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Optimization of a Steel Through-Truss Railway Bridge

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|ABSTRACT

This thesis presents an automated workflow in MATLAB to optimize a steel through-truss railway bridge. The objective focuses exclusively on reducing the self-weight of the bridge by decreasing the structural sections of its main members while keeping the overall geometry fixed. The process integrates a parametric model and automated structural analysis within an optimization loop that avoids manual iteration. The constraints consider Ultimate Limit States (ULS) and Serviceability Limit States (SLS), including resistance checking, buckling, serviceability deformations, as well as fatigue criteria for the elements and dynamic behavior (vibrations) under railway loads. The results indicate that it is possible to achieve lighter designs while maintaining safety and operational requirements and reducing the calculation effort and trial-and-error typical of conventional design. Additional potential goals, including direct cost, life cycle cost and maintenance, fabrication and assembly effort, robustness, and vibration comfort, are suggested for future research. The framework is reproducible and adaptable to different spans and operating conditions.

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| INTRODUCTION

Steel through-truss railway bridges remain a competitive solution for medium and long spans, thanks to their good strength-to-weight ratio and the ease of prefabricating and assembling repetitive elements. In Italy, Rete Ferroviaria Italiana (RFI) has a Design Manual for Civil Works (Bridges and Structures section) and annexes with typological designs, where depending on variables such as span, structural typology (U or O), and the number of tracks and others, the use of standardized solutions and profiles is guided. This streamlines repetitive projects but can lead to configurations that are not strictly optimal for specific cases. (condivisionext.rfi.it)

This thesis seeks to address this issue through an optimization workflow based in MATLAB that is exclusively focused on minimizing self-weight through reducing the structural sections of the chords, verticals, and diagonals while maintaining the global geometry of the bridge. The optimization is effectively a fusion of a parametric model and an automated structural analysis inside an optimization loop that takes place without reference to human intervention manually. The checks are executed using SAP2000 against the Ultimate Limit States (ULS) and Serviceability Limit States (SLS) and by considering the following checks: resistance, buckling, deformations, dynamic behavior (frequencies and accelerations during train passage), and fatigue, according to the current railway regulations.

The scope is intentionally restricted to the global member sizing, without a detailed examination of connections, local stiffeners, or construction aspects, and to a single-objective optimization (weight minimization). However, the idea is extensible and could incorporate, in future work, objectives such as direct cost, life cycle cost and maintenance, fabrication and assembly time/complexity, robustness against uncertainties, and vibration comfort, as well as multi-objective formulations.

The main contributions are:

- A reproducible workflow in MATLAB that automates model generation, analysis, and solution search.
- A clear problem formulation with ULS and SLS constraints that include fatigue and dynamic response
- The demonstration in a case study is that it is possible to decrease weight while maintaining regulatory compliance and reducing the iteration effort compared to manual approaches.

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|SYMBOLS AND ABBREVIATIONS

For the purposes of this document a basic list of notations is provided:

L	Length of the bridge
S	Span of the bridge (distance from bearing axes)
T_{ss}	Transverse spacing between stringers
d_g	Height of train gantry
G_w	Width of train gantry
N_{bw}	Minimum free width between train gantry and steel structure
d_{track}	Height of track
g_1	Self-weight
g_2	Track weight
g_3	Noise barrier weight
TermicS	Uniform thermal variation
TermicD	Non uniform thermal variation
v_b	Basic wind velocity
Ecs	Exposition category of the site
b_i	Bridge internal width
b_e	Bridge external width
b_{lt}	Thickness (in transverse direction) of lateral trusses
d_i	Bridge internal height
d_{top}	Bottom deck depth

d_{bot} Top deck depth

d_e External (total) depth of bridge

D/C Demand/Capacity

z height of the highest point of the deck from the average level of the ground surrounding the bridge.

3

|BRIDGE GEOMETRY

The initial geometry is derived from an existing, as-built bridge with a 43 m span. Its overall layout and principal proportions are adopted, including: the truss height, the number and spacing (or rhythm) of the panels, the distance between chords, the effective deck width, and the support scheme. Consequently, the model replicates the arrangement of verticals and diagonals from the reference case, maintaining its global configuration while the base geometry remains unmodified.

To document and reproduce the model according to the reference one, it is also necessary to consider the RFI (Rete Ferroviaria Italiana) recommendations given as minimum values for some geometry variables.

- The transversal spacing between stringers (T_{ss}) is imposed by the RFI, T_{ss}= 1.52 m
- The height of train gantry (d_G) depends on the type of line:
  - ✓ 6.60m for U type bridge and 25000V AC alimentation with inner support to power line
  - ✓ 6.20m for U type bridge and 3000V DC alimentation
  - ✓ 6.85m for O type bridge and 25000V AC alimentation internal to the bridge section
  - ✓ 6.60m for O type bridge and 25000V AC alimentation external to the bridge section
  - ✓ 6.20m for O type bridge and 3000V DC alimentation
- The width of train gantry (Gw) according to the RFI is Gw =4.50 m
- The minimum free width between train gantry and steel structure to be left to install noise barriers (N_{bw}) according to the RFI is N_{bw} = 0.25m.
- The track height (d_{trk}) according to the RFI is, sleepers + rail = 0.30m

The final initial geometry will be set according to these minimum values and with the following formulation:

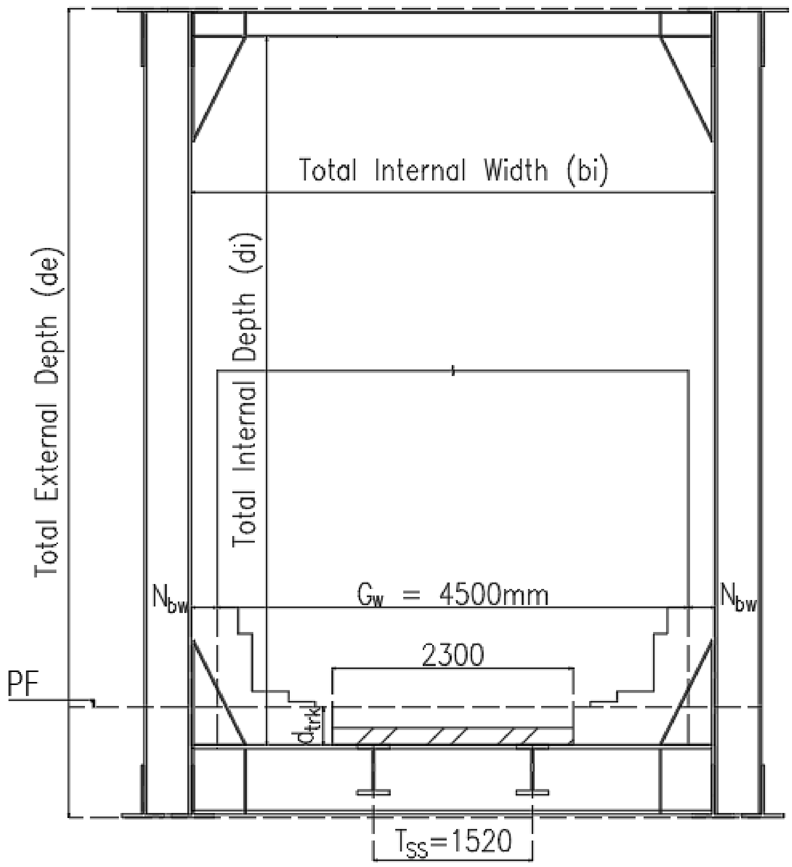


Figure 1. General bridge geometry

- Internal Width of bridge (b_i):

$$b_i > Gw + 2N_{bw} = 4.5 + 2 \cdot 0.25 = 5m$$

- External (total) width of bridge (b_e):

$$b_e = b_i + 2 \cdot b_{lt}$$

- Internal height (d_i) of bridge, this limit applies only for O type

$$d_i > d_G + d_{trk} = d_G + 0.3m$$

- External (total) height of bridge(d_e):

$$d_e = d_i + d_{bot} + d_{top}$$

As previously stated, the analysis maintains the bridge's overall geometry (which is based on the actual case with a 43 m span) while focusing on the sections of the elements that comprise the primary structure of the bridge. In SAP2000, the sections are arranged into families, each of which will contain a unique set of candidate profile

3.1 Member families and section type

Consistent with conventional practice, the design space includes catalogue sections—rolled or extruded (e.g., HEA, HEM, IPE)—as well as non-standard welded built-up elements; both were used in the analysis.

3.1.1 Rolled or extruded sections:

Vertical or Uprights elements (VER):	HE450B
Stringers (LG):	HE450M
Transversal Top beams (TRS):	HE240B
Transversal Stringer beams (R-TR-B):	HE360A
Upper bracing (CTT):	DA125x14
Lower bracing (CTB):	DA180X15
Stringer bracing (ST-CT-B):	DA100x12

**DA = Double Angle sections*

3.1.2 *Welded built-up sections:*

- <u>Bottom chord (BC):</u>

The bottom chord is formed by two single welded T profiles as shown in Figure 2.

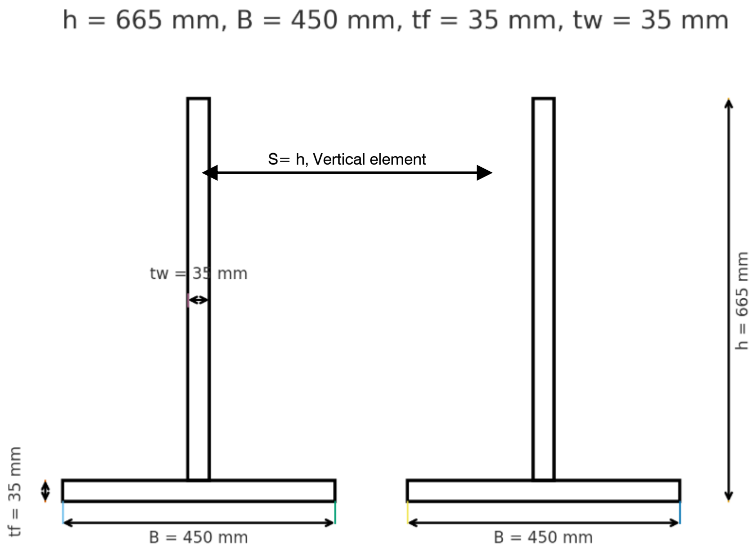


Figure 2. Bottom chord geometry

- <u>Top Chord (TC):</u>

The Top chord is formed by two single welded T profiles, as is shown in Figure 3.

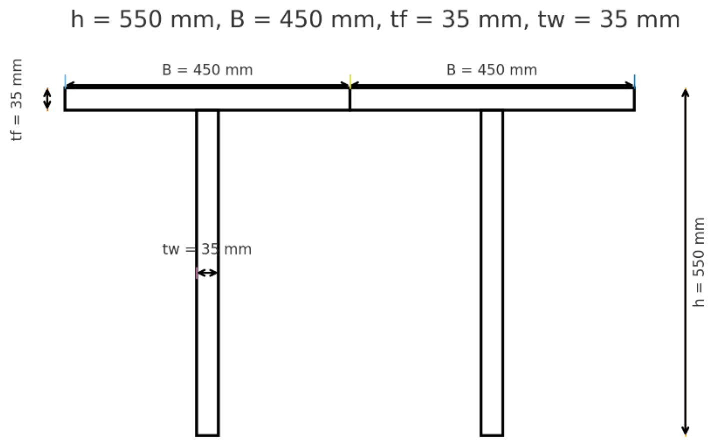


Figure 3. Top chord geometry

- <u>Diagonal elements (DGA, DGB):</u>

The Diagonal elements are a welded built-up I/Wide flange profile, as is shown in Figure 4 and Figure 5

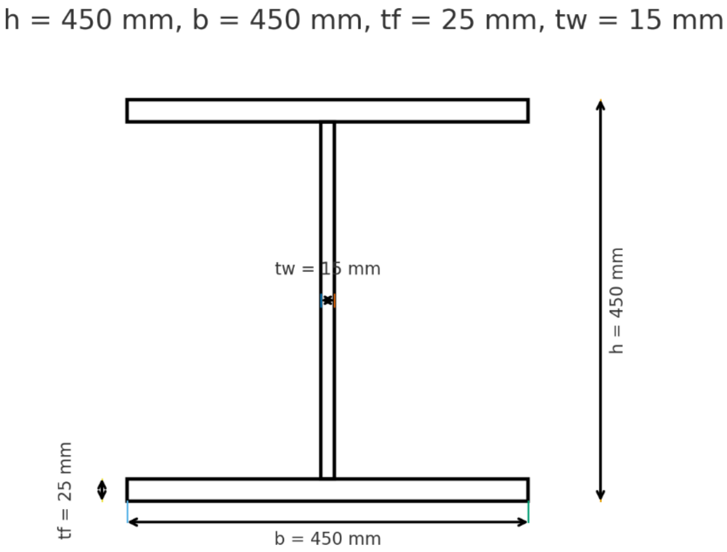


Figure 4.Diagonal DGA geometry

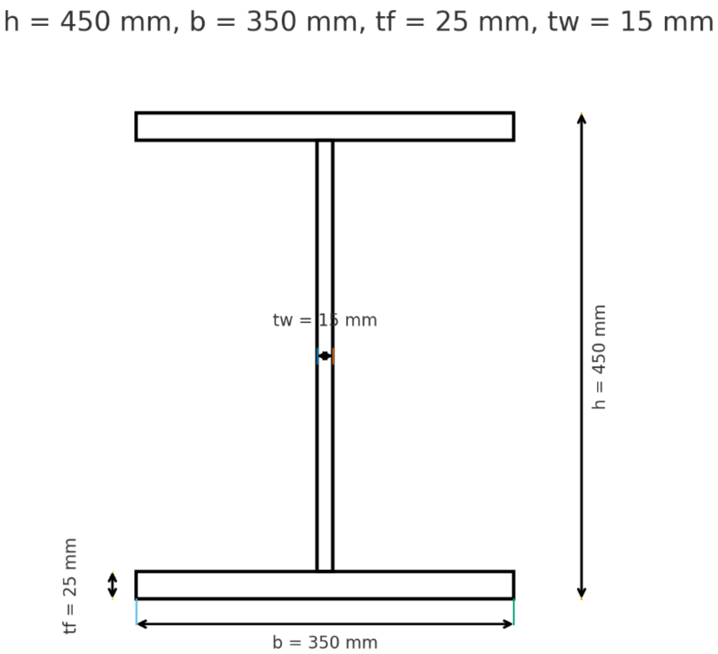


Figure 5. Diagonal DGB geometry

- Bottom transversal elements (TRB):

The bottom transversal elements are a welded profile given by the following geometry

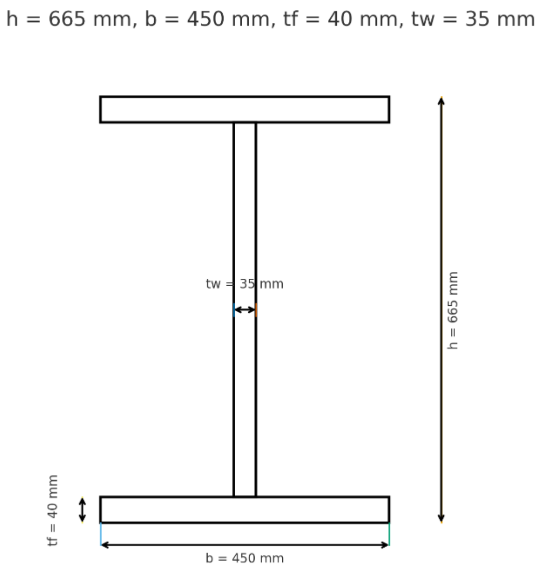


Figure 6. Bottom transversal beam section

4

|ACTIONS AND LOAD DISTRIBUTION IN SAP2000

This chapter presents the load definition and its distribution path within the SAP2000 model. It starts with a clear separation between permanent loads (self-weight, deck, track/ballast, and accessories) and variable railway loads (design trains and associated effects), in addition to complementary actions like longitudinal breaking/acceleration, temperature, and wind.

Each group is defined in distinct Load Patterns and combined in Load Cases and in chapter 7 in Load Combinations for ULS (Ultimate Limit State) and SLS (Serviceability Limit State).

4.1 PERMANENT ACTIONS

4.1.1 G1 STRUCTURAL SELF-WEIGHT

The self-weight of the structure must be evaluated based on the geometric characteristics of the elements constituting it and the specific weights of the different materials.

Structural self-weight is automatically calculated by the software. This load is increased by a coefficient $\delta_{G1} = 1.15$, to consider the weights of the connections (plates, bolts, welding, etc..) and the weight of zinc coating and paint.

4.1.2 G2 WEIGHT OF THE TRACK

The track will be composed by wooden elements called "sleepers", the dimensions of the sleepers according to the "**ALL.C : DISEGNI TIPOLOGICI E PARTICOLARI COSTRUTTIVI**"**[1]** are 2300 x 160 x 220 mm, each element has a weight of 90 kg.

According to the "**MANUALE DI PROGETTAZIONE DELLE OPERE CIVILI PARTE II - SEZIONE 2 PONTI E STRUTTURE**"Error! Reference source not found.**,** the maximum distance between sleepers will be 600 mm.

Over these sleepers will be placed the rail profiles, the properties of these elements are given in Table 1, according to EN 13674-1.

Profile	**Cross-sectional area (cm^2)**	**Mass per meter (kg/m)**
46 E 1	58.52	46.17
46 E 2	58.94	46.27
46 E 3	59.44	46.66
46 E 4	59.78	46.9
49 E 1	62.92	49.39
49 E 2	62.55	49.1
49 E 3	62.55	49.1
49 E 4	62.55	49.1
50 E 1	64.16	50.37
50 E 2	63.65	49.97
50 E 3	63.71	50.02
50 E 4	63.71	50.9
50 E 5	63.71	50.9
50 E 6	64.84	50.9
52 E 1	64.84	50.9
54 E 1	69.77	54.77
54 E 2	68.56	53.82
54 E 3	64.84	54.57
55 E 1	64.84	54.57
56 E 1	71.69	56.3
60 E 1	76.48	60.21

Table 1. Rail cross-sections

This G2 will be loaded as a uniform load on the stringers, according to the following expression (1):

$$G_2 = \delta_{g2}\left(\frac{SW}{S} + RPW\right) \tag{1}$$

This load will be increased by a coefficient $\delta_{G2} = 1.05$, to consider the weights of the connections.

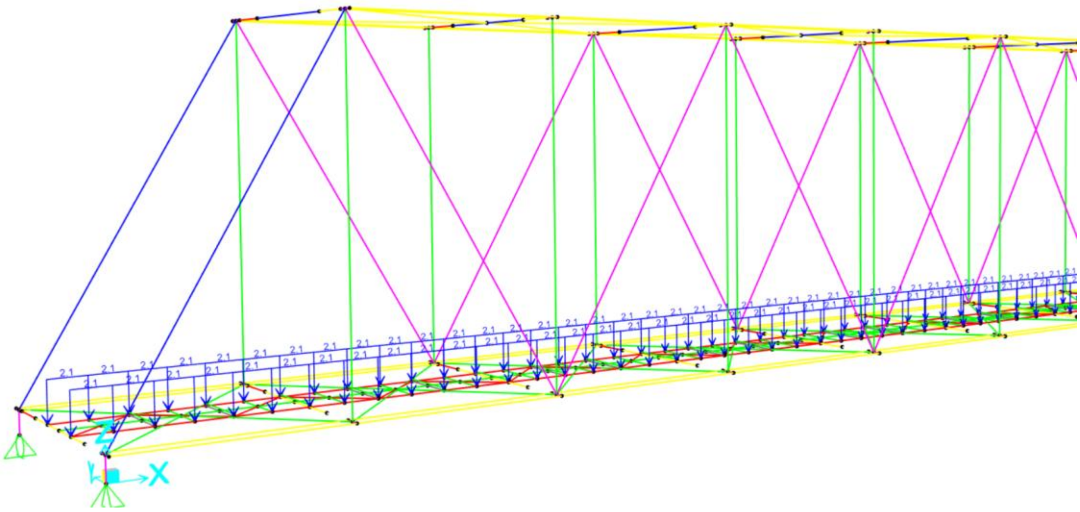


Figure 7.Application of g2 load on the 3D model

4.1.3 G3 WEIGHT OF THE NOISE BARRIERS

According to paragraph 5.2.2.1.1 of the DM 17.01.2018**Error! Reference source not found.**, When designing new railway bridges, the weights, actions and encumbrances associated with the introduction of noise barriers must always be considered, even in cases where the construction of this type of element was not originally planned.

From paragraph 2.5.1.3.2 of the MANUALE DI PROGETTAZIONE DELLE OPERE CIVILI PARTE II - SEZIONE 2 PONTI E STRUTTURE”**Error! Reference source not found.** it is going to be assumed a value not less than 3.2 kN/m^2 and a height of the same of 4 m measured from the external level of the slab.

$$G_3 = 3.2\frac{kN}{m^2}$$

This weight will be applied on the bottom chord of the side trusses of the deck as shown in Figure 8.

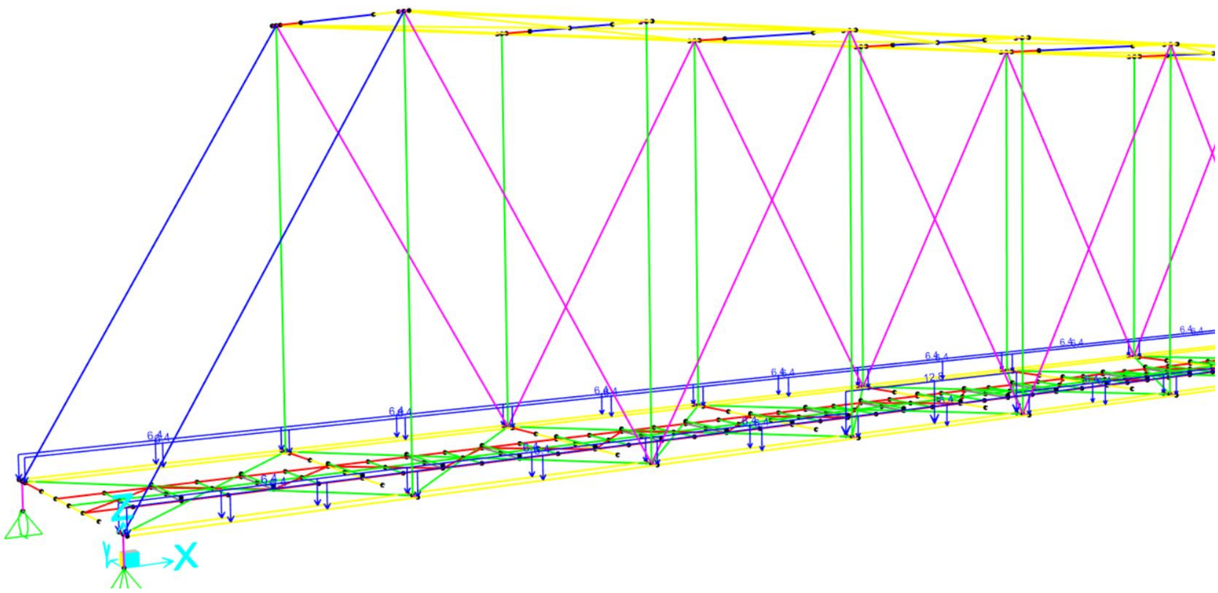


Figure 8. Application of g3 load on the 3D model

4.1.4 G4 WEIGHT OF THE WALKABLE DECK

For this case, a uniform distributed permanent load g_4 equal to 70 kg/m^2 will be assumed and introduced to 3D model to consider the weight of the walkable metal grids which are fixed to the transverse beams and to the sleepers to allow for inspections and maintenance of the deck and the railway material.

This load will be assigned over the bottom chord as it is shown in Figure 9.

