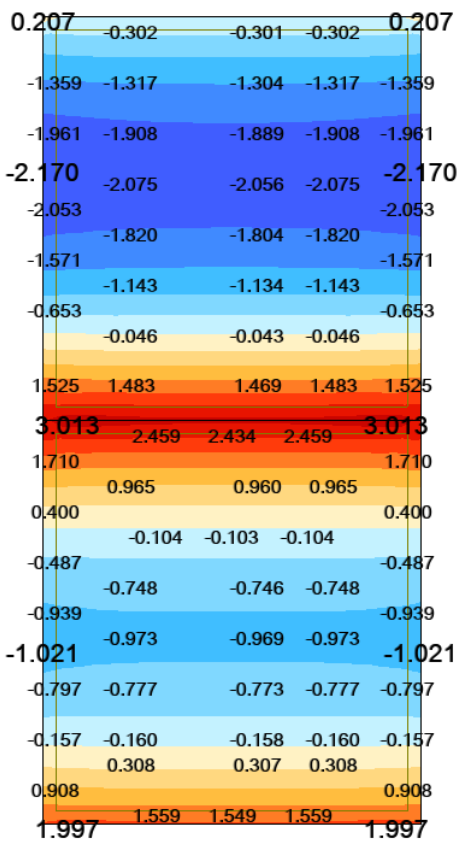
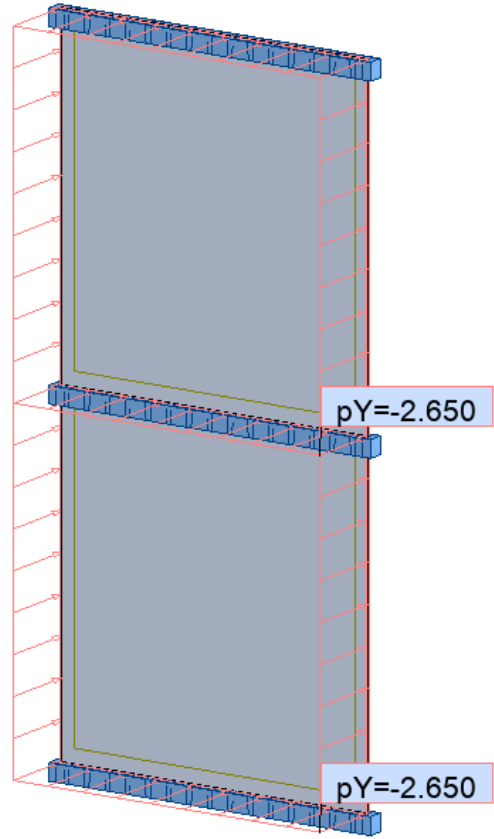
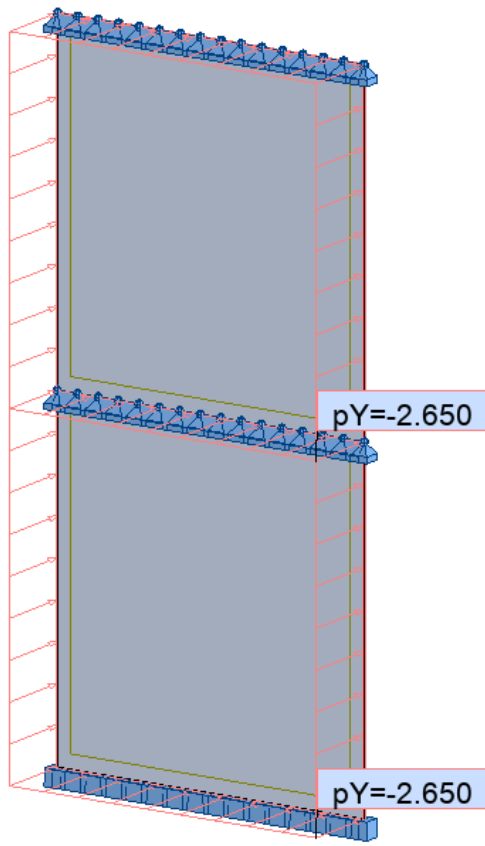
Profundidad de la excavación, H (m)	7.90
Peso volumétrico, γ (t/m ³)	1.70
Ángulo de fricción, φ (grados)	25.00
Cohesión, c (t/m ²)	1.50

Ka 0.41

Arena	Pb (t/m ²)	3.54	
Arcilla	Pc (t/m ²)	5.37	
Arcilla	Pd (t/m ²)	7.43	
Suelo Compuesto	Pd (t/m ²)	1.63	Aplica

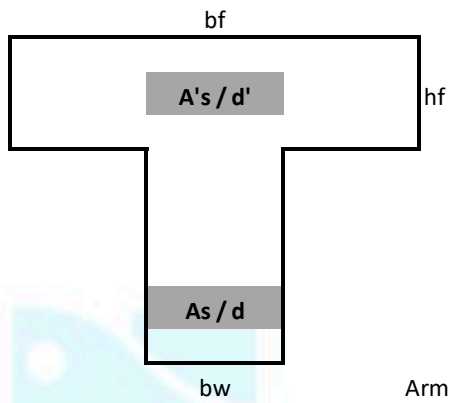
P diseño (t/m²) 1.65 Valor utilizado

1.6p = 2.65 T/m²



MY, (Tm/m)

Resistencia del Hormigón, $f'c$ **280.00** (Kg/cm²) ==> **27.46** (MPa)
 Fluencia del Acero, f_y **4,200.00** (Kg/cm²) ==> **411.88** (MPa)



$A's =$ **0.00** (cm²) ==>

$\rho_{max} =$ **0.50** ρ_{bal}

$A_s =$ **5.65** (cm²) ==>

Armadura Mínima, $A_{smin} =$ **6.53** (cm²) ==>

Ancho del Ala, b_f **100.00** (cm) ==> **39.37** (in)
 Altura del Ala, h_f **0.00** (cm) ==> **0.00** (in)
 Ancho del Alma, b_w **100.00** (cm) ==> **39.37** (in)
 Altura Efectiva, d **19.50** (cm) ==> **7.68** (in)
 Altura Efectiva, d' **5.50** (cm) ==> **2.17** (in)

Momento Ultimo Actuante, M_u **4.06** (t.m) ==> **39.81** (kN.m)

$\Phi 12@200 \rightarrow A_s=5.65\text{cm}^2 / \text{m}$

→ Se comprueba el diseño adecuado.