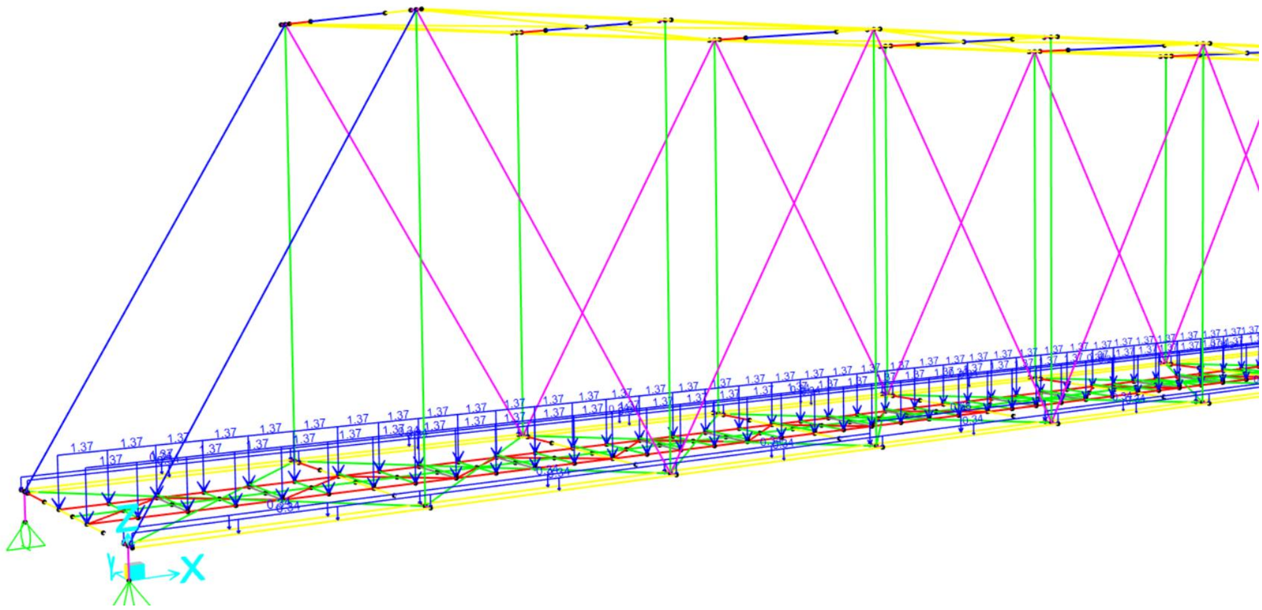


Figure 9 Application of g4 load on the 3D model

4.2 THERMAL ACTIONS

According to 2.5.1.4.4.1**Error! Reference source not found.**, there will be two types of thermal actions:

- Uniform thermal variation
- Non-uniform thermal variation

4.2.1 UNIFORM THERMAL ACTION (TS, SEASONAL)

Based on 2.5.1.4.4.1**Error! Reference source not found.**, In the absence of in-depth studies, the uniform thermal variations to be considered for works directly exposed to atmospheric actions, with respect to the average temperature of the site, are to be assumed to be equal to:

- Deck with steel structures and direct armature ΔT = ± 25° C (see Figure 10)

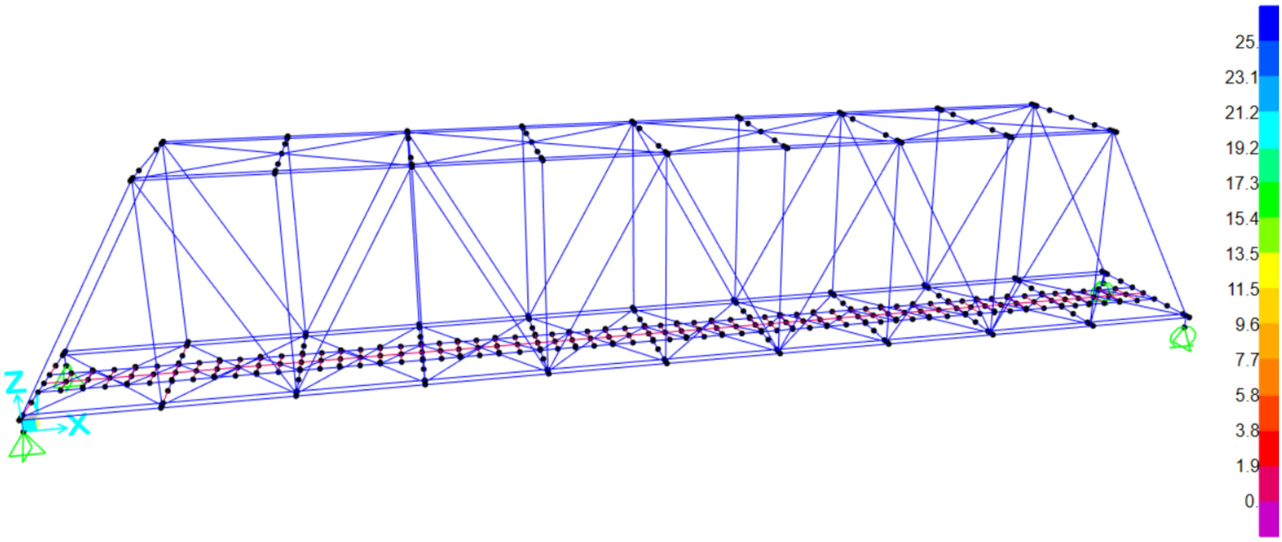


Figure 10 Application of thermal (C) action on the 3D model°

4.2.2 NON-UNIFORM THERMAL ACTION, (TD, DAILY)

Based on 2.5.1.4.4.1, in addition to the uniform thermal variation, a temperature gradient of 5 °C must be considered between the external and internal of the deck, with the direction to be determined on a case-by-case basis.

This load will be applied on the bottom and top part of the bridge according to the correspondent depth (see Figure 11)

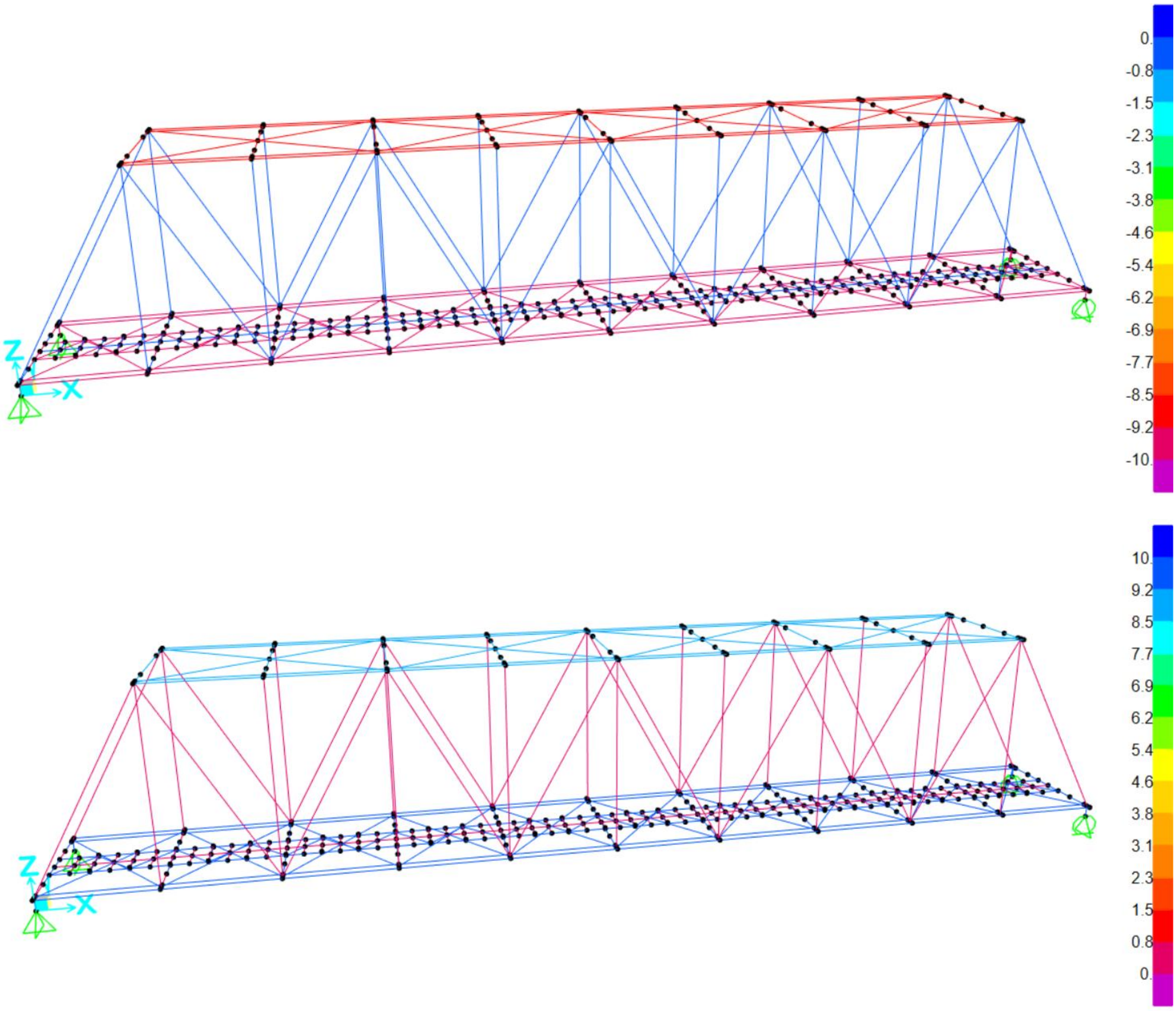


Figure 11.Application of daily thermal action(°C/m) on the 3D model

4.3 WIND ACTION

4.3.1 WIND VELOCITY

4.3.1.1 BASIC WIND VELOCITY

To determine the basic wind velocity, it will be used chapter 3.3.1 of the NTC 2018**Error! Reference source not found.**

It is necessary to define the Location of the bridge, for this case it will be assumed that is located in **TURIN**, according to this and with table 3.3.I from the NTC 2018**Error! Reference source not found.**, we get the values of V_b, a_0 and k_s.(see Table 2)

Zone	Description	$v_{b,0}$ (m/s)	a_0 (m)	k_s
1	Valle d'Aosta, Piemonte, Lombardia, Trentino Alto Adige,Veneto, Friuli-Venezia Giulia (con l'eccezione della pro-vincia di Trieste)	25	1000	0.4
2	Emilia Romagna	25	750	0.45
3	Toscana, Marche, Umbria, Lazio, Abruzzo, Molise, Puglia,Campania, Basilicata, Calabria (esclusa la provincia di Reggio Calabria)	27	500	0.37
4	Sicilia e provincia di Reggio Calabria	28	500	0.36
5	Sardegna (zona a oriente della retta congiungente Capo Teulada con l'Isola di Maddalena	28	750	0.4
6	Sardegna (zona a occidente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	500	0.36
7	Liguria	28	1000	0.54
8	Provincia di Trieste	30	1500	0.5
9	Isole (con l'eccezione di Sicilia e Sardegna) e mare aperto	31	500	0.32

Table 2.- Parameters values $v_{b,0}$, a_0, k_s

4.3.1.2 SOIL ROUGHNESS AND EXPOSURE CLASS

For this case, it will be assumed that the bridge is in a suburban or industrial area, according to this and with Table 3.3.III from the NTC 2018**Error! Reference source not found.**, it is selected a roughness type "B".

Classe di rugosità del terreno	Description
A	Aree urbane in cui almeno il 15% della superficie sia coperto da edifici la cui altezza media superi i 15 m
B	Aree urbane (non di classe A), suburbane, industriali e boschive
C	Aree con ostacoli diffusi (alberi, case, muri, recinzioni…); aree con rugosità non riconducibile alle classi A, B, D
D	a) Mare e relativa fascia costiera (entro 2 km dalla costa); b) Lago (con larghezza massima pari ad almeno 1 km) e relativa fascia costiera (entro 1 km dalla costa) c) Aree prive di ostacoli o con al più rari ostacoli isolati (aperta campagna, aeroporti, aree agricole, pascoli, zone paludose o sabbiose, superfici innevate o ghiacciate…

Table 3.Soil roughness class

Now, knowing that Turin is defined a Zone 1 and with a Soil Roughness "B" it is possible to determine the values for K_r,Z_0 and Z_{min} from Table 3.3. II**Error! Reference source not found.**.

Categoria di esposizione del sito	K_r	z_0 (m)	z_{min} (m)
I	0.17	0.01	2
II	0.19	0.05	4
III	0.2	0.1	5
IV	0.22	0.3	8
V	0.23	0.7	12

Table 4. Parameters for defining the exposure coefficient

In this case it will be used the values for an exposure class "IV".

4.3.1.3 TOPOGRAPHIC FACTOR

According to chapter 3.3.7**Error! Reference source not found.**,the topography coefficient c_t is generally set equal to 1, both for flat areas and for undulating, hilly and mountainous areas.

$$C_t = 1.0$$

4.3.1.4 EXPOSURE FACTOR

According to chapter 3.3.7**Error! Reference source not found.**, the exposure coefficient c_e depends on the height z above the ground of the point considered, the topography of the land and the exposure category of the site where the building is located. In the absence of specific analyses that consider the direction of the wind and the actual roughness and topography of the land surrounding the building, for heights above the ground not greater than z = 200 m, it is given by the formula:

$$c_e(z) = {k_r}^2 \cdot c_t \cdot ln\left(\frac{z}{z_0}\right)\left[7 + c_t \cdot ln\left(\frac{z}{z_0}\right)\right] \qquad if\ z \geq z_0 \tag{2}$$

$$c_e(z) = c_e(z_{min}) \qquad if\ z < z_{min} \tag{3}$$

Where:

- Z=The distance of the considered point from the ground
- C_t=The orography factor
- k_r , z_0, z_{min} , The exposure class parameters
-

4.3.1.5 BASIC VELOCITY PRESSURE:

According to EN 1991-1-4 [2], The basic velocity pressure (q_b) is the pressure corresponding to the wind momentum determined at the basic wind velocity v_b.

The basic velocity pressure is given by EN1991-1-4 §4.5(1)[2], or in paragraph 3.3.6**Error! Reference source not found.**,

$$q_r = \frac{\rho \cdot v_b^2}{2} \tag{4}$$

Where:

ρ, Air density ρ = 1.25 kg/m^3, given by EN1991-1-4 §4.5(1) [2].

4.3.2 RESULTS:

According to the procedure explained in 4.3.1 it will be shown the data considered and the results obtained:

Table 5. Bridge general parameters

Bridge Length	L	[m]	43
Type of line			0 Bridge x 25000V AC (Internal)
Gantry height (Recommended)	d_g	[m]	6.85
Gantry height	d_g	[m]	6.85
Width of train gantry	G_W	[m]	4.5
Free width	N_{bw}	[m]	0.25
Track height	$d_{L,Track}$	[m]	0.36
Total Bridge height	h_T	[m]	7.73

In Table 5 it is resumed the general data of the bridge as, the bridge length, data based on the type of line as the gantry height, and also other important data defined by the ALL.C : DISEGNI TIPOLOGICI E PARTICOLARI COSTRUTTIVI" [1] as the width of train gantry and the free width between the train and the structure. All this data is important to define a preliminary geometry of the bridge.

Following the procedure indicated in 4.3.1 it is necessary to define the Input data as the Location of the bridge, the zone (see Table 2), the soil category and the roughness factor, the data used is showed in the following Table 6.

Table 6. Input data

Location			Turin
Zone			1
Soil category			B
Roughness factor	$C_r(z)$	[-]	IV
Altitude above sea level	a_s	[m]	50

With this information it is possible to define the fundamental basic wind velocity and then the basic velocity pressure q_r given in (4).

Table 7. Basic velocity pressure q_b

Fundamental basic wind velocity			
	$v_{b,0}$	[m/s]	25
	a_0	[m]	1000
	k_s	[-]	0.4
Altitude factor	C_{alt}	[-]	1
Basic wind velocity	v_b	[m/s]	25
Period	T_R	[-]	50
	C_r	[-]	1
Reference wind velocity	v_r	[m/s]	25
Wind density	ρ	[kg/m^3]	1.25
Basic velocity pressure	q_b	[kN/m^2]	0.39

Finally, the value according to the procedure in 4.3.1 for the basic velocity pressure is 0.39 kN/m^2.

4.4 WIND PRESSURE:

According to paragraph 3.3.4**Error! Reference source not found.**, the wind pressure $p(z)$ can be calculated as:

$$p(z) = q_r \cdot c_e(z) \cdot c_p \cdot c_d \qquad (5)$$

Where:

- q_r is the basic velocity pressure
- c_e is the exposure coefficient
- c_p is the pressure coefficient
- c_d is the dynamic coefficient that is set equal to 1 as specified in paragraph 3.3.9**Error! Reference source not found.**

4.4.1.1 AERODYNAMC COEFFICIENTS:

According to paragraph 3.3.8**Error! Reference source not found.**, The pressure coefficient c_p depends on the type and geometry of the construction and its orientation with respect to the wind direction.

The friction coefficient c_f depends on the roughness of the surface on which the wind exerts tangent action.

Both these coefficients, defined as aerodynamic coefficients, can be obtained from data supported by appropriate documentation or from experimental tests in a wind tunnel.

4.4.1.1.1 Transversal wind direction (X-direction)

According to Section 8[2], it is required to consider the wind actions on the two main directions, transverse to the bridge deck and parallel to the longitudinal axis.

The chapter defines the following axis, according to Figure 12.

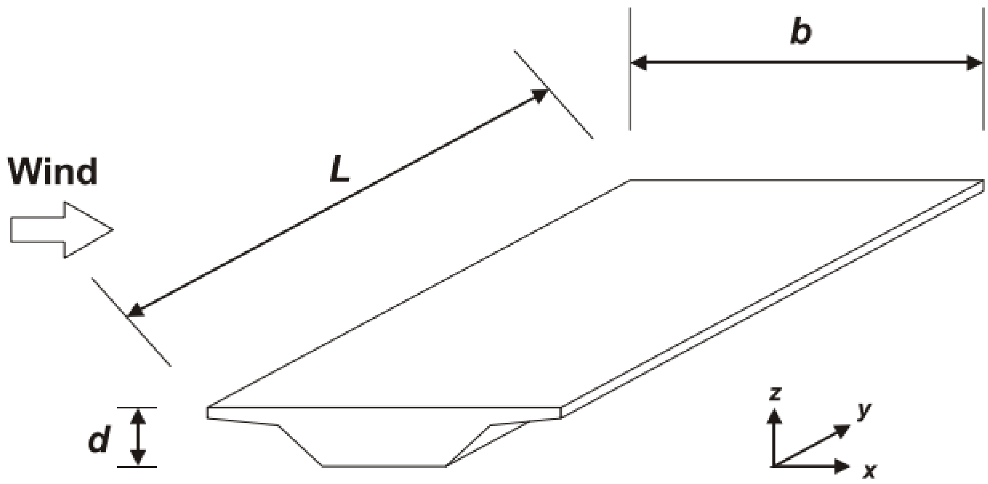


Figure 12 Directions of wind actions on bridge.

Where:

- X- direction, is the direction parallel to the deck width, perpendicular to the span (b, width)
- Y direction is the direction along the span (L, length)
- Z direction is the direction perpendicular to de deck (d, depth)

According to 8.1[2], The forces produced in the X direction are due to wind blowing in different directions and normally are not simultaneous. The forces produced in the Z direction can result from the wind blowing in a wide range of directions- if they are unfavorable and significant, they should be considered as simultaneous with the forces produced in any other direction.

As in this chapter we are looking for the forces acting on the transversal direction (X-direction) it would be necessary to define the Total deck width of the bridge (b_e) also called external width.

This is given by (6):

$$b = b_e = b_i + 2 \cdot b_{lt} \tag{6}$$

Where:

- b_i: Internal width
- b_{lt}: thickness in transverse direction of lateral trusses

In the same way it is necessary to determine the depth of the deck according to the following expression:

$$d = d_e = d_i + d_{trk} + d_{bot} + d_{top} \tag{7}$$

Where:

- d_i: Internal depth
- d_{track}:Track depth, according to (3-|BRIDGE GEOMETRY)
- d_{bot}:Bottom chord depth
- d_{top}:Top chord dep

It is possible to calculate the effect of wind in transverse direction through a coefficient $C_{fx,0}$ given by 8.3.1[2], where $C_{fx,0}$ is the force coefficient without free-end flow.

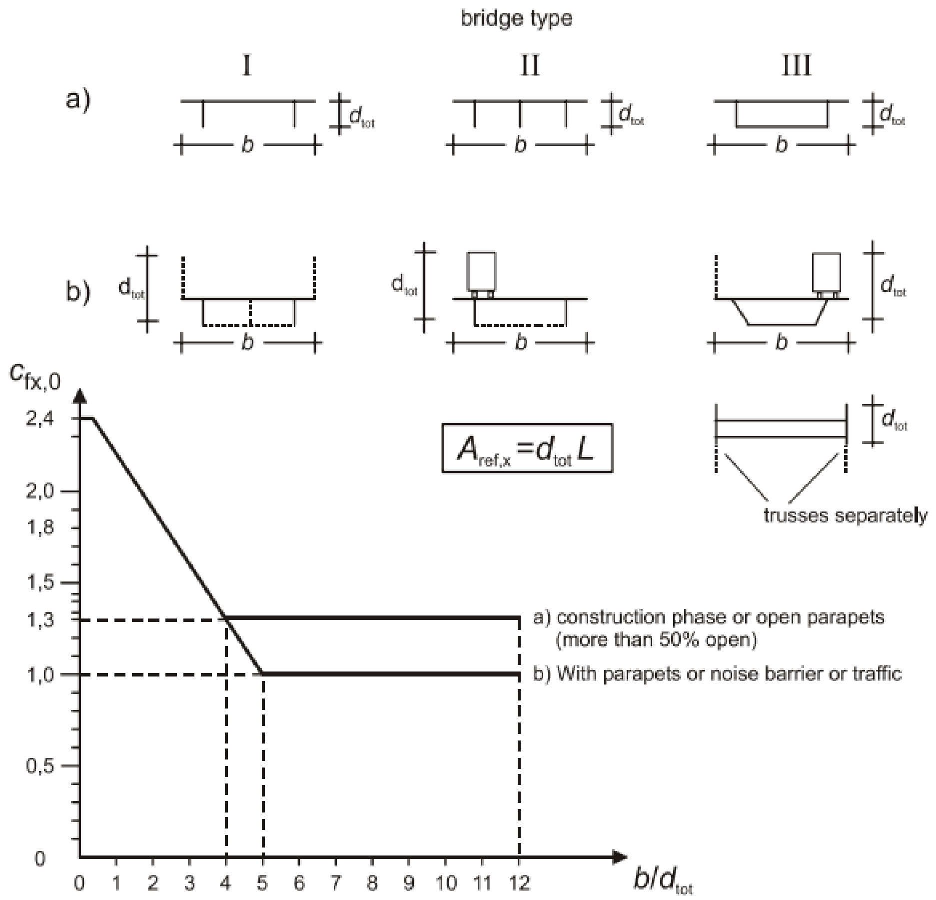


Figure 13. Force coefficients for bridges, $C_{fx,0}$

4.4.1.1.2 **Vertical wind direction (Z-direction):**

Transverse wind also generates a simultaneous vertical action that can be either upwards or downward (in two separate load cases). This action is described in paragraph 8.3.3 [2], force coefficients $c_{f,z}$ (lift force coefficient) should be defined for wind action on the bridge decks in the z-direction.

According to the NOTE 1 in 8.3.3 [2], it is possible to define $c_{f,z} = \pm 0.9$, this value takes globally into account the influence of a possible transverse slope of the deck, of the slope of terrain and of fluctuations of the angle of the wind direction with the deck due to turbulence.

As an alternative it is possible to calculate $c_{f,z}$ from Figure 8.6 in chapter 8.3.3 [2]

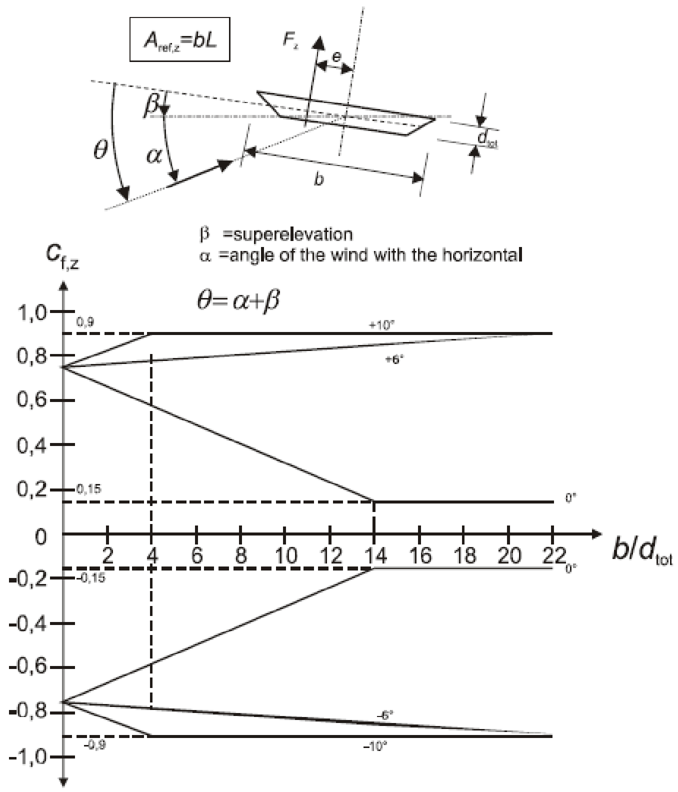


Figure 8.6 — Force coefficient $c_{f,z}$ for bridges with transversal slope and wind inclination

Figure 14.Alternative to calculate $C_{f,z}$ according to 8.3.3[2]

However, to simplify the analysis it will be considered NOTE 1 and the value of the coefficient $c_{f,z}$ will be assumed as $c_{f,z} = \pm 0.9$.

It is also important to consider NOTE 2 in 8.3.3 [2], this force may have significant effects only if the force is of the same order as the dead load.

The wind action in vertical direction is not uniform in transverse direction, according to 8.3.3(5) [2], the eccentricity of the force in the X-direction may be set to e= b/4, with b = b_e(6).

4.4.2 RESULTS:

According to equation (5), to determine the wind pressure, it is necessary to define the values of the following coefficients:

- Exposure coefficient, c_e given by 4.3.1.4
- Pressure coefficient, $c_p = c_{f,x}$ given by 4.5
- Dynamic coefficient, $c_d = 1$

The results are shown in the following Table 8.

Table 8. Wind pressure

Distance "Z" from the ground	z_g	[m]	0.00
	k_r	[-]	0.220
	z_0	[m]	0.300
	z_{min}	[m]	8.000
Total height	z_T	[m]	6.85
Topographic Coeffcient	c_T	[-]	1.00
Exposure factor	c_e	[-]	1.63
Dynamic factor	C_d	[-]	1.00
Wind pressure	p_z	[kN/m^2]	0.64

The value of the wind pressure, p(z)= 0.64 kN/m^2, however this value must be affected by the pressure coefficient given in 4.5.

4.5 WIND CASES:

As mentioned in the previous chapter the wind load will be applied in both directions' transversal and vertical, these should be considered as simultaneous with the forces produced in any other direction.

4.5.1.1 HORIZONTAL WIND:

The horizontal forces applied on the bridge will be divided into 4 cases according to Table 9, these will depend on if there is traffic and if there is a noise barrier.

Table 9.Horizontal wind cases

	WITHOUT NOISE BARRIER	WITH NOISE BARRIER
TRAFFIC	CASE 1	CASE 2
NO TRAFFIC	CASE 3	CASE 4

According to these 4 cases and based on the graph showed in Figure 13, it will be possible to define the different values of b(width) and d (depth) in order to determine the coefficient C_{fx} for each case.

As for CASE 1, CASE 2 and CASE 4, it is defined that there will be presence of traffic, parapet or noise barrier, the values of *b* and *d* will be given by Figure 15:

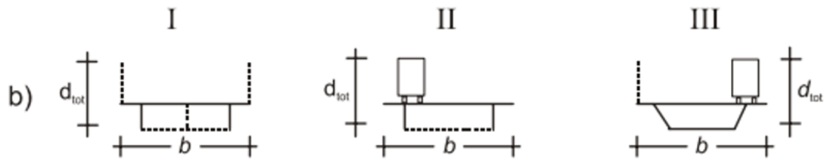


Figure 15.Values of b and d, for CASE 1, CASE 2 and CASE 4

While for CASE 3, the values of *b* and *d* will be given by Figure 16

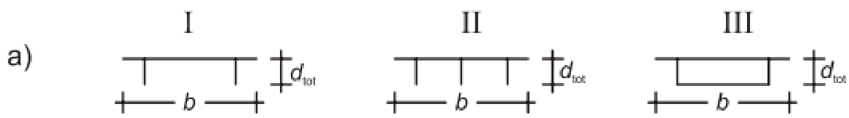


Figure 16.Values of b and d, for CASE 3

4.5.1.1.1 <u>CASE 1, TRAIN, NO NOISE BARRIER:</u>

As it was explained before, for CASE 1 the wind forces and the coefficient C_{fx} will be calculated following the condition, "There is traffic (Train) and there is not any noise barrier" (see Figure 18 and Figure 18).

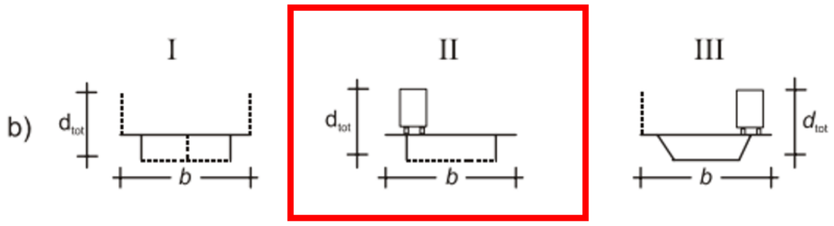


Figure 17.Values of b and d, for CASE 1

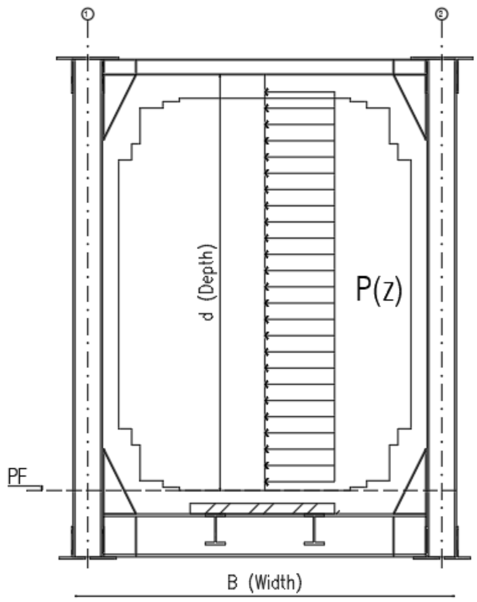


Figure 18. Geometry Case 1. Traffic without Noise barrier

For this case the values for *b* and *d* will be taken as the ones in Figure 18, where *b* will be the total width of the "deck" while *d* will be the total gantry depth.

For case 1, it will be assumed that the wind forces will be applying directly on the Train height, because of this wind pressure (P(z)) there will be different forces acting on the bridge structure (see Figure 19).

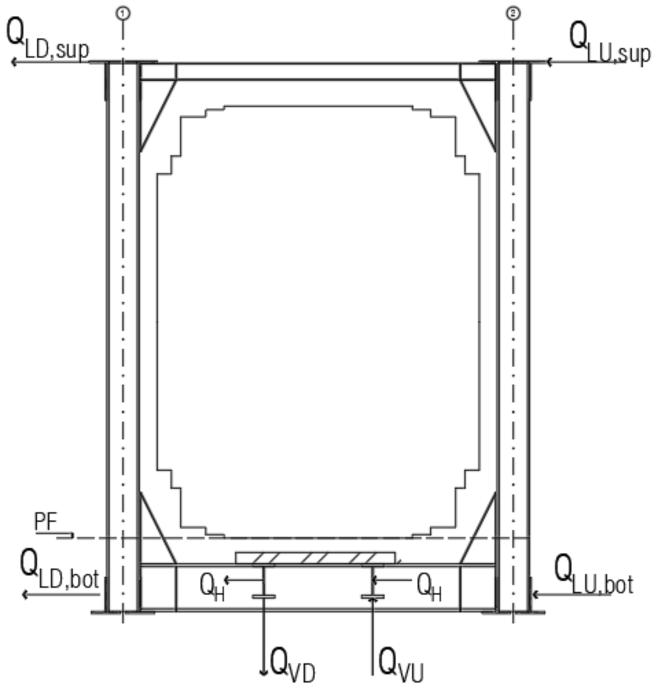


Figure 19. Case 1, Acting forces due to wind pressure

Where:

- $Q_{LD,top}$, $Q_{LU,\ top}$: Forces acting on the top chord.
- $Q_{LD,bot}$, $Q_{LU,\ bot}$: Forces acting on the bottom chord.
- Q_{VD}, Q_{VU}, Q_H: Forces acting on the stringers

This pressure is applied to the bridge deck, creating the uniformly distributed loads shown in Figure 19.

These forces will be given by the following set of equations:

4.5.1.1.1.1 <u>TOP CHORD</u>

On the top chord of the upwind lateral truss systems

$$Q_{LU,top} = \frac{2}{3} \cdot p(z) \cdot d_{L,top} \tag{8}$$

On the top chord of the downwind lateral truss systems

$$Q_{LD,top} = \frac{1}{3} \cdot p(z) \cdot d_{L,top} \tag{9}$$

Where, $d_{L,top}$ is the height of the top chord of the lateral truss systems.

4.5.1.1.1.2 <u>BOTTOM CHORD</u>

On the bottom chord of the upwind lateral truss systems

$$Q_{LU,bot} = \frac{2}{3} \cdot p(z) \cdot d_{L,bot} \tag{10}$$

On the bottom chord of the downwind lateral truss systems

$$Q_{LD,bot} = \frac{1}{3} \cdot p(z) \cdot d_{L,bot} \tag{11}$$

Where, $d_{L,bot}$ is the height of the bottom chord of the lateral truss systems.

4.5.1.1.1.3 <u>STRINGERS</u>

While the pressure p(z) applied to the gantry and the track depth generated a load:

$$Q_{L,G+T} = p(z) \cdot (d_G + d_{trk}) \tag{12}$$

The horizontal uniformly distributed loads applied to the stringers are:

$$Q_{H,D} = Q_{H,U} = \frac{Q_{L,G+T}}{2} \tag{13}$$

The load $Q_{L,G+T}$ is applied at a vertical distance from the axis of the stringers equal to:

$$d_{L,G+T} = \left(d_G + d_{trk} + d_{L,string}\right)/2 \tag{14}$$

Therefore, the vertically uniformly distributed loads applied to the stringers are:

$$Q_{V,U} = -Q_{V,D} = Q_{L,G+T} \cdot d_{L,G+T}/Tss \tag{15}$$

4.5.1.1.2 CASE 1 RESULTS:

The results will be shown below according to 4.5.1.1.1 and 4.4.2.

Table 10.Wind pressure, CASE 1

Internal deck width	b_i	[m]	5
Thickness of lateral trusses	$b_{Trusses}$	[m]	0.45
External deck width	b_e	[m]	5.9
Internal bridge height	d_i	[m]	7.21
O Bridge Limit	$di > dg + dtrk$		7.21
	d_{trk}	[m]	0.36
	d_{top}	[m]	0.55
	d_{bot}	[m]	0.50
External deck depth	d_e	[m]	8.26
	b_e/d_e	[-]	0.71
Pressure factor	$C_p = C_{f,x}$	[-]	2.20
Wind pressure	$p(z)$	[kN/m^2]	1.40
Stringer's depth	$d_{L,String}$	[m]	0.478

From Table 10, based on the initial condition for CASE 1, the value $C_{f,x} = 2.20$ and with this the value of the wind pressure, $p(z) = 1.40$ kN/m^2.

With the value of p(z) it is possible to get the values of the forces acting on the structure according to Figure 19 and using the set of equation given in 4.5.1.1.1.1,4.5.1.1.1.2 and 4.5.1.1.1.3.

Table 11. Forces acting on the top chord

Depth Top chord	$d_{L,Top}$	[m]	0.55
Upwind	Q_{Top}	[kN/m]	0.51
Downwind	Q_{Top}	[kN/m]	0.26

Table 12. Forces acting on the bottom chord

Depth Bottom chord	$d_{L,Bot}$	[m]	0.5
Upwind	Q_{Bot}	[kN/m]	0.47
Downwind	Q_{Bot}	[kN/m]	0.23

Table 13.Forces acting on the stringers

Load	$Q_{L,G+T}$	[kN/m]	10.13
Horizontal Load	Q_H	[kN/m]	5.06
Application Distance	$d_{L,G+T}$	[m]	3.84
Stringers Interasis	Tss	[m]	1.52
Vertical Load	Q_V	[kN/m]	25.61

These forces will be introduced in the 3D model as it is shown in Figure 20 and Figure 21.

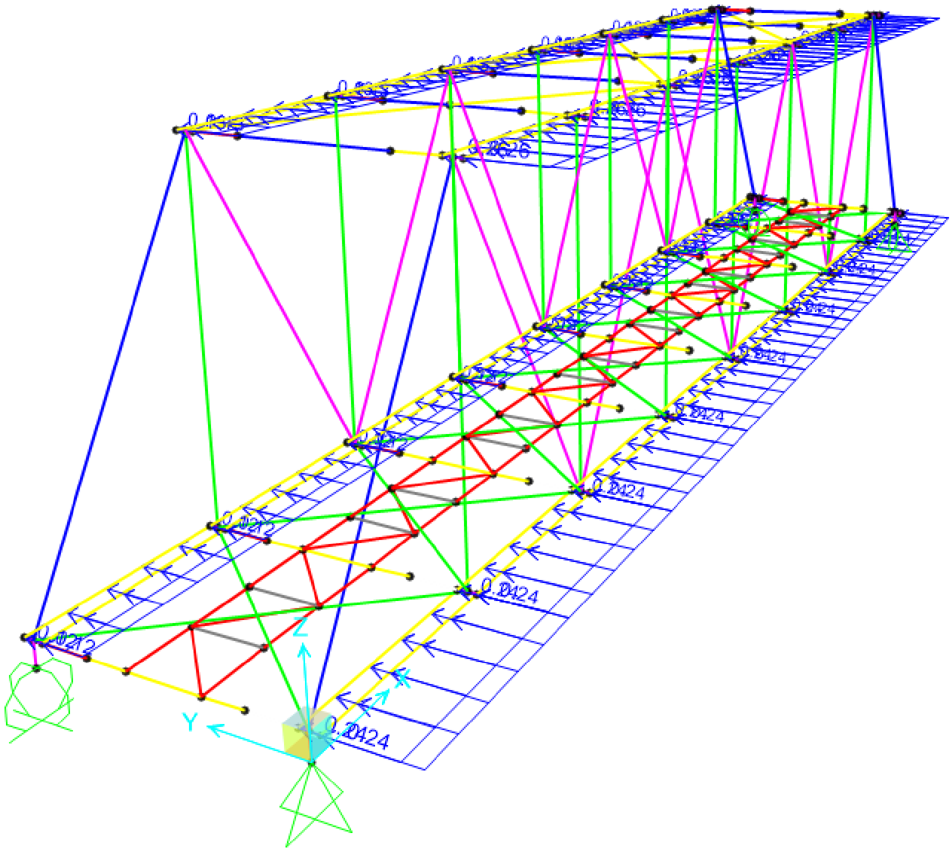


Figure 20. Forces acting on the Bottom and Top chords

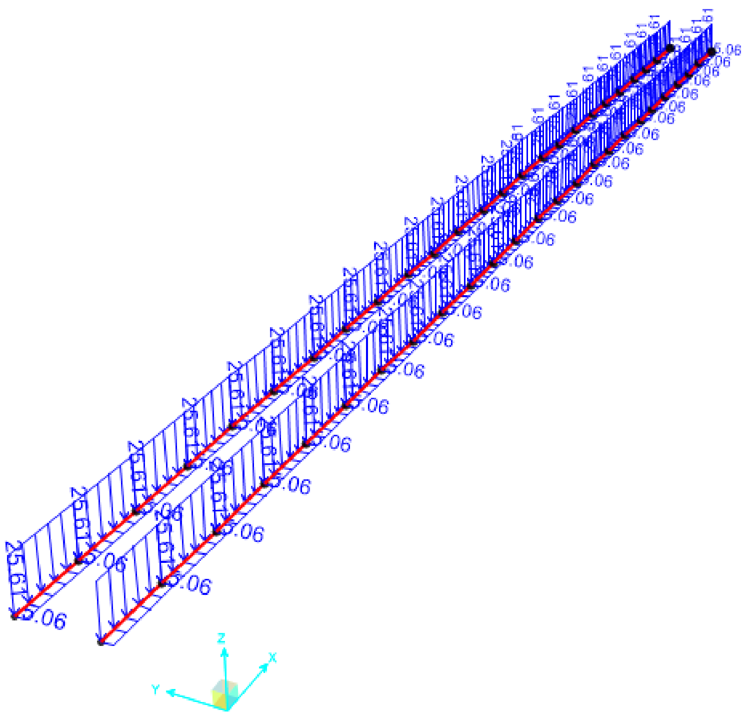


Figure 21. Forces acting on the stringers

4.5.1.1.3 CASE 2, TRAIN, NOISE BARRIER:

For CASE 2 the wind forces and the coefficient C_{fx} will be calculated following the condition, “There is traffic (Train) and there is a noise barrier placed in the free space between the train and the structure.” (see Figure 22 and Figure 23)

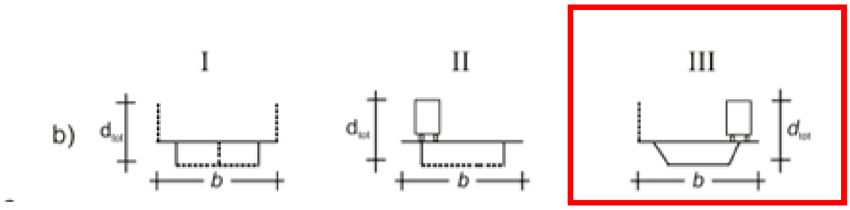


Figure 22..Values of b and d, for CASE 2

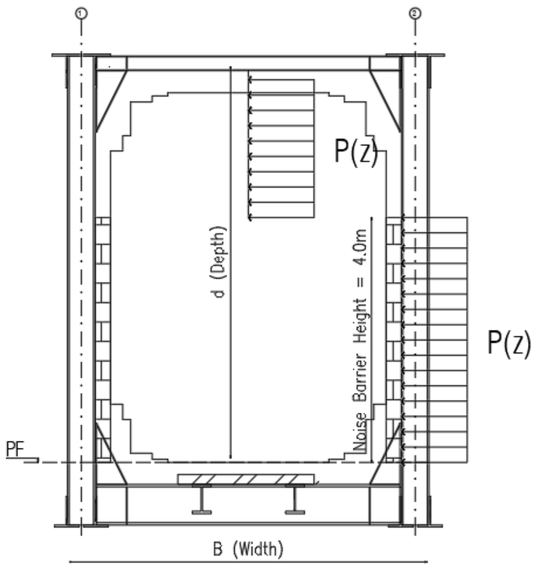


Figure 23. Geometry Case 2. Traffic with Noise barrier

For this case the values for *b* and *d* will be taken as the ones in Figure 23, where *b* will be the total width of the “deck” while *d* will be the total gantry depth as for CASE 1.

With this, the coefficient C_{fx} will be the same for CASE 1 and CASE 2.

For CASE 2, it will be assumed that the wind forces will be applying on the Gantry height and on the noise barrier as it is shown in Figure 23, as a result of this wind pressure (P(z)) there will be different forces acting on the bridge structure (see Figure 24)

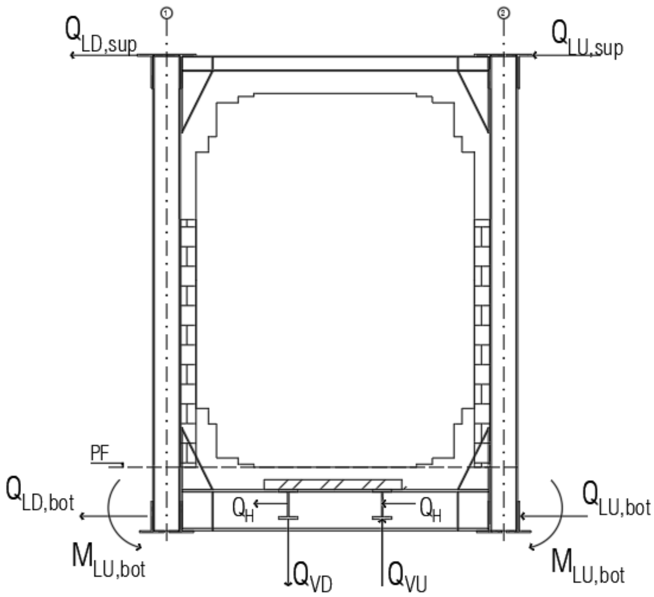


Figure 24. Case 2, Acting forces due to wind pressure

Where:

- $Q_{LD,sup}$, $Q_{LU,\ sup}$: Forces acting on the top chord.
- $Q_{LD,bot}$, $Q_{LU,\ bot}$: Forces acting on the bottom chord
- $M_{LU,\ bot}$: Additional Torsional moment acting on the bottom chord.
- Q_{VD}, Q_{VU}, Q_H: Forces acting on the stringers

The forces acting on the top and bottom chords will be calculated as it was made for CASE 1 in 4.5.1.1.1.1,4.5.1.1.1.2.

The additional forces and the torsional moment acting on the bottom chord will be given by the following set of equations:

4.5.1.1.3.1 BOTTOM CHORD

Additional load on the bottom chord due to the wind acting on the noise barrier:

$$Q_{L,bot\,(NB)} = \frac{h_{NB} \cdot p(z)}{2} \tag{16}$$

The load $Q_{L,bot\,(NB)}$ is applied at a vertical distance from the axis of the bottom chord equal to:

$$d_{L,M} = \left(d_{L,trans} + d_{trk} + \frac{d_{NB}}{2}\right) \tag{17}$$

Additional Torsional moment on the bottom chord due to the wind acting on the noise barrier

$$M_{T,NB} = Q_{L,bot\,(NB)} \cdot d_{L,M} \tag{18}$$

This torsional moment will be introduced in the model as a pair of forces given by:

$$F_{T,NB} = M_{T,NB} / \, d_{L,bot(2)} \tag{19}$$

4.5.1.1.3.2 STRINGERS

The forces on the stringers due to the wind acting on the train will be calculated as it was made for CASE 1 in 4.5.1.1.1.3

Equation (14) will be given by the load $Q_{L,G+T}$ is applied at a vertical distance from the axis of the stringers equal to:

$$d_{L,G+T} = (\frac{d_G - h_{NB}}{2}) + d_{trk} + d_{L,string} + h_{NB} \tag{20}$$

4.5.1.1.4 CASE 2 RESULTS:

The results will be shown below according to 4.5.1.1.3 and 4.4.2.

Table 14. Wind pressure, CASE 2

Internal deck width	b_i	[m]	5
Thickness of lateral trusses	$b_{Trusses}$	[m]	0.45
External deck width	b_e	[m]	5.9
Internal bridge height	d_i	[m]	7.21
O Bridge Limit	di > dg + dtrk		7.21
	d_{trk}	[m]	0.36
	d_{top}	[m]	0.55
	d_{bot}	[m]	0.50
External deck depth	d_e	[m]	8.26
	b_e/d_e	[-]	0.71
Pressure factor	$C_p=C_{f,x}$	[-]	2.20
Wind pressure	p(z)	[kN/m^2]	1.40
Noise barrier height	h_3	[m]	4.00
Transverse Beam height	$d_{L,trans}$	[m]	0.665
Stringer's height	$d_{L,String}$	[m]	0.478

From Table 14, based on the initial condition for CASE 2, the value $C_{f,x}$ = 2.20 and with this the value of the wind pressure, p(z) = 1.40 kN/m^2, as it was mentioned before the values are the same as for CASE 1.

With the value of p(z) it is possible to get the values of the forces acting on the structure according to Figure 24 and using the set of equation given in 4.5.1.1.1.1,4.5.1.1.1.2, 4.5.1.1.1.3, in addition with the modify ones in 4.5.1.1.3.1 and 4.5.1.1.3.2.

Table 15. Forces acting on the top chord

Depth Top chord	$d_{L,Top}$	[m]	0.55
Upwind	Q_{Top}	[kN/m]	0.51
Downwind	Q_{Top}	[kN/m]	0.26
Depth Bottom chord	$d_{L,Bot}$	[m]	0.50

Table 16.Forces acting on the bottom chord

Load at Noise Barrier	Q_L	[kN/m]	5.62
Horizontal Load	Q_H	[kN/m]	2.81
Application Distance	$d_{L,M}$	[m]	3.03
Additional torsional moment	$M_{t,NB}$	[kN-m]	8.50
Application Distance	$d_{L,Bot(2)}$	[m]	0.40
Pair of forces (Torsional Moment)	$F_{t,NB}$	[kN/m]	21.24
Upwind	Q_{Bot}	[kN/m]	3.28
Downwind	Q_{Bot}	[kN/m]	3.04

Table 17.Forces acting on the stringers

Load	$Q_{L,G+T}$	[kN/m]	4.00
Horizontal Load	$Q_{H,G+T}$	[kN/m]	2.00
Application Distance	$d_{L,G+T}$	[m]	6.26
Stringers Interasis	Tss	[m]	1.52
Vertical Load	$Q_{V,G+T}$	[kN/m]	16.49

These forces will be introduced in the 3D model as it is shown in Figure 25 , Figure 26 and Figure 27.

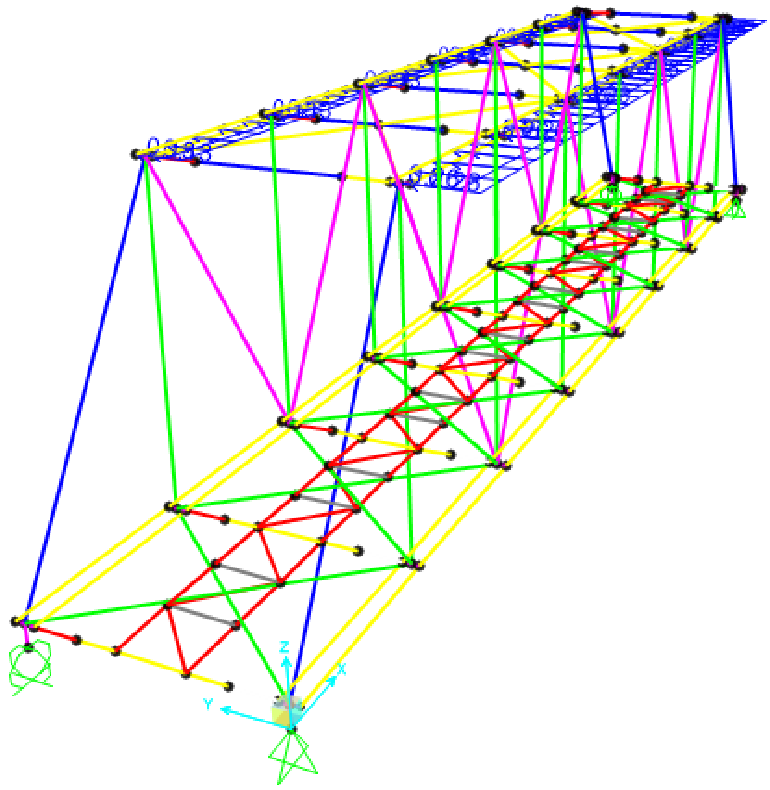


Figure 25. Forces acting on the Top chord

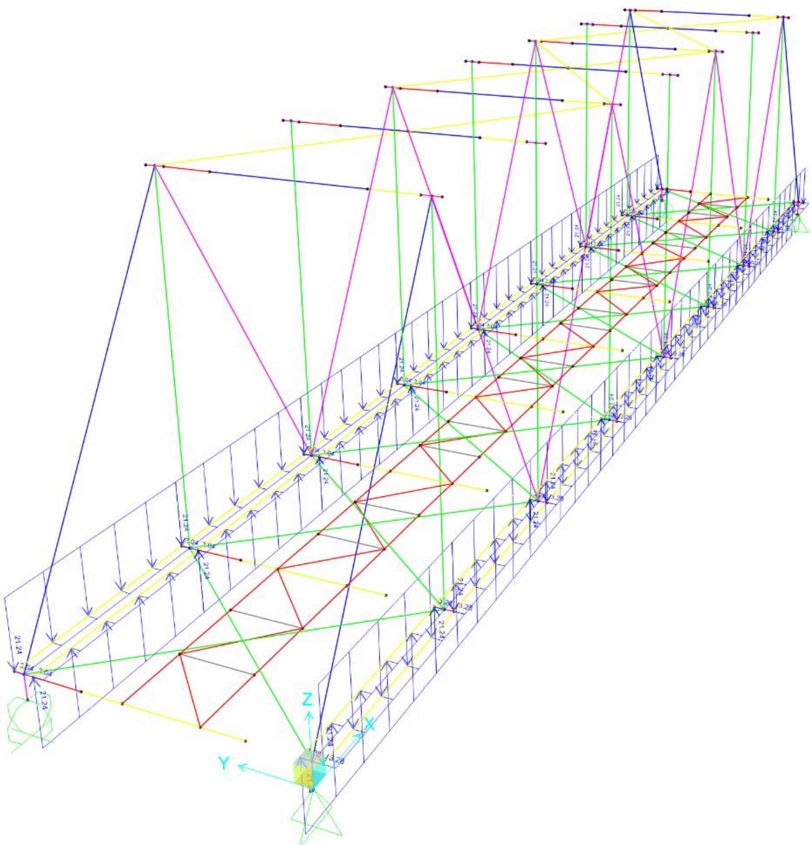


Figure 26. Forces acting on the bottom chord

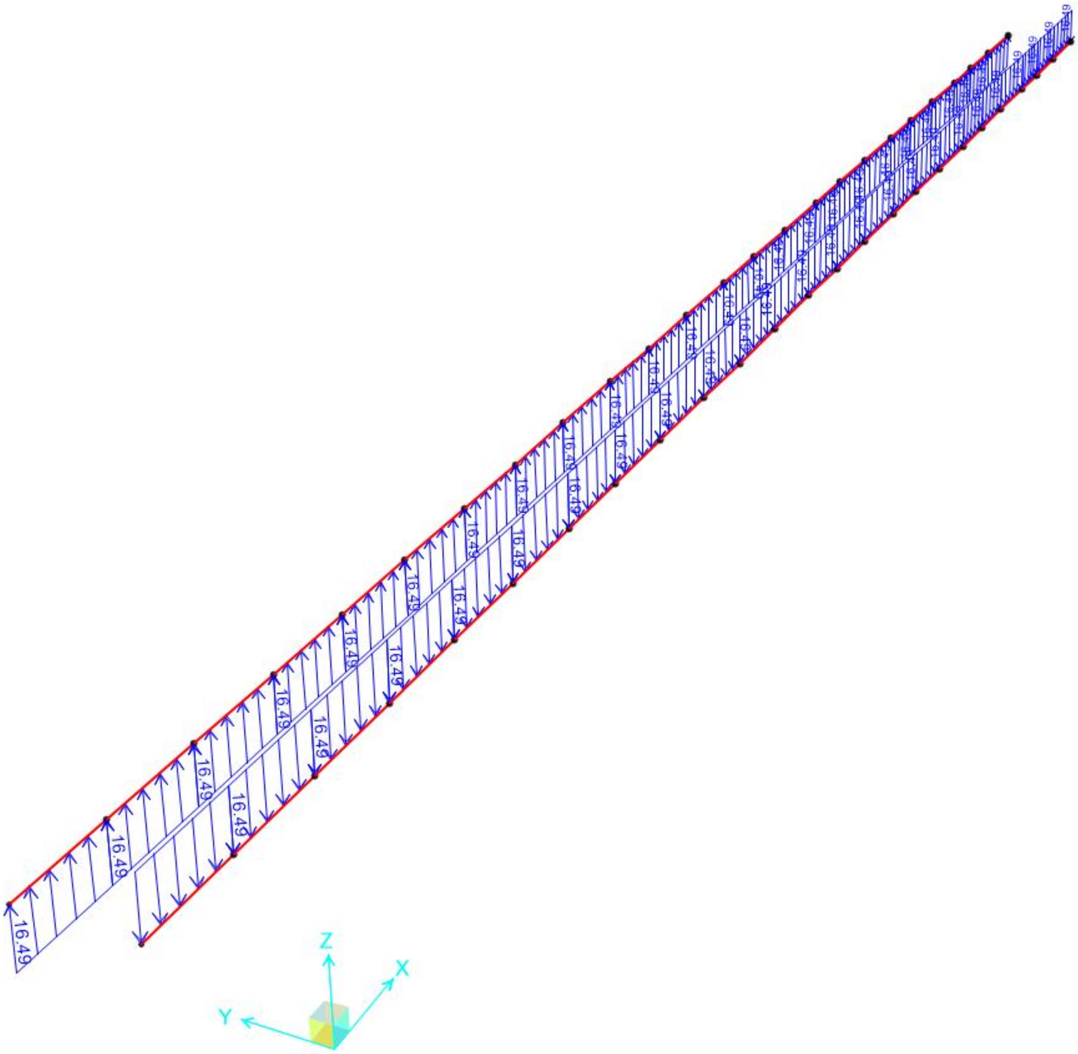


Figure 27. Forces acting on the stringers

4.5.1.1.5 CASE 3, NO TRAIN, NO NOISE BARRIER:

For CASE 3 the wind forces and the coefficient C_{fx} will be calculated following the condition, “There is no traffic (Train) and there is no noise barrier” (see Figure 29). However in this case, the wind forces will be divided in two different conditions, CONDITION 1 will be according to 8.3.1[2](see Figure 28) for the “deck” of the bridge, while CONDITION 2 will be according to 7.11[2] (see Figure 30 and Figure 31).

4.5.1.1.5.1 CONDITION 1:

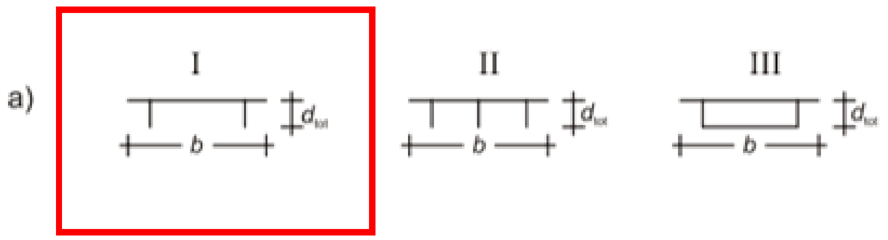


Figure 28. Values of b and d, for CASE 3

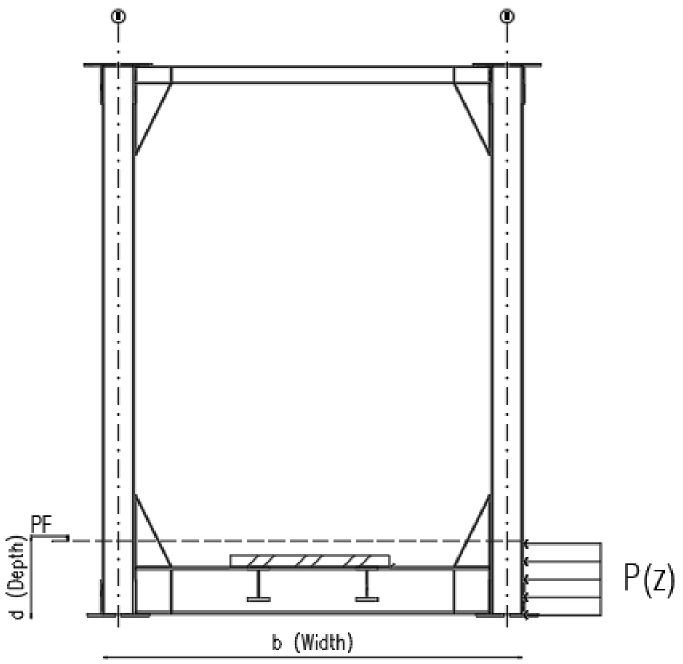


Figure 29.Geometry Case 3. No Traffic and No Noise barrier

For case 3- CONDITION 1, it will be assumed that the wind forces will be applying to the bottom chord (see Figure 29).

4.5.1.1.5.2 <u>CONDITION 2:</u>

As it was mentioned above for CONDITION 2 it is necessary to refer to paragraph 7.11 of the EN 1991-1-4:2005[2], The force coefficient, C_{fx} of lattice structures and scaffoldings with parallel chords should be obtained by the following expression(21).

$$C_{f,x} = C_{f,0} \cdot \psi_{\lambda} \qquad (21)$$

where:

- $C_{f,0}$: it is the force coefficient of lattice structures and scaffoldings without end-effects. It is given by Figure 31 as a function of solidity ratio φ and Reynolds number Re.
- Re: it is the Reynolds number using the average member diameter b_i, Figure 31 is based on the Reynolds number with $v = \sqrt{\frac{2\,q_p}{\rho}}$ and q_p is given in chapter 4.5 of the EN 1991-1-4:2005[2]
- ψ_{λ}: it is the end-effect factor as a function of the slenderness of the structure, λ, calculated with *L* and width *b=d* (see Figure 30)

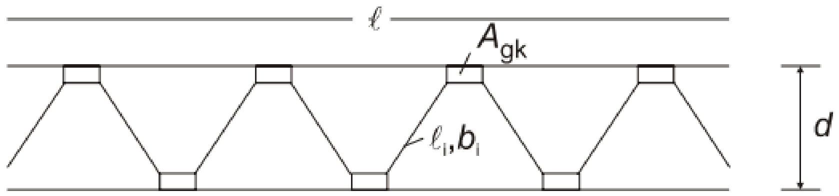


Figure 30.Lattice structure or scaffolding

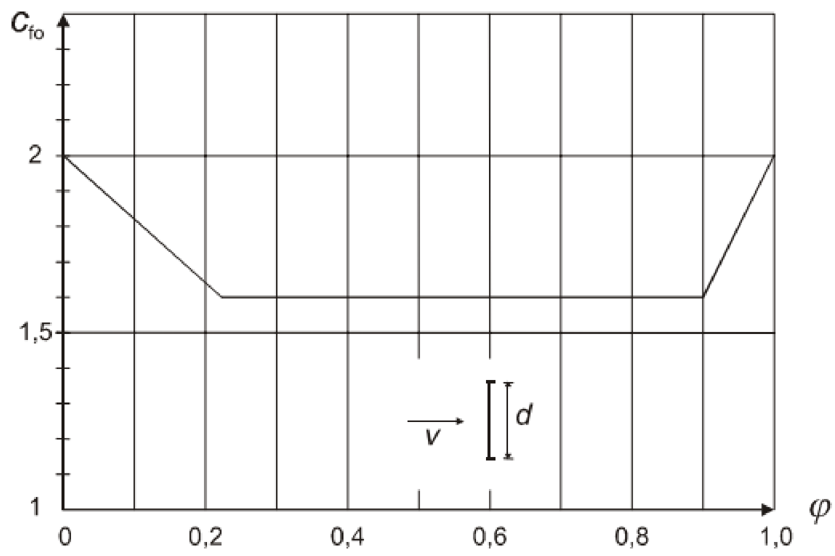


Figure 31. Force coefficient $C_{f,0}$ for a plane lattice structure with angle members as a function of solidity ratio φ

Now, in the same paragraph 7.11(2) [2] the solidity ratio φ, is defined by the following expression(22):

$$\varphi = \frac{A}{A_c} \tag{22}$$

Where:

- A: is the sum of the projected area of the members and the gusset plates of the face projected normal to the face.
- $A_{c:}$ is the area enclosed by the boundaries of the face projected normal to the face.

With A and A_c given by:

$$A = \sum_i b_i \cdot l_i + \sum_k A_{gk} \tag{23}$$

$$A_c = d \cdot l \tag{24}$$

Where:

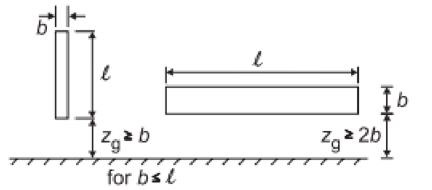
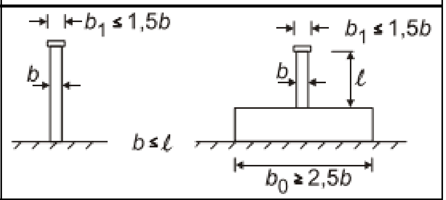
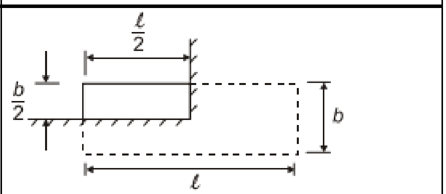
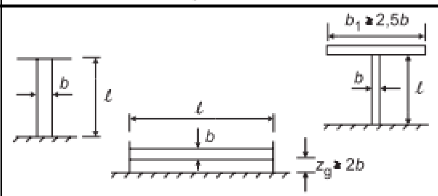
- *L*: is the length of the lattice (see Figure 30)
- *d*: is the width of the lattice (see Figure 30)
- b_i, l_i: is the width and length of the individual member projected normal to de face (see Figure 30).
- A_{gk}: is the area of the gusset plate k

Now, it is necessary to determine the value of the end-effect factor (ψ_λ).

According to paragraph 7.13[2] the end-effect factor should be determined as a function of slenderness ratio λ. And in the same way, the effective slenderness λ should be defined depending on the dimensions of the structure and its position.

According to the NOTE in 7.13[2] Recommended values for λ are given in Table 18, and indicative values for ψ_λ are given in Figure 32 for different solidity ratio φ.

Table 18. Recommended values of λ for cylinders, polygonal, rectangular, sharp edge and lattice structures

No.	Position of the structure, wind normal to the plane of the page	Effective slenderness λ
1		For polygonal, rectangular and sharp edged sections and lattice structures: for ℓ ≥ 50 m, λ =1,4 ℓ/b or λ = 70, whichever is smaller; for ℓ <15 m, λ =2 ℓ/b or λ = 70, whichever is smaller. For circular cylinders: for ℓ ≥ 50, λ =0,7 ℓ/b or λ =70, whichever is smaller; for ℓ <15 m, λ =ℓ/b or λ =70, whichever is smaller. For intermediate values of ℓ, linear interpolation should be used
2		
3		
4		for ℓ ≥ 50 m, λ =0,7 ℓ/b or λ = 70, whichever is larger; for ℓ <15 m, λ =ℓ/b or λ = 70, whichever is larger. For intermediate values of ℓ, linear interpolation should be used

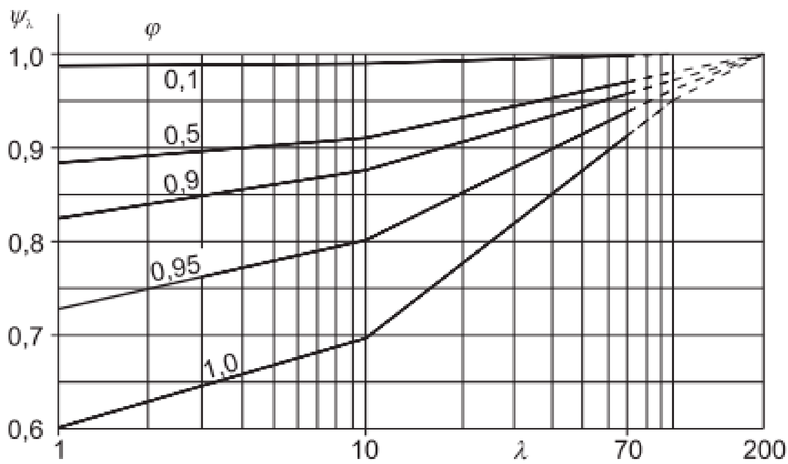


Figure 32. Indicative values of the end-effect factor ψ_λ, as function of solidity ratio φ versus slenderness λ

For case 3- CONDITION 2, it will be assumed that the wind forces will be applied on the lattice structures (trusses) as it is shown in Figure 33.

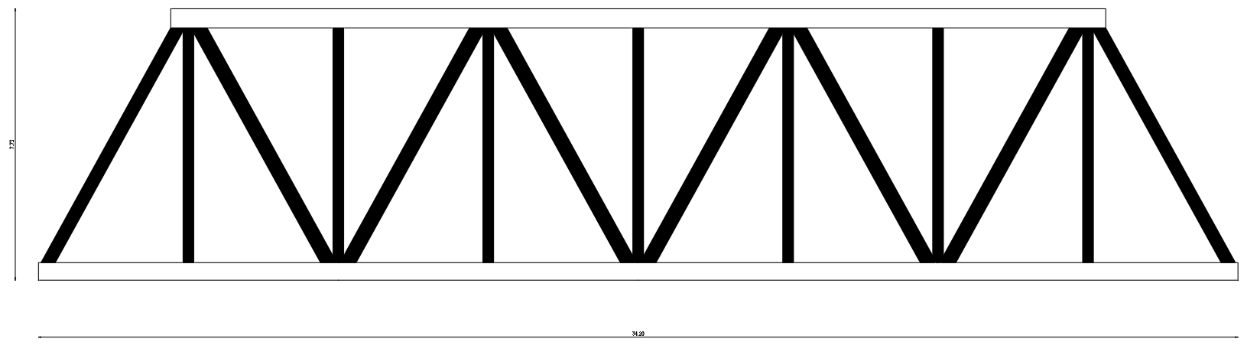


Figure 33.Lateral view of the bridge

The wind forces will be applied in the black area indicated in Figure 33, the forces will be perpendicular to the plane as it is shown in Figure 34

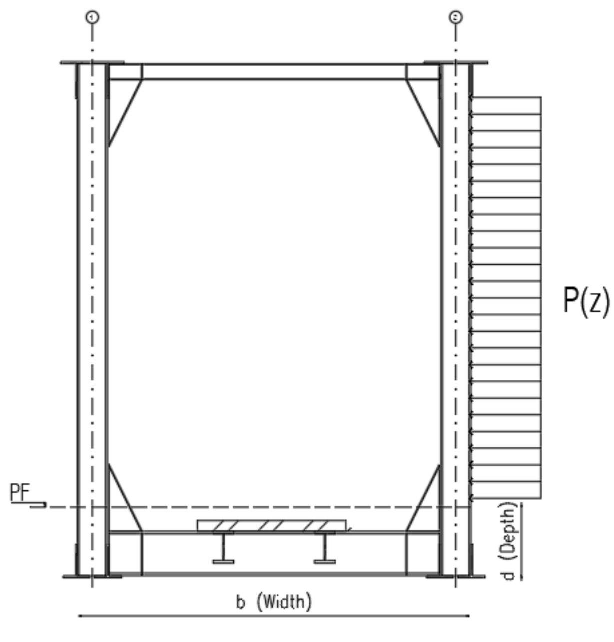


Figure 34. Force applied on the truss system.

4.5.1.1.6 CASE 3, RESULTS:

The results will be shown below according to 4.5.1.1.5 and 4.4.2.

4.5.1.1.6.1 CONDITION 1

Table 19.Wind pressure, CASE 3, CONDITION 1.

Internal deck width	b_i	[m]	5
Thickness of lateral trusses	$b_{Trusses}$	[m]	0.45
External deck width	b_e	[m]	5.9
Transversal Beam height	$d_{L,trans}$	[m]	0.665
	b_e/d_e	[-]	8.87
Pressure factor	$C_p=C_{f,x}$	[-]	1.30
Wind pressure	p(z)	[kN/m^2]	0.83

From Table 19, based on the initial condition for CASE 3, the value of the pressure factor for CONDITION 1 will be $C_{f,x}$ = 1.30 and with this the value of the wind pressure, p(z) = 0.83 kN/m^2.

With the value of p(z) it is possible to get the values of the forces acting on the structure according to Figure 29 and using the set of equation given in 4.5.1.1.1.1.

Table 20.Forces acting on the bottom chord

Depth Bottom chord	$d_{L,Bot}$	[m]	0.50
Upwind	Q_{Top}	[kN/m]	0.28
Downwind	Q_{Top}	[kN/m]	0.14

These forces will be introduced in the 3D model as it is shown in Figure 35.

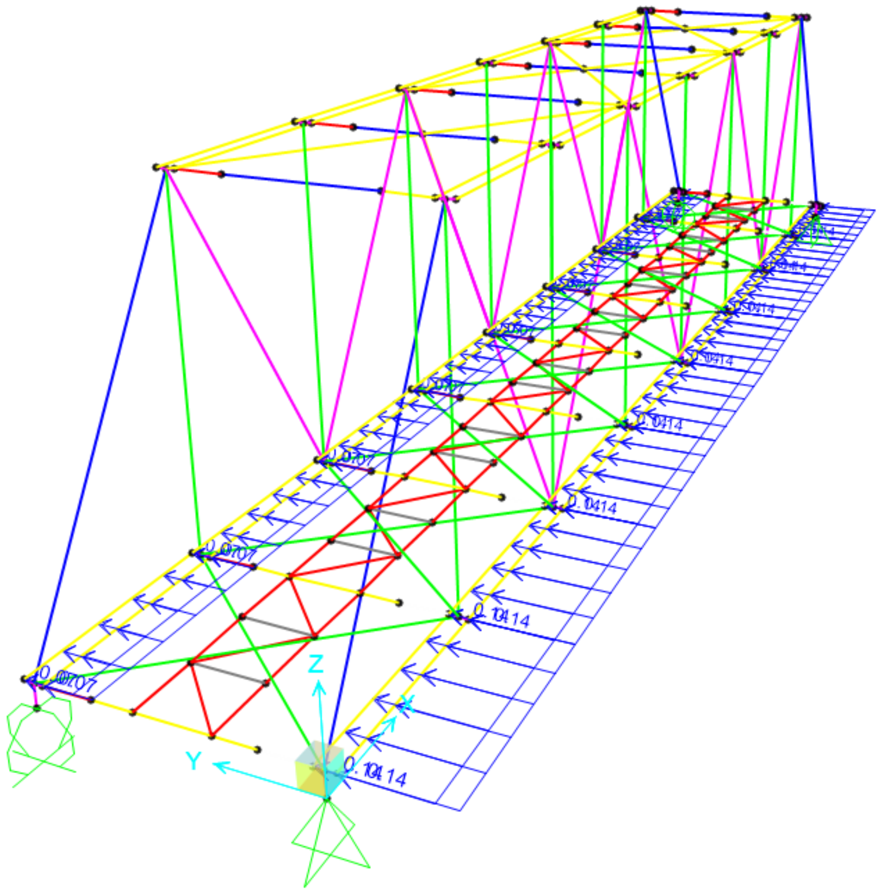


Figure 35. Forces acting on the bottom chord, CONDITION 1, CASE 3

4.5.1.1.6.2 CONDITION 2:

For this condition, it is necessary to get the values of the solidity ratio from the ratio of areas and then the force coefficient according to Figure 30 and Figure 31.

Table 21.Value of Solidity ratio and Force coefficient

Projected Area	A	[m^2]	83.24
Enclosed Area	A_c	[m^2]	264.37
Solidity Ratio	φ	[-]	0.31
Force Coefficient (Max. value)	$C_{f,0}$	[-]	2.00
Force Coefficient (Min. value)	$C_{f,0}$	[-]	1.60

Starting from the solidity ratio φ =A/Ac = 0.31, it is possible to get the value of the force coefficient ($C_{f,o}$) according to Figure 31, this value of $C_{f,0}$ will be 1.6 for any value of φ between,$0.2 < \varphi < 0.4$.

As for the optimization, it will be necessary to recalculate the value of φ according to the geometry of the elements, and to simplify the analysis this value will be set according to the following conditions:

$$C_{f,0} = 1.6 \quad if \quad 0.2 < \varphi < 0.4 \tag{25}$$

$$C_{f,0} = 2 \quad if \quad 0 < \varphi < 0.2 \tag{26}$$

In order to get the value of $C_{f,x}$, it is necessary to obtain the value of the end-effect factor ψ_λ, this will be get according to Table 18 and Figure 32.

Table 22. Wind pressure, CASE 3, CONDITION 2

Total Length	L	[m]	40.00
Total height	b	[m]	10.00
Effective Slenderness			
Case 1,2 and 3	λ	[-]	6.29
L >50 m			5.60
L <15m			8.00
15<L<50			6.29
Case 4	λ	[-]	70.00
L >50 m			70.00
L <15m			70.00
15<L<50			70.00
Select CASE of analysis			**CASE 1**
End effect factor (Max. value)	ψ_λ	[-]	1.00
End effect factor (Min. value)	ψ_λ	[-]	0.60
Pressure factor			
Force Coefficient (Critical value)	**C_f**	**[-]**	**2.00**
COMB 1 (Max-Max)		[-]	2.00
COMB 2 (Max-Min)		[-]	1.20
COMB 3 (Max-Max)		[-]	1.60
COMB 4 (Max-Min)		[-]	0.96
Wind pressure	p(z)	[kN/m^2]	1.28

The effective slenderness is given by Table 18 selecting one the 4 cases indicated, for this analysis it will select CASE 1, this value also depends on the total length and height of the bridge.

As for the optimization of the bridge, the length is a parameter that will not change, it will be set to a value of L=43 m(original value), and as the approximate value for height is given by the codes it will be set not larger than b=10m, with this information the value of λ will be, **λ = 6.29.** .

Now with the value of λ and φ is possible to get a value for ψ_λ, according to Figure 32 and in order to simplify the analysis it will be considered just the maximum and minimum value, for this case **ψ_λ,=1 or ψ_λ,=0.6.**

With the conditions given in equations (25) and (26) for the value of $C_{f,0,}$ and the two values given for ψ_λ,it is possible to create 4 different conditions, however as these parameters will affect directly the value of wind pressure p(z), it will be considered the critical value, and for this will be selected the maximum one, **$C_{f,x}$= 2.00.**

Finally, from Table 22, based on the initial condition for CASE 3, CONDITION 2, and with the values above, the value of $C_{f,x}$ will be $C_{f,x}$ = 2.00 and with then the value of the wind pressure, p(z) = 1.28 kN/m^2.

Table 23. Forces acting on the lattice structure

Truss Flange width	$d_{L,Bot}$	[m]	0.45
Upwind	Q_{Top}	[kN/m]	0.34
Downwind	Q_{Top}	[kN/m]	0.13

These forces will be introduced in the 3D model as it is shown in Figure 36.

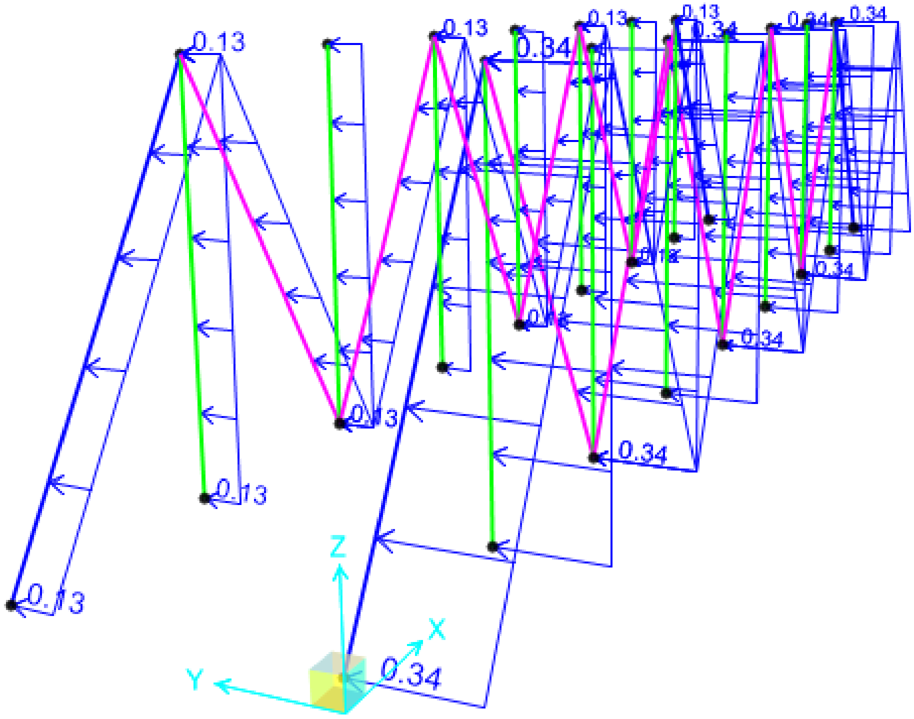


Figure 36. Forces acting on the lattice structure, CONDITION 2, CASE 3

4.5.1.1.7 CASE 4, NOISE BARRIER, NO TRAIN:

For CASE 4 the wind forces and the coefficient C_{fx} will be calculated following the condition, "There is no traffic (Train) and there is a noise barrier placed in the free space between the train and the structure."(see Figure 37 and Figure 38)

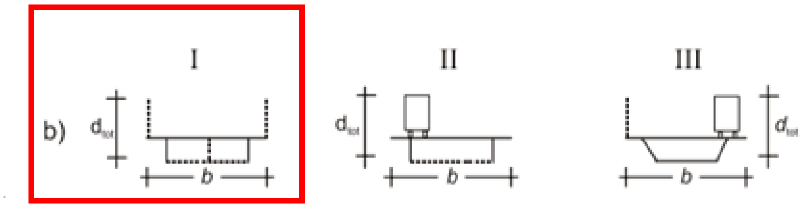


Figure 37. Values of b and d, for CASE 4

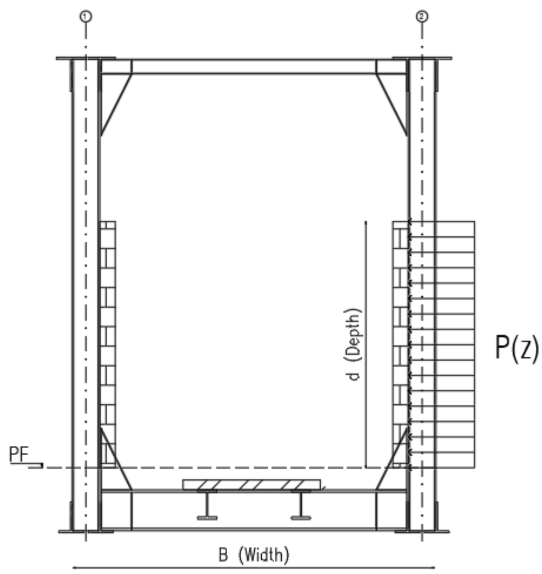


Figure 38. Geometry Case 4. No Traffic with Noise barrier

For this case the values for *b* and *d* will be taken as the ones in Figure 38, where *b* will be the total width of the "deck" while *d* will be the noise barrier height.

For CASE 4, it will be assumed that the wind forces will be applied to noise barrier height, as a result, there are going to be forces applied on the bottom and top chords, and an additional torsional moment on the bottom chord.(see Figure 39)

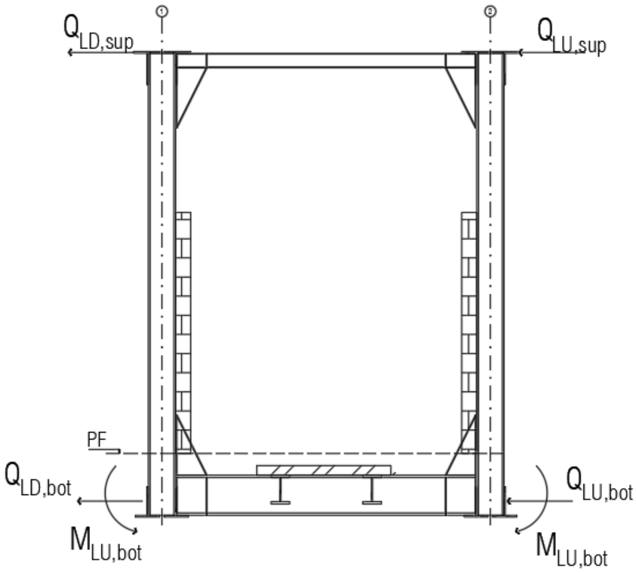


Figure 39. Case 4, Acting forces due to wind pressure

Where:

- $Q_{LD,sup}$, $Q_{LU,\ sup}$: Forces acting on the top chord.
- $Q_{LD,bot}$, $Q_{LU,\ bot}$: Forces acting on the bottom chord
- $M_{LU,\ bot}$: Additional Torsional moment acting on the bottom chord.

As for CASE 2 the torsional moment M_{Lu} will be applied as a pair of forces acting on the bottom chords.

The forces acting on the top and bottom chords will be calculated as it was made for CASE 1 in 4.5.1.1.1.1,4.5.1.1.1.2.

The additional forces and the torsional moment acting on the bottom chord will be calculated as it was made in 4.5.1.1.3.1.

4.5.1.1.8 CASE 4, RESULTS:

The results will be shown below according to 4.5.1.1.7 and 4.4.2.

Table 24. Wind pressure, CASE 4

Internal deck width	b_i	[m]	5
Thickness of lateral trusses	$b_{Trusses}$	[m]	0.45
External deck width	b_e	[m]	5.9
Internal bridge height	d_i	[m]	7.21
Noise barrier height	h_3	[m]	4.00
	d_{trk}	[m]	0.36
	d_{top}	[m]	0.55
	d_{bot}	[m]	0.50
External deck depth	d_e	[m]	4.86
	b_e/d_e	[-]	1.21
Pressure factor	$C_p = C_{f,x}$	[-]	2.06
Wind pressure	p(z)	[kN/m^2]	1.32

From Table 24, based on the initial condition for CASE 3, the value $C_{f,x}$ = 2.06 and with this the value of the wind pressure, p(z) = 1.32 kN/m^2.

With the value of p(z) it is possible to get the values of the forces acting on the structure according to Figure 39 and using the set of equation given in 4.5.1.1.1.1,4.5.1.1.1.2, in addition with the modify one in 4.5.1.1.3.1, it is possible to get the forces on the bottom and top chords, in this case there is no any force acting on the stringers.

Table 25. Forces acting on the top chord

Depth Top chord	$d_{L,Top}$	[m]	0.55
Upwind	Q_{Top}	[kN/m]	0.48
Downwind	Q_{Top}	[kN/m]	0.24

Table 26.. Forces acting on the bottom chord

Depth Bottom chord	$d_{L,Bot}$	[m]	0.50
Transverse Beam height	$d_{L,trans}$	[m]	0.665
Load at Noise Barrier	Q_L	[kN/m]	5.26
Horizontal Load	Q_H	[kN/m]	2.63
Application Distance	$d_{L,G+T}$	[m]	3.03
Additional torsional moment	M_t	[kN-m]	7.96
Application Distance	$d_{L,Bot(2)}$	[m]	0.40
Pair of forces (Torsional Moment)	$F_{t,NB}$	[kN/m]	19.89
Upwind	Q_{Bot}	[kN/m]	3.07
Downwind	Q_{Bot}	[kN/m]	2.85

These forces will be introduced in the 3D model as it is shown in Figure 40 and Figure 41.

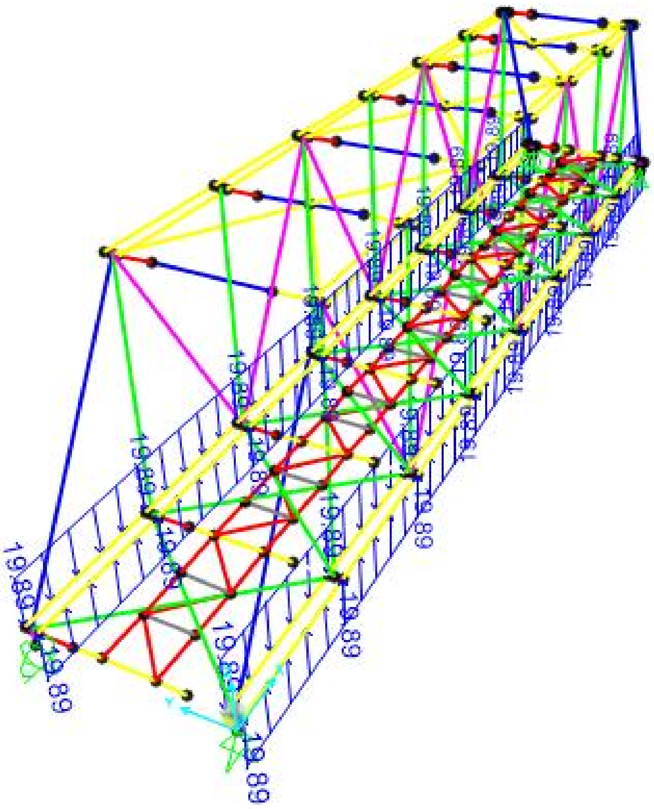


Figure 40. Torsional forces acting on the bottom chord

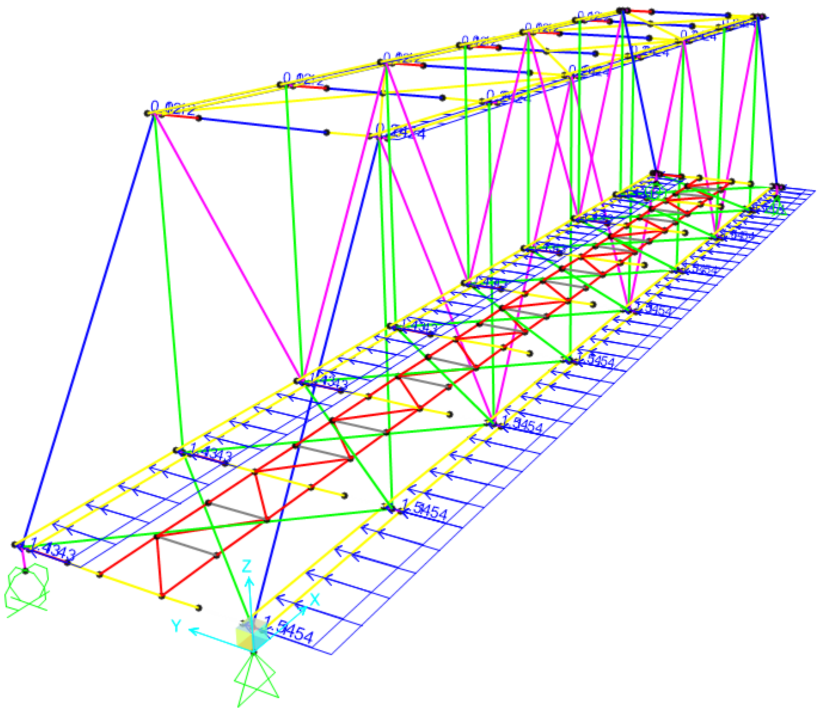


Figure 41. Forces acting on bottom and top chord

4.5.1.2 *VERTICAL WIND*

The vertical forces applied on the bridge will be divided in 4 cases depending on the direction of the wind application, it can be summed in Table 27.

Table 27.Vertical Wind Cases

	UPLIFTING	DOWN-LIFTING
RIGHT	CASE 5	CASE 6
LEFT	CASE 7	CASE 8

However, these cases are assumed not to happen at the same time, they are mutually exclusive so within the 3D model they will be an envelope.

NOTE: Due to the symmetry of the structure, it will be considered just 1 direction of action in this case the wind forces will be applied just in the right direction and with this just 2 cases will be analyzed CASE 5 and CASE 6.

According to 4.4.1.1.2, the wind pressure acting in vertical direction is not uniform in transverse direction as it generates a load with a transverse eccentricity of b/4, such eccentricity generates a difference between the upwind and the downwind given by the following set of equations:

4.5.1.2.1.1 UPLIFTING:

$$p_{z,UW} = \frac{7}{4} p(z) \tag{27}$$

$$p_{z,DW} = \frac{1}{4} p_z \tag{28}$$

4.5.1.2.1.2 DOWN-LIFTING:

$$p_{z,UW} = \frac{1}{4} p(z) \tag{29}$$

$$p_{z,DW} = \frac{7}{4} p_z \tag{30}$$

4.5.1.3 VERTICAL WIND, RESULTS:

The results will be shown below according to 4.5.1.2 and 4.4.1.1.2.

Table 28. Wind pressure, Vertical Direction

Wind tunnel test?			NO
Lift force coefficient	$C_{f,z}$	[-]	0.90
Average wind pressure	p_z	[kN/m^2]	0.57

As mentioned in 4.4.1.1.2 in absence of a Wind tunnel test it is possible to set the lift coefficient $C_{f,z} = \pm 0.9$.

With this the value of the wind pressure will be $P(z) = 0.57$ kN/m^2.

Table 29. Uplifting and Down-lifting forces

Transverse Beam Interaxis	T_{st}	[m]	4.30
<u>UPLIFTING</u>			
Up	$p_{z,UW}$	[kN/m]	4.32
Down	$p_{z,DW}$	[kN/m]	0.62
<u>DOWNLIFTING</u>			
Up	$p_{z,UW}$	[kN/m]	0.62
Down	$p_{z,DW}$	[kN/m]	4.32

With the value of wind pressure, it is possible to get the acting forces given in Table 29.

These forces will be introduced in the 3D model as it is shown in Figure 43 and Figure 45.

4.5.1.3.1 <u>CASE 5:</u>

In this case, the vertical forces resulting from the wind pressure in the right direction will be represented as it is shown in Figure 42

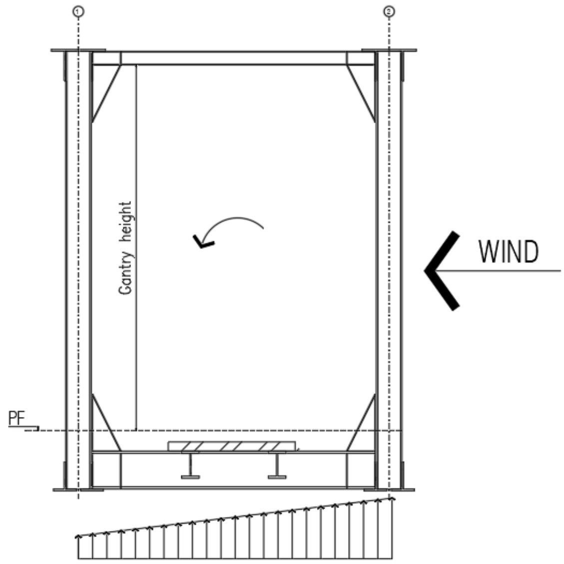


Figure 42. Case 5, Uplifting caused by the wind in the right direction

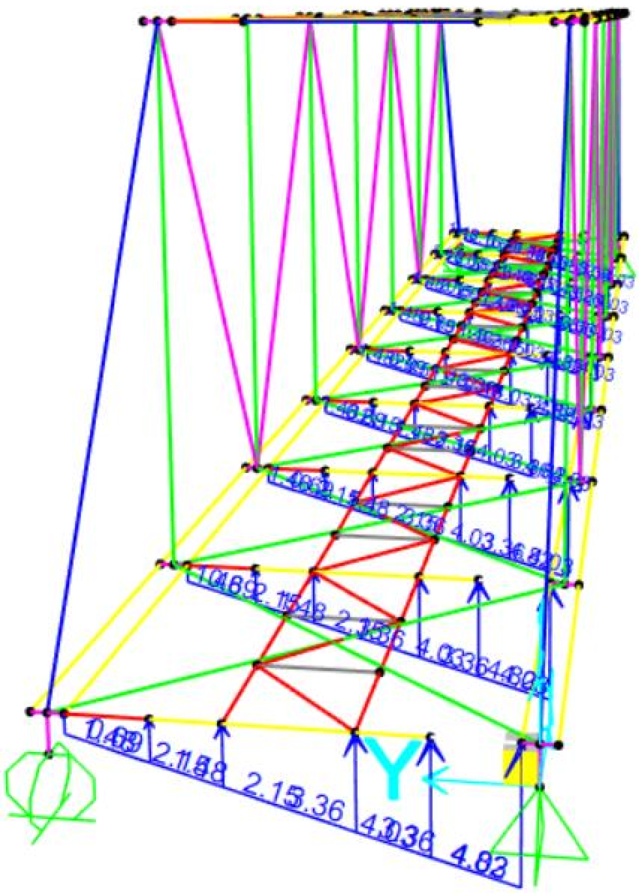


Figure 43. Case 5, 3D Model

4.5.1.3.2 <u>CASE 6:</u>

In this case, the vertical forces resulting from the wind pressure in the right direction will be represented as it is shown in Figure 44

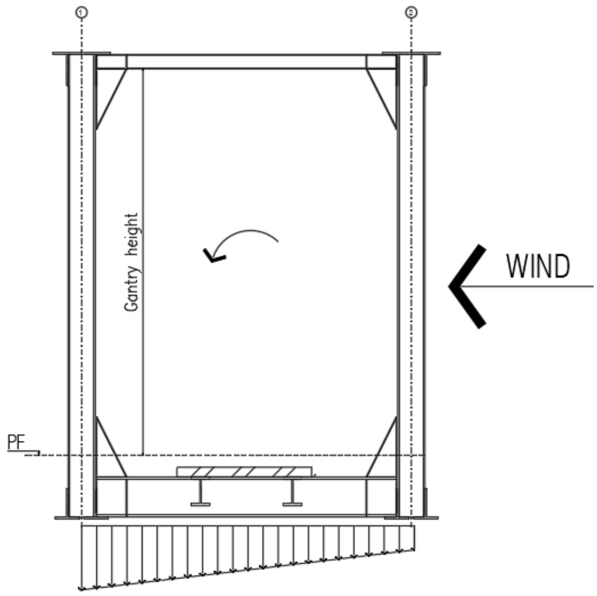


Figure 44. Case 6, Down lifting caused by the wind in the right direction

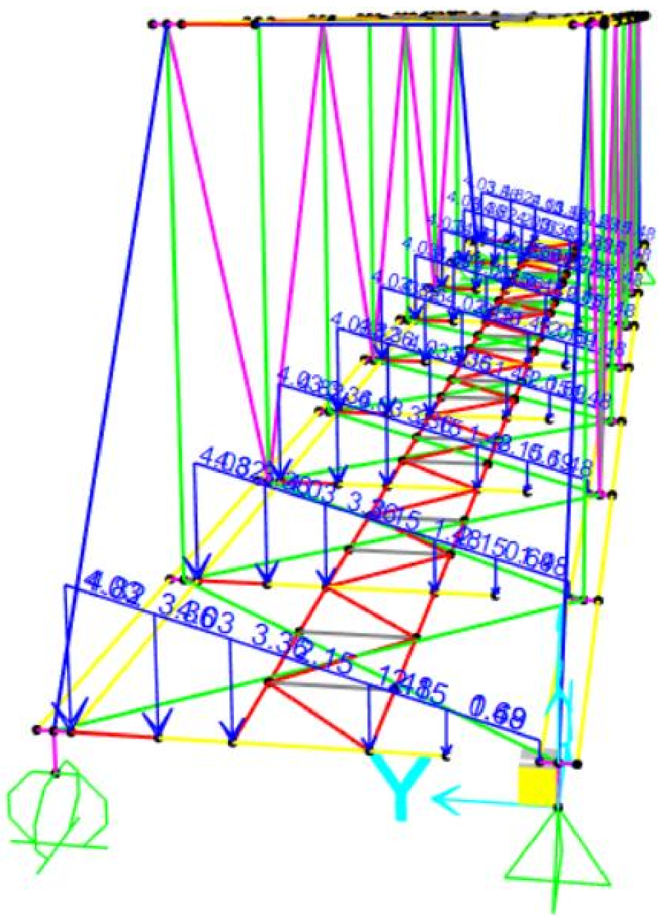


Figure 45. Case 6, 3D Model

4.6 SEISMIC ACTION

According to NTC 2018**Error! Reference source not found.**, it is necessary to determine the forces and effect that an earthquake induces on a structure. These loads must be considered in the structural design to ensure safety and performance during seismic events.

4.6.1 SEISMIC ZONE:

As was mentioned previously, it will be considered that the bridge is in Turin (Piemonte), this definition of the seismic zone reflects the likelihood and intensity of the seismic shaking, based on seismicity of the area, geological and tectonic context, and soil conditions.

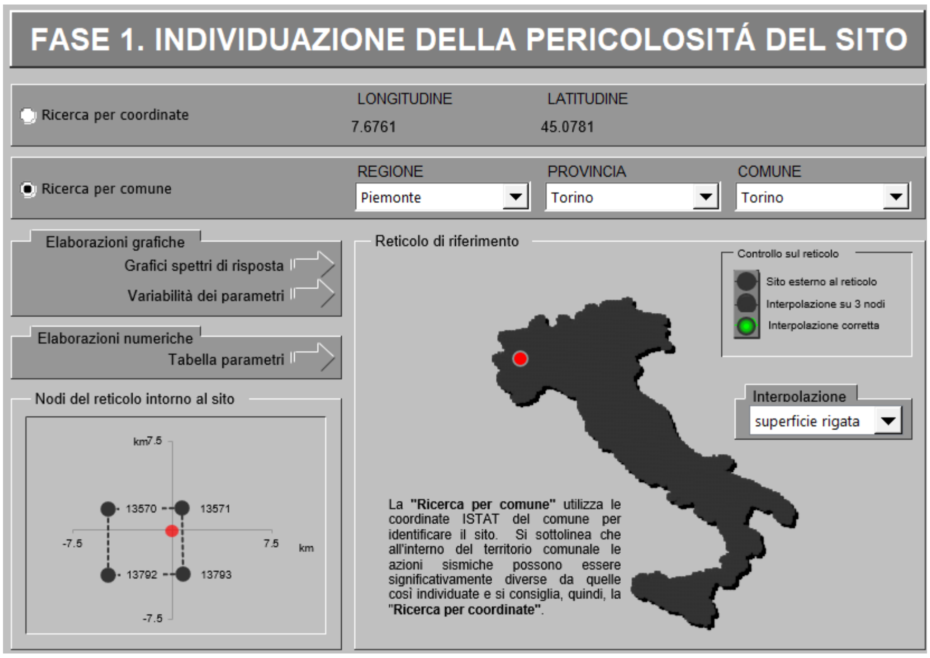


Figure 46. Identification of the seismic zone

According to the NTC18**Error! Reference source not found.**, the likelihood and intensity of the seismic action can be quantified with site specific parameters (see Table 30):

*Table 30. Values of a_g, F_0 and T_c**

SLATO LIMITE	T_R [anni]	a_g [g]	F_o [-]	T_C^* [s]
SLO	30	0.023	2.587	0.177
SLD	50	0.029	2.592	0.195
SLV	475	0.055	2.760	0.272
SLC	975	0.065	2.811	0.287

Where:

- T_R: Return period, it is used to define performance levels for structures.
- a_g : Peak ground acceleration on rock (PGA) for a specific return period.
- F_0: Maximum amplification factor of the soil

- T_c*: Period that separates short-period and long period, this value depends on the soil class.

4.6.2 *Structure service life (V_N)*

Based on chapter 2.4.1 from NTC18**Error! Reference source not found.**, the structure service life (V_N) is the design service life of a structure, the number of years during which the structure must maintain its intended function, safety, and performance under normal usage and environmental conditions.

According to Table 31 from the MANUALE DI PROGETTAZIONE**Error! Reference source not found.**:

Table 31. Nominal life of railway infrastructure

TIPO DI COSTRUZIONE (1)	Vita Nominale V_N [Anni] (1)
OPERE NUOVE SU INFRASTRUTTURE FERROVIARIE PROGETTATE CON LE NORME VIGENTI PRIMA DEL DM 14.01.2008 A VELOCITÀ CONVENZIONALE (V<250 Km/h)	50
ALTRE OPERE NUOVE A VELOCITÀ V<250 Km/h	75
ALTRE OPERE NUOVE A VELOCITÀ V ≥ 250 km/h	100
OPERE DI GRANDI DIMENSIONI: PONTI E VIADOTTI CON CAMPATE DI LUCE MAGGIORE DI 150 m	≥ 100 (2)

(1) – La stessa V_N si applica anche ad apparecchi di appoggio, coprigiunti e impermeabilizzazione delle stesse opere.

(2) - Da definirsi per il singolo progetto a cura di FERROVIE.

For this analysis a value for normal life V_N will be considered as 50 years.

This value of V_N is used together with the use category (Classe d'Uso) to determine the reference return period T_R for seismic design.

4.6.3 Use Category

Based on chapter 2.4.2 from NTC18, with reference to the consequences of an interruption of operations or a possible collapse, buildings and structures are divided based on their importance for safety, social function, and risk in case of failure, especially during events like earthquakes.

For this case and based on the different definitions given in chapter 2.4.2**Error! Reference source not found.** and on Table 32 .

Table 32.Usage coefficients for railway infrastructure

TIPO DI COSTRUZIONE	Classe d'uso(1)	Coefficiente d'uso [CU] (1)
FABBRICATI APPARTENENTI ALL'ELENCO A AI SENSI DEL DPCM 3685/2003	IV	2
GRANDI STAZIONI	IV	2
FABBRICATI APPARTENENTI ALL'ELENCO B AI SENSI DEL DPCM 3685/2003	III	1.5
OPERE D'ARTE DEL SISTEMA DI GRANDE VIABILITÀ FERROVIARIA(2)	III	1.5
ALTRE OPERE D'ARTE, FABBRICATI NON RIENTRANTI NELLE CLASSI D'USO III E IV	II	1

(1) Qualora una costruzione sia interferente con un'altra infrastruttura di cui all'elenco A del DPCM 3685 del 2003 o all'elenco B del DPCM 3685 del 2003 dovrà essere presa in conto la più alta tra la classe d'uso assegnata alla costruzione attraverso la presente tabella e quella dell'infrastruttura con cui si realizza l'interferenza.

(2) Ricadono in classe d'uso IV le opere d'arte nuove ricadenti nelle tratte di nodo di collegamento delle grandi stazioni con il sistema di grande viabilità ferroviaria

It will be assumed that the bridge will be categorized as a **Class II** structure, with a C_u coefficient equal to 1.

4.6.4 *Reference return period (V_R)*

According to chapter 2.4.3**Error! Reference source not found.**, Seismic actions on buildings are assessed in relation to a reference period V_R which is obtained, for each type of building, by multiplying its nominal design life V_N by the use coefficient C_U:

$$V_R = V_N \cdot C_U \tag{31}$$

Knowing previously the values for V_N and C_u the value for the reference return period **V_R =50 years.**

4.6.5 *Return Period (T_R)*

The NTC 2018**Error! Reference source not found.** uses return periods (T_R) to define performance levels for structures. Each level corresponds to an earthquake scenario with a different probability of exceedance during the structure's life, according to chapter 3.2., there are two main performance levels states, SLS (Serviceability Limit State) and ULS (Ultimate Limit State), the probabilities of exceedance in the Pv_R reference period, which can be used to identify the seismic action acting in each of the limit states considered, are reported in Table 33.

Table 33.Probability of exceeding Pv_R as a function of the limit state considered

Stati Limite	**P_{VR}: Probabilità di superamento nel periodo di riferimento V_R**	
Stati limite di esercizio	SLO	81%
	SLD	63%
Stati limite ultimi	SLV	10%
	SLC	5%

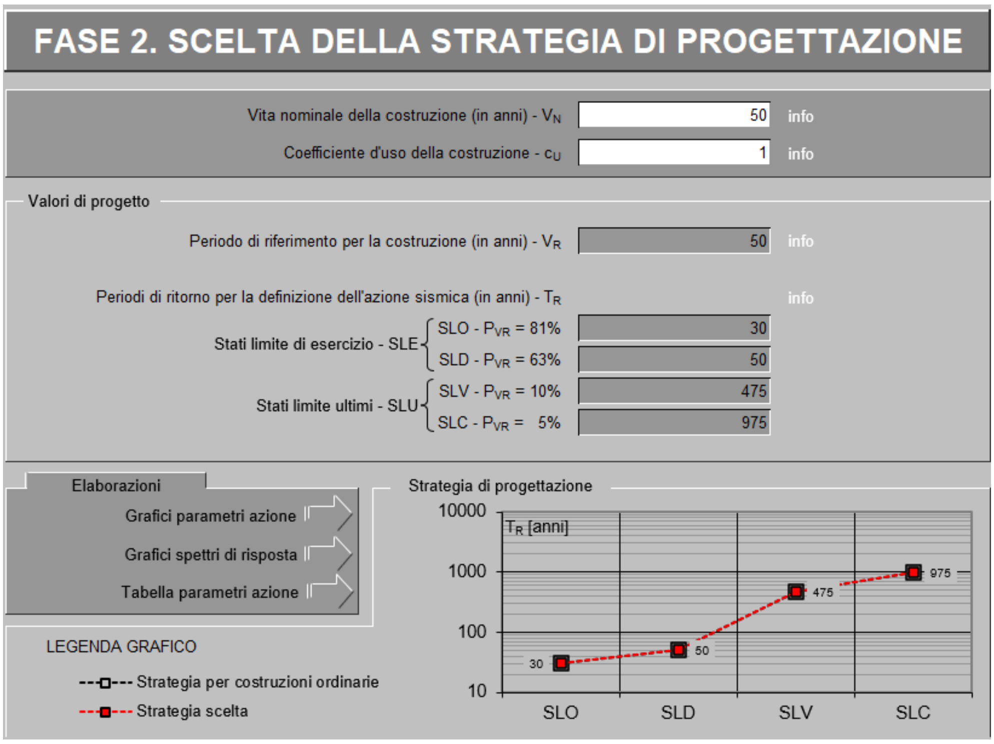


Figure 47.Design Strategy NTC18

4.6.6 SEISMIC ACTION ASSESSMENT:

To determine the seismic forces that will act on the bridge, according to chapter 3.2.2 **Error! Reference source not found.** it is possible to use a simplify approach, based on the classification of the subsoil according to the values of the shear wave propagation velocity (V_s).

4.6.6.1 Soil category

This classification is shown in Table 34

Table 34.Subsoil category

Categoria	Descrizione
A	Ammassi rocciosi affioranti o terreni molto rigidi caratterizzati da valori di $V_{s,30}$ superiori a 800 m/s, eventualmente comprendenti in superficie uno strato di alterazione, con spessore massimo pari a 3 m.
B	Rocce tenere e depositi di terreni a grana grossa molto addensati o terreni a grana fina molto consistenti con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ compresi tra 360 m/s e 800 m/s (ovvero $N_{SPT,30} > 50$ nei terreni a grana grossa e $c_{u,30} > 250$ kPa nei terreni a grana fina).
C	Depositi di terreni a grana grossa mediamente addensati o terreni a grana fina mediamente consistenti con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ compresi tra 180 m/s e 360 m/s (ovvero $15 < N_{SPT,30} < 50$ nei terreni a grana grossa e $70 < c_{u,30} < 250$ kPa nei terreni a grana fina).
D	Depositi di terreni a grana grossa scarsamente addensati o di terreni a grana fina scarsamente consistenti, con spessori superiori a 30 m, caratterizzati da un graduale miglioramento delle proprietà meccaniche con la profondità e da valori di $V_{s,30}$ inferiori a 180 m/s (ovvero $N_{SPT,30} < 15$ nei terreni a grana grossa e $c_{u,30} < 70$ kPa nei terreni a grana fina).
E	Terreni dei sottosuoli di tipo C o D per spessore non superiore a 20 m, posti sul substrato di riferimento (con $V_s > 800$ m/s).

It will be assumed that the **soil category** will be **type C.**

4.6.6.2 Topographic conditions

For simple surface configurations the following classification can be adopted (see Table 35)

Table 35.Topographic categories

Categoria	Caratteristiche della superficie topografica
T1	Superficie pianeggiante, pendii e rilievi isolati con inclinazione media i ≤ 15°
T2	Pendii con inclinazione media i > 15°
T3	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media 15° ≤ i ≤ 30°
T4	Rilievi con larghezza in cresta molto minore che alla base e inclinazione media i > 30°

It will be assumed that the **topographic category** will be **T1.**

4.6.7 SEISMIC ACTION

Seismic action is expressed as an elastic response spectrum, later reduced to a design spectrum.

4.6.7.1 ELASTIC RESPONSE SPECTRUM

The elastic spectrum represents the maximum acceleration a structure would experience during an earthquake if it remained perfectly elastic, according to chapter 3.2.3.2**Error! Reference source not found.**, the elastic acceleration response spectrum is expressed by a spectral shape (normalized spectrum) referred to a conventional damping of 5%, multiplied by the value of the maximum horizontal acceleration a_g on a rigid horizontal reference site. Both the spectral shape and the value of a_g vary with the probability of exceedance in the reference period Pv_R.

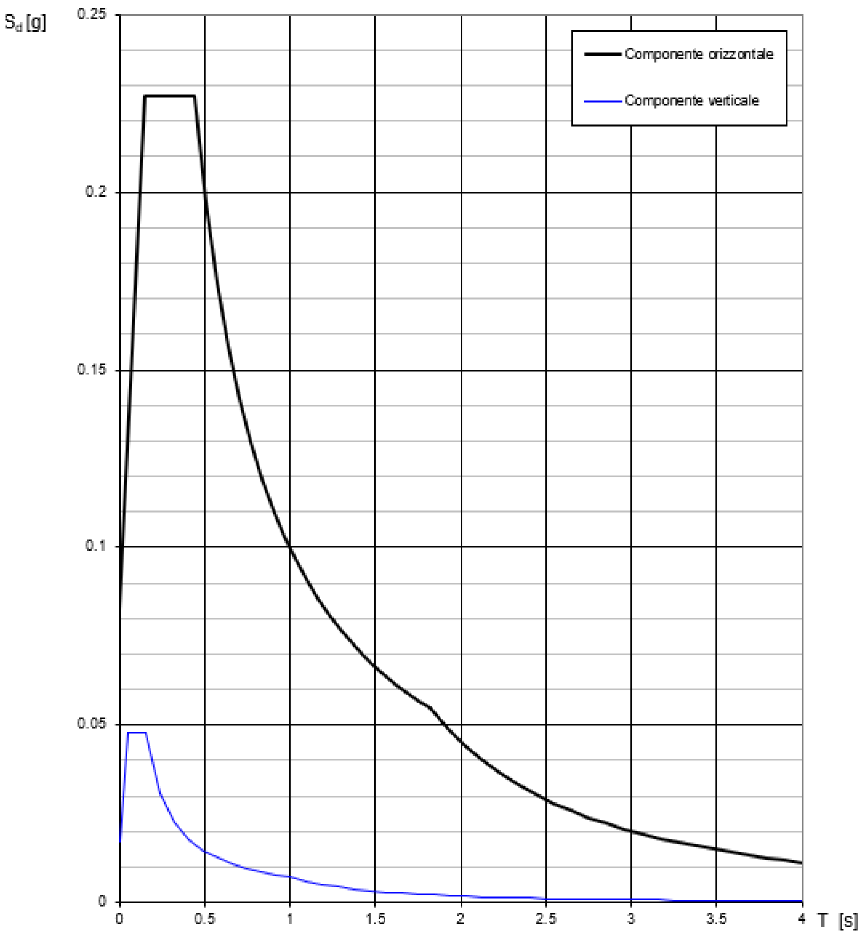


Figure 48. Elastic response spectrum for SLS

4.6.7.2 DESIGN RESPONSE SPECTRUM

The design spectrum is a reduced version of the elastic spectrum that accounts for the structure's ability to dissipate energy through inelastic (ductile) behavior, according to 3.2.3.5**Error! Reference source not found.** the design response spectrum $S_d(T)$ to be used, both for the horizontal components and for the vertical component, is the corresponding elastic response spectrum referred to the probability of exceedance in the reference period Pv_R considered, and taken into account the behavior factor (q) which reduces the spectral values defined in chapter 7.3.1**Error! Reference source not found.**.

4.6.7.2.1 <u>Behavior factor (q)</u>

According to chapter 7.3.1**Error! Reference source not found.**, in the case of dissipative structural behavior, the value of the behavior factor q, to be used for the limit state considered and in the direction considered for the seismic action, depends on the structural typology, its degree of hyperstatic and the design criteria adopted and conventionally takes into account the dissipative capacities of the material.

Structures can be classified as belonging to one typology in a horizontal direction and to another typology in the horizontal direction orthogonal to the previous one, using the corresponding behavior factor for each direction, this value is given by the following equation:

$$q = q_0 \cdot K_R \tag{32}$$

Where:

- q_0: is the basic value of the SLV behavior factor, this value depends on the ductility class, the structural typology, the coefficient λ referred to in chapter 7.9.2.1**Error! Reference source not found.** and the ratio α_u/α_1 between the value of the seismic action for which plasticization occurs in a number of dissipative zones such as to make the structure a mechanism and that for which the first structural element reaches flexural plasticization; the choice of q_0 must be explicitly justified.
- K_R is a factor that depends on the height regularity characteristics of the construction, with a value equal to 1 for constructions regular in height and equal to 0.8 for constructions not regular in height.

According to chapter 7.9.2.1**Error! Reference source not found.** and in order to simplify the analysis it is going to be assumed that the bridge will have a non-dissipative structural behavior, for the two horizontal components of the seismic action, and with this $\mathbf{q_0=1}$.

Also, it will be considered a construction regular in height and with this a value of $\mathbf{K_R=1}$.

Finally, the design response spectrum will be as in Figure 49

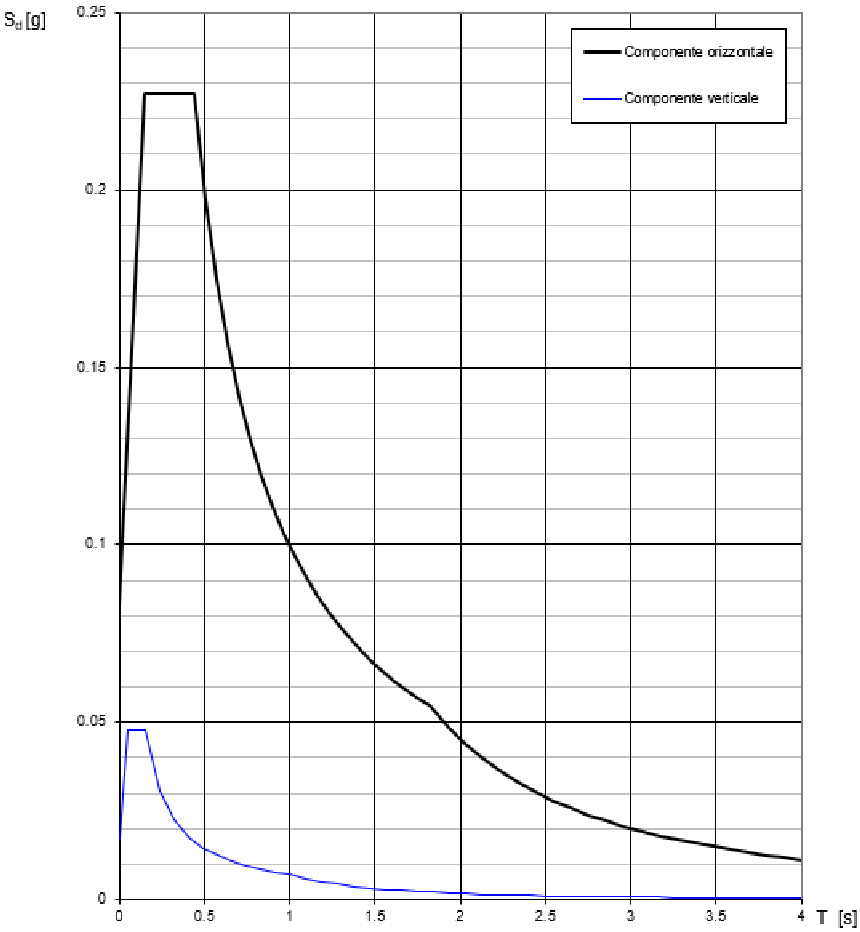


Figure 49. Design response spectrum for ULS

And as it is shown, the two response spectrums are the same.

The parameters for the construction of the horizontal and vertical response spectrum are shown in the following tables (see Table 36 and Table 37).

Table 36. Parameters for Horizontal response spectrum

Limit state	SLU
a_g	0.0559
F_0	2.758
T_c*	0.27
q	1
S	1.5
T_B	0.146
T_C	0.437
T_D	1.824

Table 37.Parameters for Vertical response spectrum

Limit state	SLU
a_g	0.0559
F_v	0.881
T_c*	0.27
q	1
S_s	1
T_B	0.05
T_C	0.15
T_D	1

5

|DYNAMIC EFFECTS:

According to chapter 2.5.1.4.2**Error! Reference source not found.**, the stresses and deformations produced in the bridge structures by the static application of the load trains must be increased to account for the dynamic nature of the passage of trains.

Based on 2.5.1.4.2.4**Error! Reference source not found.**, the following are the requirements for determining whether a static or dynamic analysis is needed are indicated Figure 50, where:

- V: is the maximum speed of the line [km/h]
- L is the span length for a simply supported span, to be taken equal to LΦ for continuous bridges [m], with LΦ defined in paragraph 2.5.1.4.2.5.3**Error! Reference source not found.**.
- n_0 is the first natural bending frequency of the bridge under permanent loads [Hz]
- n_t is the first natural torsional frequency of the bridge under permanent loads [Hz].

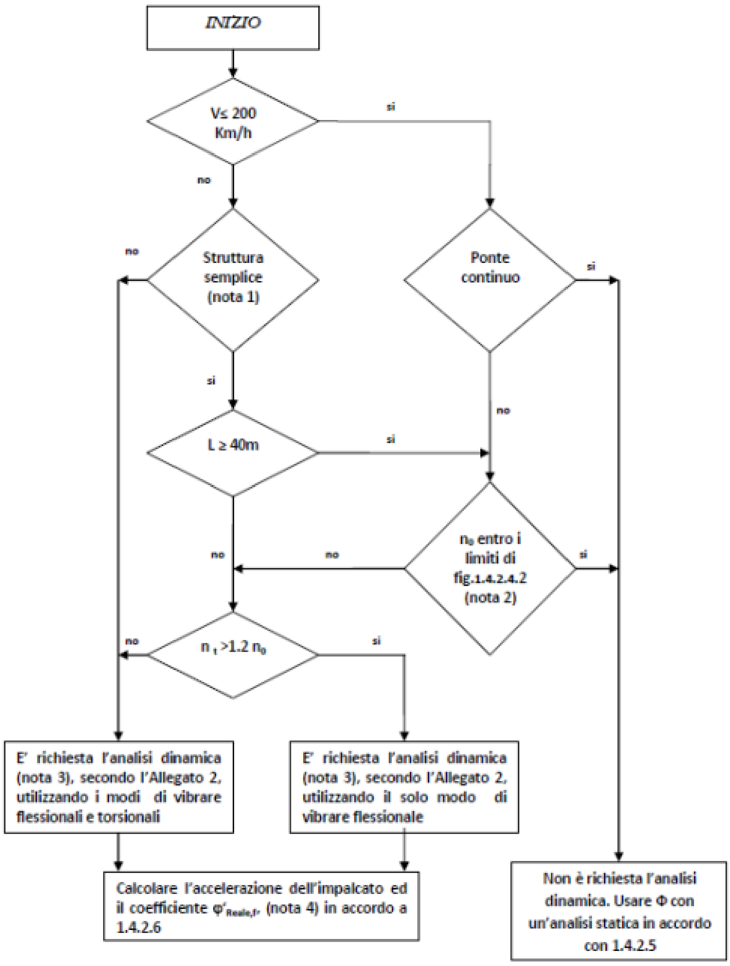


Figure 50.Flowchart for determining whether dynamic analysis is required

For a simply supported beam, the first bending frequency and the first torsional frequency can be estimated using the formulas (see equation (33) and (34)).

$$n_0 = \frac{17.75}{\sqrt{\delta_0}} \; [Hz] \tag{33}$$

$$n_T = \frac{1}{2L}\sqrt{\frac{GJ_p}{m\rho^2}} \; [Hz] \tag{34}$$

Where:

- δ_0: represents the deflection, expressed in mm, evaluated at midspan and due to permanent loads.
- GJ_p is the torsional stiffness of the cross-section.
- m is the average mass per unit length of the bridge.
- ρ is the radius of gyration."

The first bending frequency must be between the following limits according to NOTE 2 in paragraph 2.5.1.4.2.4.

The upper limit of n_0 is characterized by:

$$n_0 = 94.76\, L^{-0.748} \tag{35}$$

and the lower limit is given by

$$n_0 = \frac{80}{L} \text{ for 4m < L< 20 m} \tag{36}$$

$$n_0 = 23.58\, L^{-0.592}\, for\ 20m\ < \ L < \ 100\ m \tag{37}$$

Where:

- n_0: Is the first bending frequency
- L: is the span length for a simply supported span, to be taken equal to LΦ for continuous bridges [m].

According to these limits it is possible to use the Figure 51 to conclude if it is necessary or no to do a dynamic analysis according to the flowchart given in Figure 50.

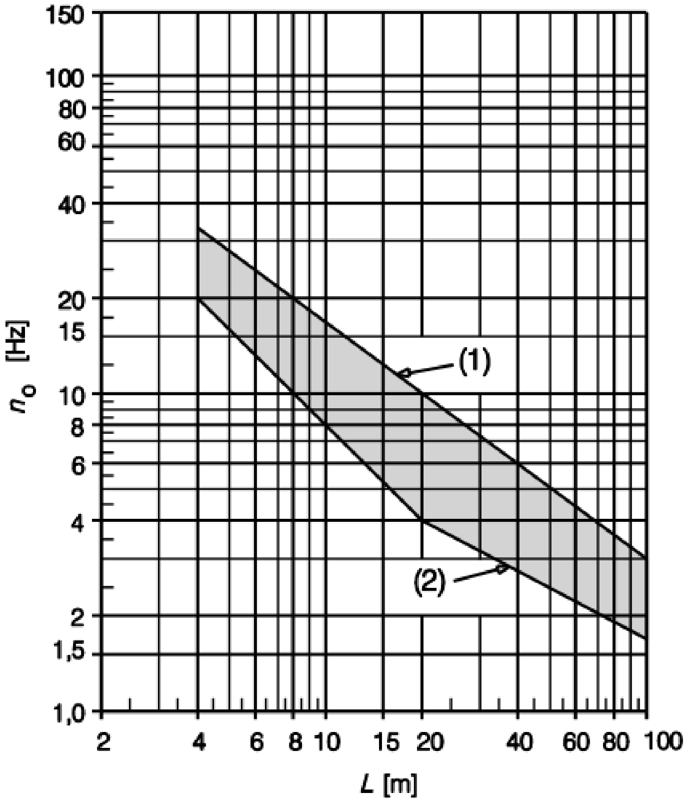


Figure 51.Limits of the natural frequency of the bridge n_0 [Hz] as a function of L (m)

The value of the first bending frequency (n_0) can be estimated in two different ways:

By equation (33), it is necessary to determine the maximum deflection (δ_0) in mm for the permanent loads, this value of deflection is given by the 3D model.

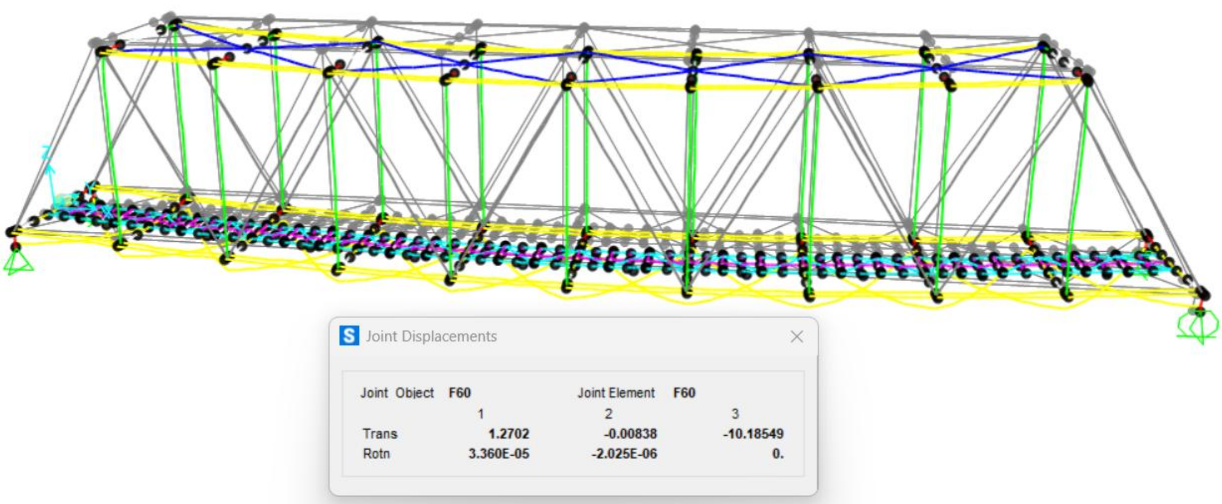


Figure 52. Maximum displacement due permanent loads

According to Figure 52 the maximum displacement due to the permanent loads is approximately equal to $\delta_0 = 10.18$mm.

Now replacing in equation (44), the first bending frequency $n_0 = 5.56$.[Hz].

In the other hand is possible to get the value of the first bending frequency n_0 directly from the modal analysis using the 3D-model.

In this case it is necessary to identify the first bending modal shape, for this case it occurs at the 3th mode.

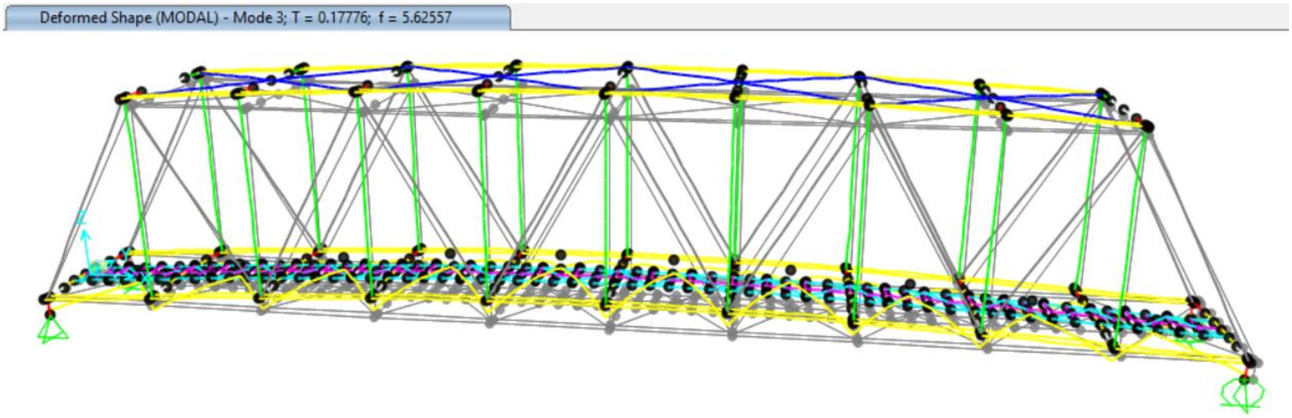


Figure 53. Frequency at the first bending mode shape

For this case the natural first frequency at the first bending mode is equal to n_0 = 5.62.[Hz].

Using equations (46) and (54) it is possible to plot the range where the bridge will be.

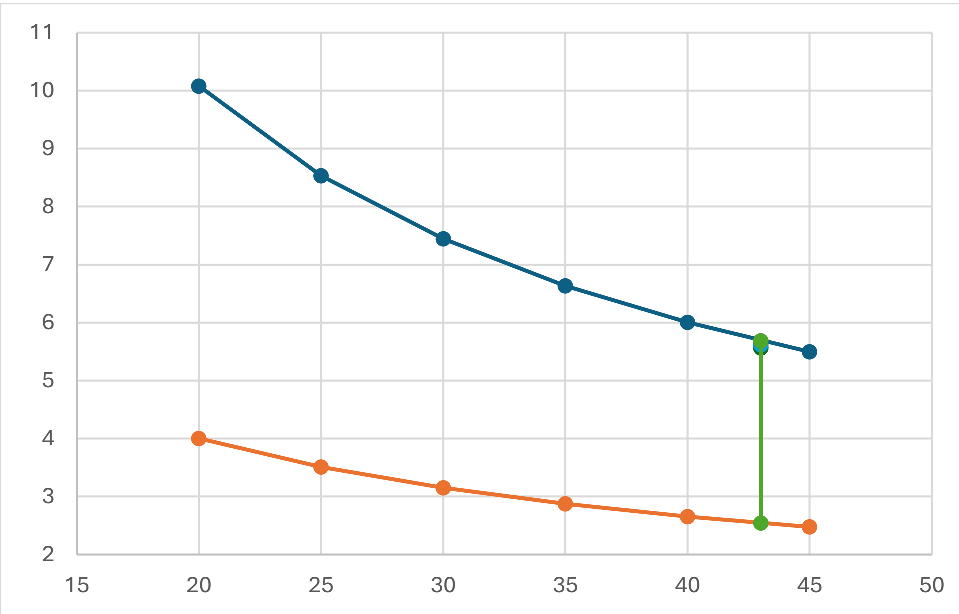


Figure 54. Limits of the natural frequency of the bridge n_0 [Hz, L ranged between 20 and 40 meters

As is shown, both results are really close between them, in both cases the first natural frequency n_0 is located between the limits and with this it is possible to follow the path indicated with red arrows in Figure 55.

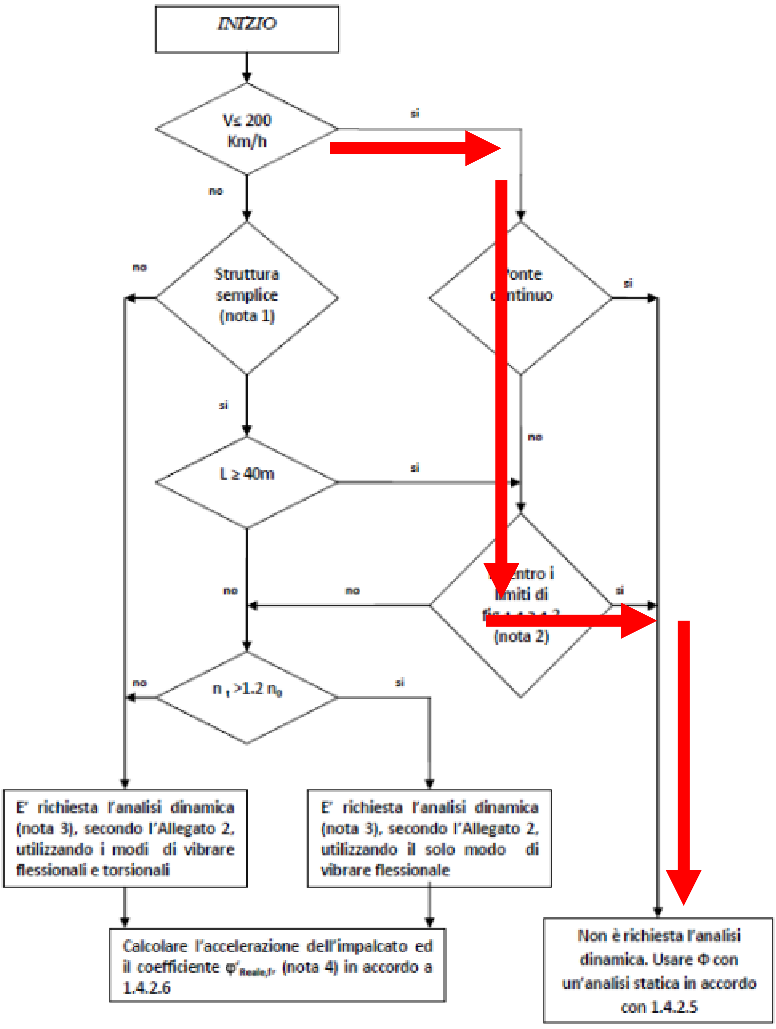


Figure 55.Favored path

Following the path described above it is possible to determine the dynamic coefficients based on 2.5.1.4.2.5**Error! Reference source not found.**.

The dynamic coefficients Φ apply to the theoretical load models LM71, SW/0, and SW/2 and account for the dynamic amplification of stresses, deformations, and the effects of structural vibrations.

The dynamic amplification coefficients ϕ, which increase the intensity of the theoretical load models, are taken as ϕ_2 or ϕ_3 depending on the level of line maintenance. In this case, the values corresponding to 'lines with a normal standard of maintenance' are conservatively adopted, even though the bridge undergoes maintenance interventions on an annual basis.

$$\phi_2 = \frac{1.44}{\sqrt{L_\emptyset} - 0.20} + 0.82 \tag{38}$$

Equation (55 is given for lines with a high maintenance standard (such as, for example, high-speed/high-capacity lines), with:

$$1.00 < \phi_2 < 1.67 \tag{39}$$

While for lines with a normal maintenance standard ϕ_3 is given by:

$$\phi_3 = \frac{2.16}{\sqrt{L_\emptyset} - 0.20} + 0.73 \tag{40}$$

With:

$$1.00 < \phi_3 < 2.00 \tag{41}$$

Where in both cases $L_\emptyset$ represents the characteristic length given in 2.5.1.4.2.5.3**Error! Reference source not found.**.

Additionally, it is indicated in 2.5.1.4.2.5.2**Error! Reference source not found.** that for steel bridges with direct track structures, an additional adjustment coefficient for the dynamic amplification must be considered β (introduced to account for the greater dynamic increase due to this particular type of track structure). This coefficient varies according to the characteristic length $L_\emptyset$ of the element, given by:

$$\beta = 1.00 \text{ if } L_\emptyset < 8\text{m or } L_\emptyset > 90\text{ m} \tag{42}$$

$$\beta = 1.10 \text{ if } 8\text{m} < L_\emptyset < 90\text{ m} \tag{43}$$

5.1 DETERMINATION OF THE LENGTH $L_{\emptyset}$:

The characteristic lengths $L_{\emptyset}$ to be used are provided in Table 38.

If Table 38.does not specify the value of $L_{\emptyset}$ it is recommended that the characteristic length be taken as equal to the length of the influence line of displacement for the element in analysis.

If the stresses in a structural element result from different effects, each relating to separate structural behaviors, it is recommended that each effect be calculated using the appropriate characteristic length.

For Table 38, the following considerations are going to be used:

5.1.1 STEEL BRIDGE DECK WITHOUT BALLAST (FOR LOCAL STRESSES)

For this, there are defined the following elements:

a. Rail supports (stringers):

Defined as:

- An element of a grid deck, for this element the characteristic length will be 3 times the spacing of the cross girders
- a simple supported element. For this element the characteristic length will be distance between the cross girders plus 3m.

b. Intermediate cross beams:

The characteristic length will be 2 times the span of the cross beams.

c. End cross beams:

The characteristic length will be most unfavorable case between:

- Span of the cross beam
- 3.60 m

5.1.2 *PRINCIPAL BEAMS:*

- Beams and simply supported slabs (including large slabs with embedded beams):

The characteristic length will be the span in the direction of the main beams.

Table 38.Characteristic length L_ϕ

IMPALCATO DI PONTE IN ACCIAIO SENZA BALLAST (PER TENSIONI LOCALI)		
3	3.1 Sostegni per rotaie (longherine) - come elemento di un grigliato - come elemento semplicemente appoggiato	 3 volte l'interasse delle travi trasversali distanza fra le travi trasversali + 3 m
	3.2 Sostegni per rotaie a mensola (longherine a mensola) per travi trasversali di estremità	caso più sfavorevole tra: - 3,60 m - Φ_3=2,00
	3.3 Travi trasversali intermedie 3.4 Travi trasversali d'estremità	2 volte la luce delle travi trasversali caso più sfavorevole tra: - lunghezza della trave trasversale - 3,60 m
TRAVI PRINCIPALI		
5	5.1Travi e solette semplicemente appoggiate (compresi i solettoni a travi incorporate)	Luce nella direzione delle travi principali
	5.2 Travi e solette continue su n luci, indicando con: $L_m = 1/n \cdot (L_1+L_2+.....+L_n)$	$L_\phi = k \cdot L_m$ dove: n = 2 - 3 - 4 - ≥ 5 k = 1,2 - 1,3 - 1,4 - 1,5
	5.3 Portali: - a luce singola	 da considerare come trave continua a tre luci (usando la 5.2 considerando le altezze dei piedritti e la lunghezza del traverso)
	- a luci multiple	da considerare come trave continua a più luci (usando la 5.2 considerando le altezze dei piedritti terminali e la lunghezza di tutti i traversi)
	5.4 Solette ed altri elementi di scatolari per uno o più binari (sottovia di altezza libera ≤ 5,00 m e luce ≤ 8 m Per gli scatolari che non rispettano i precedenti limiti vale il punto 5.3, trascurando la presenza della soletta inferiore e considerando un coefficiente riduttivo pari a 0,9, da applicare al coefficiente Φ.	Φ_2 =1,20 = Φ_3 = 1,35
	5.5 Travi ad asse curvilineo, archi a spinta eliminata archi senza riempimento.	metà della luce libera
	5.6 Archi e serie di archi con riempimento	due volte la luce libera
	5.7 Strutture di sospensione (di collegamento a travi di irrigidimento)	4 volte la distanza longitudinale fra le strutture di sospensione

5.2 Results:

The characteristic length for each element will be shown in Table 39 below:

Table 39. Characteristic Length $L_{\emptyset}$

ITEM	Real Length (m)	Characteristic Length ($L_{\emptyset}$)(m)
Bottom and Top chords	43	43
Stringers	4.3	7.30
Intermediate cross beams	5.6	8.60
End cross beams	5.6	3.60

With these values it is possible to determine the values for ϕ_2, ϕ_3 and β. according to equations (38),(40),(42) and (43).

Table 40. Dynamic amplification coefficients φ_2, φ_3 and β

ITEM	ϕ_2	ϕ_3	β
Bottom and Top chords	1.04	1.06	1.1
Stringers	1.39	1.59	1.0
Intermediate cross beams	1.34	1.52	1.1
End cross beams	1.66	2.00	1.0

The maintenance of the bridge will be treated as standard. Accordingly, and in reference to Section 5.2.2.2.3 [4], only the value calculated for ϕ_3 will be considered in this case.

6

|VARIABLE LOADS (TRAFFIC LOADS) IN SAP2000

According to chapter 2.5.1.4**Error! Reference source not found.**, the vertical actions associated with railway convoys are indicated, through the definition of load models. By the following "theoretical" load models defined as: LM71, SW/0 and SW/2, and conventional "real train" it is possible to consider the traffic loads.

6.1 Loading trains

Based on paragraph 2.5.1.4.1.1**Error! Reference source not found.**, Vertical loads are defined by means of load models; in particular, two distinct load models are provided: the first one representing normal traffic (LM71 load model), the second one representing heavy traffic (SW load model).

6.2 LM71 load model

According to chapter 5.2.2.2.1.1 from NTC18**Error! Reference source not found.**,

The LM71 load model is a static load model used to represent the standard vertical loads exerted by railway traffic on a bridge, this model can be represented according to Figure 56.

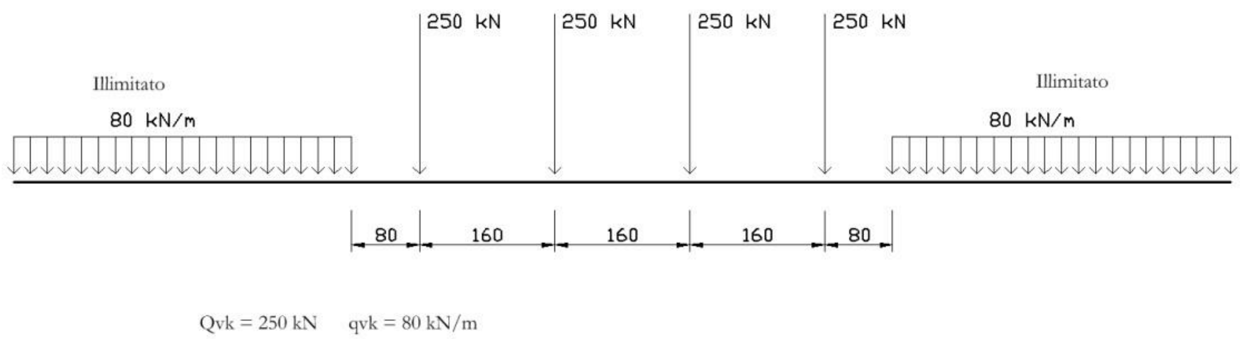


Figure 56. LM71 Load Model

The model consists of 4 Axle loads, each axle exerts 250 kN and they are spaced 1.60 m part, and a uniformly distributed load of 80 kN/m, applied along the track, starting from 0.8 m away from each end of the axle group.

To simulate an uneven wheel loading it will be considered a side offset (eccentricity) of s/18, where s=1435 mm (standard track gauge), this must be placed in the most unfavorable direction for design.

It will be considered an amplification factor (D) equal to 1.1, this will be applied for ordinary railway lines to increase the load and generate a more conservative model.

This load model is applied as a moving load in the 3D-model, as it is shown in Figure 57.

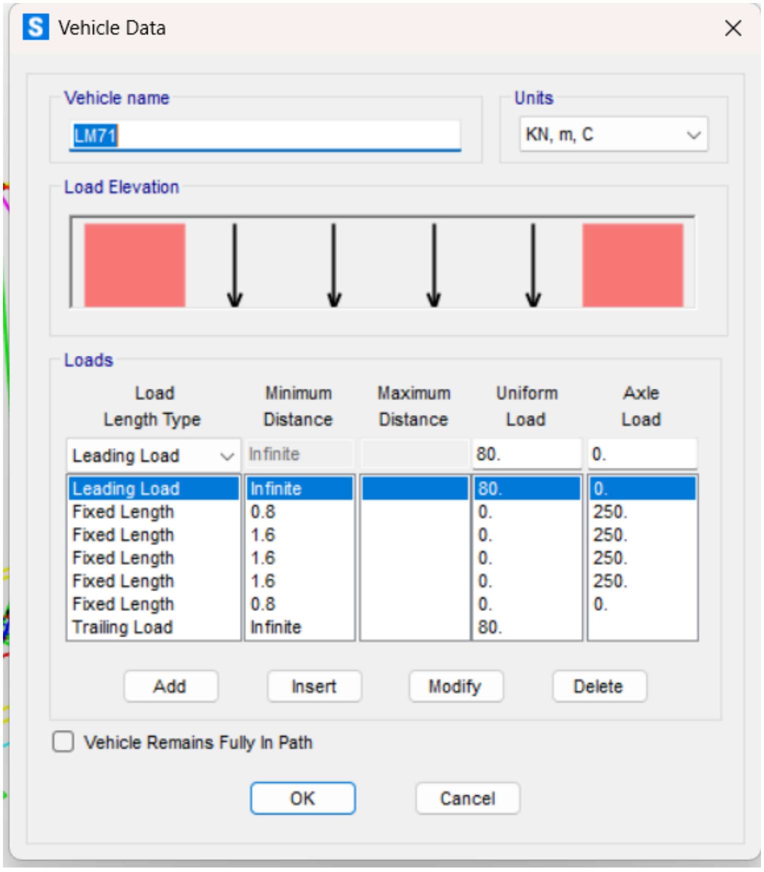


Figure 57. LM71 load model - 3D Model

The local load (250 kN) indicated before will be distrusted according to paragraph 5.2.2.2.1.4 **Error! Reference source not found.**, this addresses how local loads (Q_{vi}) from railway traffic are distributed locally through the sleepers and the superstructure elements.

This distribution will be done according to Figure 58.

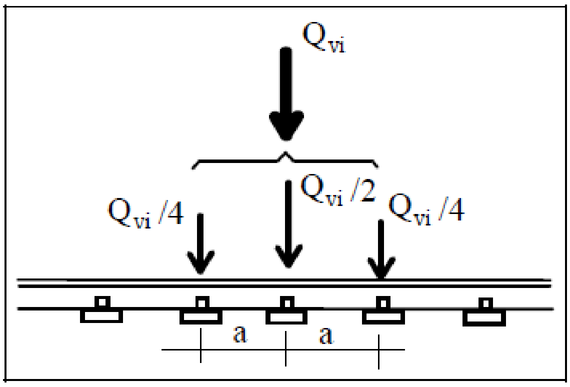


Figure 58. Longitudinal distribution of axial loads

Each axle load Q_{vi} is spread longitudinally over three consecutive sleepers according to the following proportions showed in Figure 58.

Finally, the load model LM71 will be represented as it is shown in Figure 59.

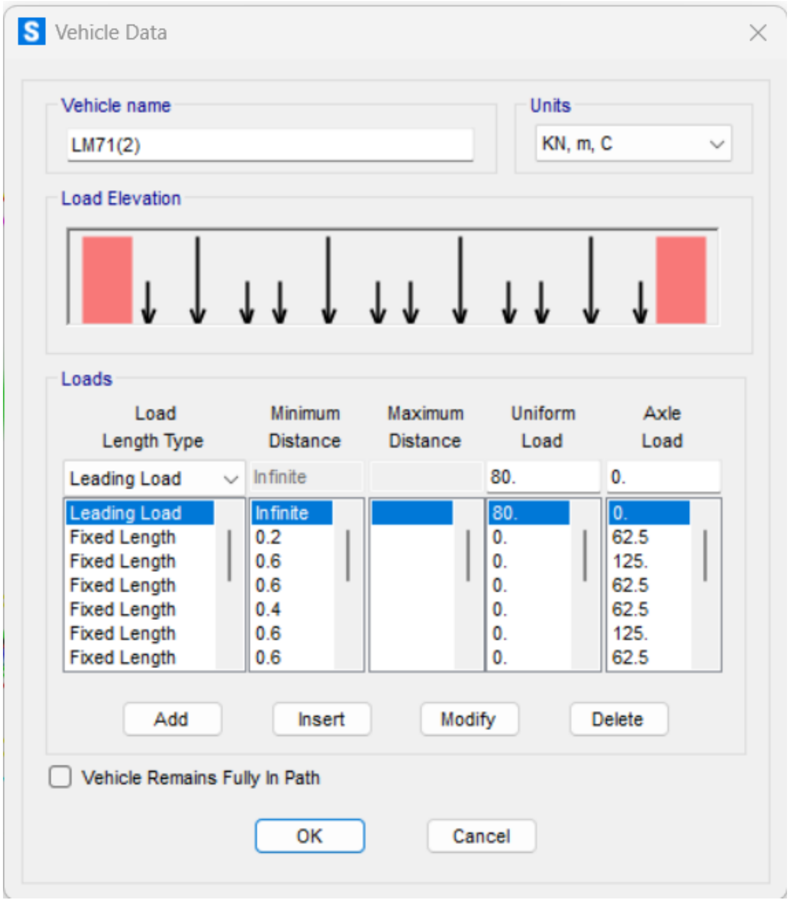


Figure 59. Distribution of LM71 load model - 3D Model

It is possilbe to verify if the moving load in the 3D-model it was correctly applied, to do this it is possible to cosider a simple supported beam with the load distribution given in Figure 58.

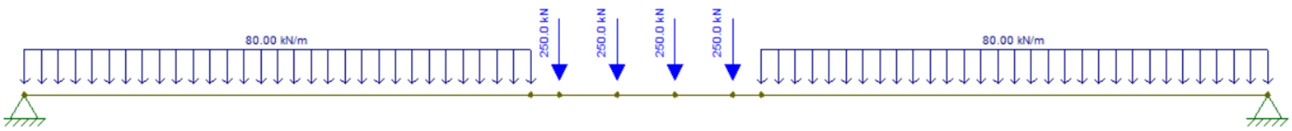


Figure 60. Simple supported beam - LM71 load model

With the beam shown in Figure 60, it is possible to determine the acting bending moment and with this value compare with the one obtained in the 3D model.

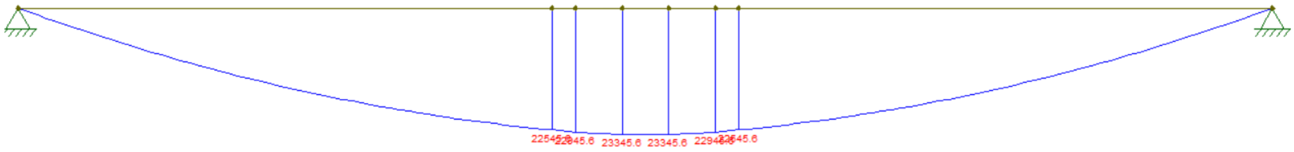


Figure 61. Bending moment acting on the simple supported beam.

The maximum value obtained in the center of the beam is 23345.6 kN·m.

Now according to the axial forces resulting from the same load model LM71 on the 3D-model, it is possible to determine the maximum bending moment and with this compare it with the one obtained in the simple supported beam.

To do this, it will be considered the maximum axial force acting on the bridge, the axial forces on the bottom and top chords and on the transversal beam.

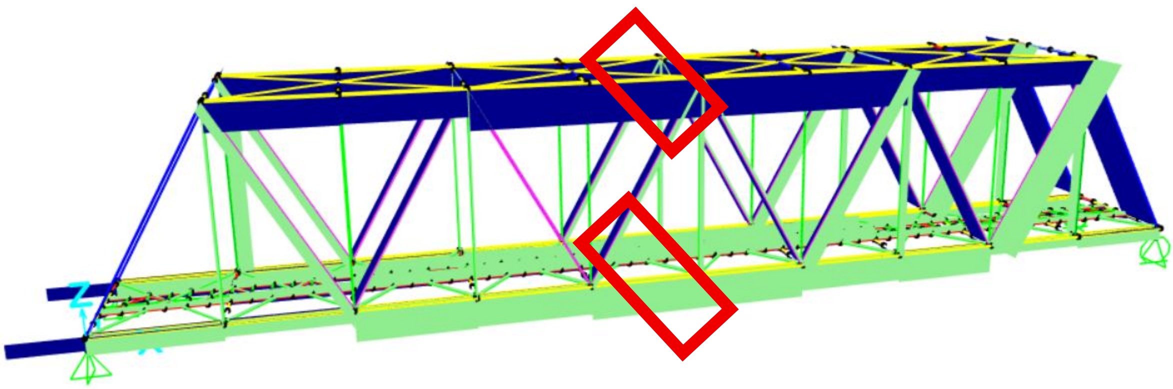


Figure 62. Axial forces LM71 load model

The values on the bottom chords are:

F_{1B}: 694.99 kN

F_{2B}: 671.62 kN

F_{3B}: 676.37 kN

F_{4B}: 701.54 kN

The values on the top chords are:

F_{1T}: 748.25 kN

F_{2T}: 738.17 kN

F_{3T}: 745.57 kN

F_{4T}: 758.76 kN

The values on the rail profiles are:

F_{1R}: 249.23 kN

F_{2R}: 264.47 kN

The approximately total axial load acting on the bottom and top part of the bridge will be equal to:

Bottom axial force:

$$F_B=694.99+671.62+676.37+701.54+249.23+264.47 = 3213.22 \text{ kN}$$

Top axial force:

$$F_T=748.25+738.17+745.57+758.76+44.81+44.89= 3080.45 \text{ kN}$$

By approximation, it is possible to get the medium value between the top and bottom force equal to F_p= 3146.8 kN.

The total height of the bridge in the 3D Model is 8.23 m, and with this the bending moment for the LM71 load model will be:

$$M_{LM71} = 3146.8\text{kN} \cdot 8.23\text{m} =25898.4.\ \text{kN} \cdot \text{m}$$

It is important to remark that the value obtained by the simple supported beam has not into account the amplification factor (D) equal to 1.1 that it is already applied in the 3D model, for this the value in Figure 61 must be multiplied by 1.1:

$$M_B = 23345.6 \text{ kN} \cdot \text{m} \cdot 1.1 =25680.16.\ \text{kN} \cdot \text{m}$$

Finally, and according to the ‘real’ value obtained by the simple supported beam, the difference between the ‘real’ value and the measure one is approximately 0.84%.

With this it is possible to consider that the distribution of the moving load LM71 made by the 3D model is ok.

6.3 SW load model:

According to chapter 5.2.2.2.1.2 from NTC18**Error! Reference source not found.**, the SW load model represents the different levels of railway traffic loading:

- SW/0, represents the standard traffic effects
- SW/2, represents the heavy traffic effects.

These two load models will be given by Figure 63.

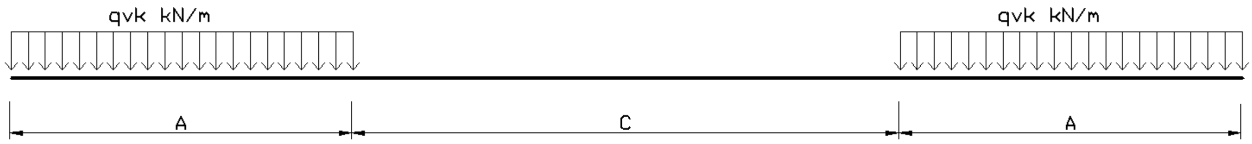


Figure 63. SW Load Model

And its values are given in Table 41.

Table 41. SW load model values

Tipo di carico	Q_{vk} [kN/m]	A [m]	C [m]
SW/0	133	15,00	5,30
SW/2	150	25,00	7,00

In standard railway design, it is necessary to apply an adaptation factor (D).

- SW/0, D = 1.1
- SW/2, D= 1.0.

The SW/0 and SW/2 are defined in the 3D model according to Figure 64 and Figure 65.

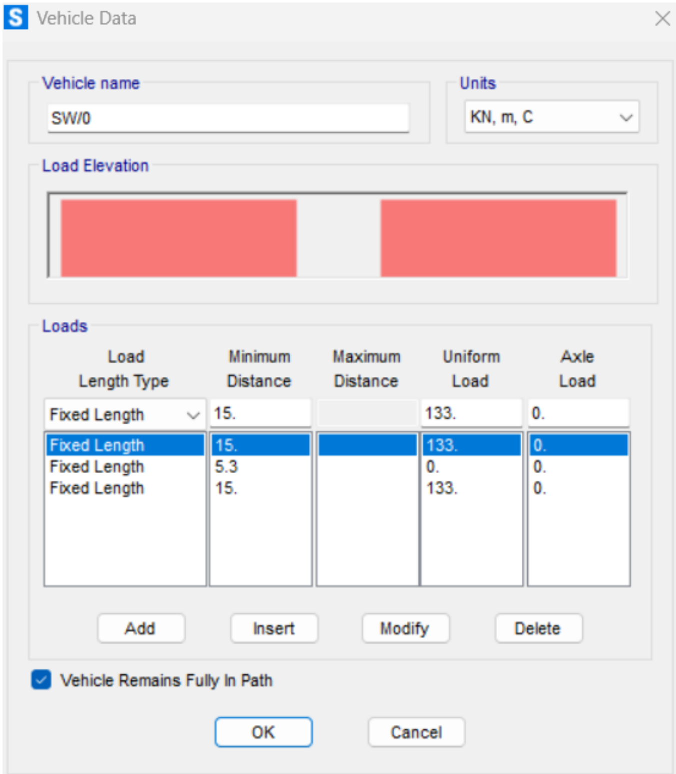


Figure 64. SW/0 load model - 3D model

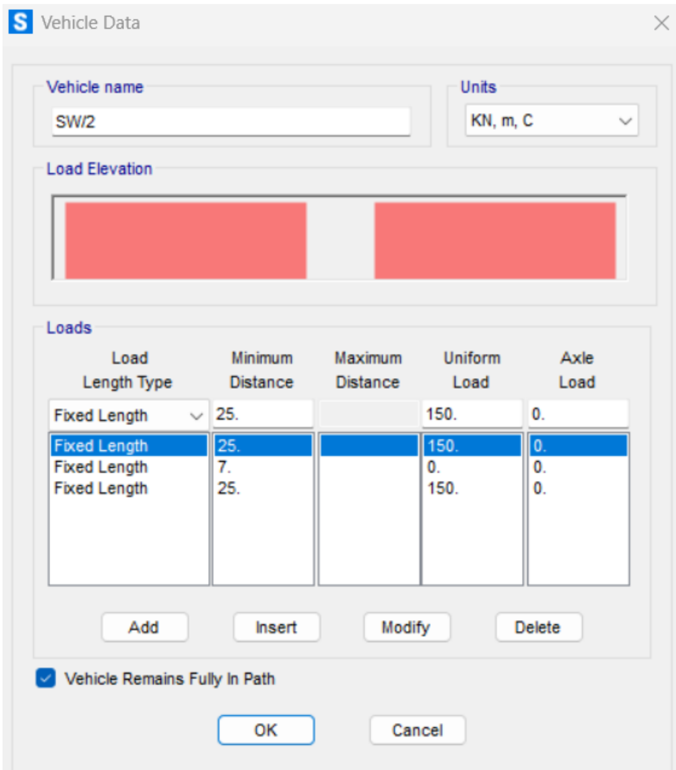


Figure 65.SW/2 load model - 3D model

In the same way as it was made for the LM71 load model, the verification for the SW/0 load model will be made. It will be considered a simple supported beam with the load model SW/0 according to Table 41.

Figure 66. Simple supported beam - SW/0 load model

From this it is possible to get the value for the bending moment equal to 22851.3 kN · m.

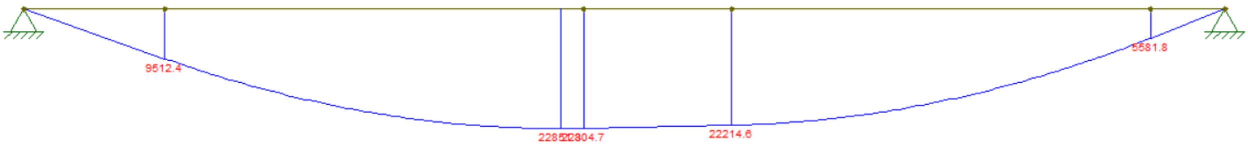


Figure 67. Bending moment acting on the simple supported beam.

Now according to the axial forces resulting from the same load model SW/0 on the 3D-model, it is possible to determine the maximum bending moment and with this compare it with the one obtained in the simple supported beam.

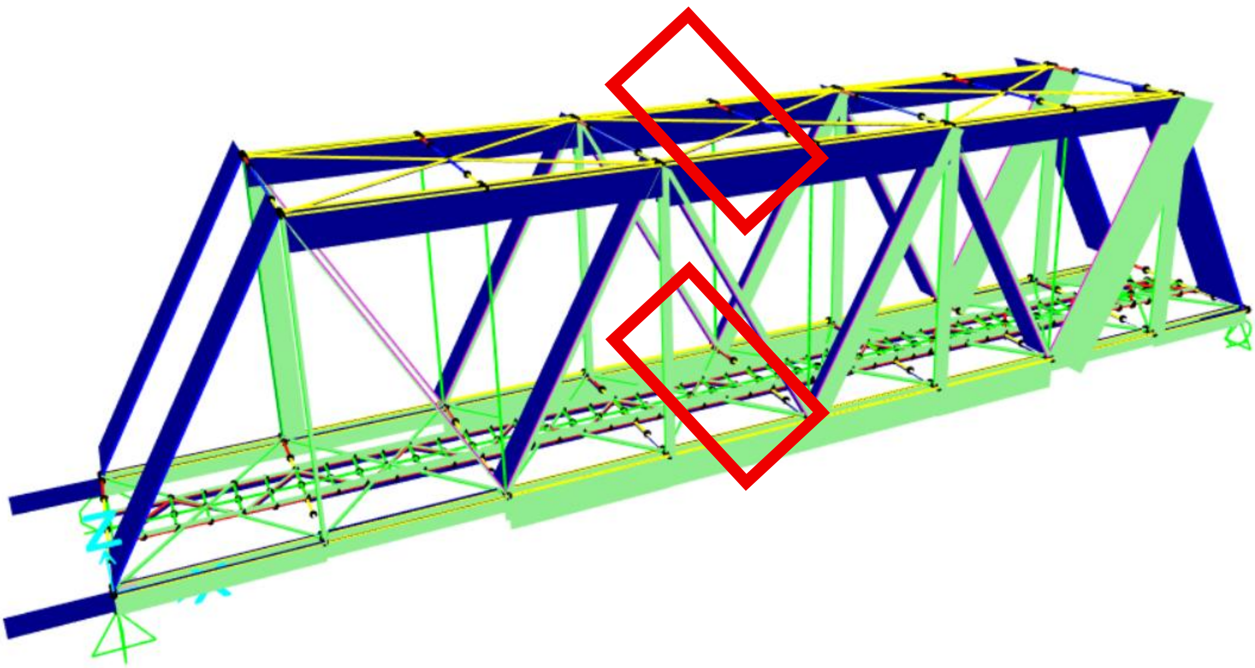


Figure 68. Axial forces SW/0 load model

The values on the bottom chords are:

F_{1B}: 679.27 kN

F_{2B}: 645.44 kN

F_{3B}: 640.68 kN

F_{4B}: 686.17 kN

The values on the top chords are:

F_{1T}: 753.61 kN

F_{2T}: 747.32 kN

F_{3T}: 754.30 kN

F_{4T}: 761.36 kN

The values on the rail profiles are:

F_{1R}: 269.06 kN

F_{2R}: 264.43 kN

The approximately total axial load acting on the bottom and top part of the bridge will be equal to:

Bottom axial force:

$$F_B = 679.27 + 645.44 + 640.68 + 686.17 + 269.06 + 264.43 = 3185.05 \text{ kN}$$

Top axial force:

$$F_T = 753.61 + 747.32 + 754.30 + 71.36 + 45.46 + 44.51 = 3106.56 \text{ kN}$$

By approximation, it is possible to get the medium value between the top and bottom force equal to F_p = 3145.80 kN.

The total height of the bridge in the 3D Model is 8.23 m, and with this the bending moment for the LM71 load model will be:

$$M_{LM71} = 3145.80 \text{ kN} \cdot 8.23\text{m} = 25889.97. \text{ kN} \cdot \text{m}$$

It is important to remark that the value obtained by the simple supported beam has not into account the amplification factor (D) equal to 1.1 that it is already applied in the 3D model, for this the value in Figure 61 must be multiplied by 1.1:

$$M_B = 22851.3 \text{ kN} \cdot \text{m} \cdot 1.1 = 25136.43. \text{ kN} \cdot \text{m}$$

Finally, and according to the 'real' value obtained by the simple supported beam, the difference between the 'real' value and the measure one is approximately 2.99%.

With this it is possible to consider that the distribution of the moving load SW/0 made by the 3D model is ok.

For load model SW/2, it will be assumed that the error is approximately the same value meaning that the 3D model is working well.

In addition to these load models (LM71, SW/0 and SW/2) and according to paragraph 5.2.2.1.3 from NTC18**Error! Reference source not found.** it will be considered a load train called “Treno scarico”(unloaded train), this will be represented as an uniform load equal to 10 kN/m (see Figure 69).

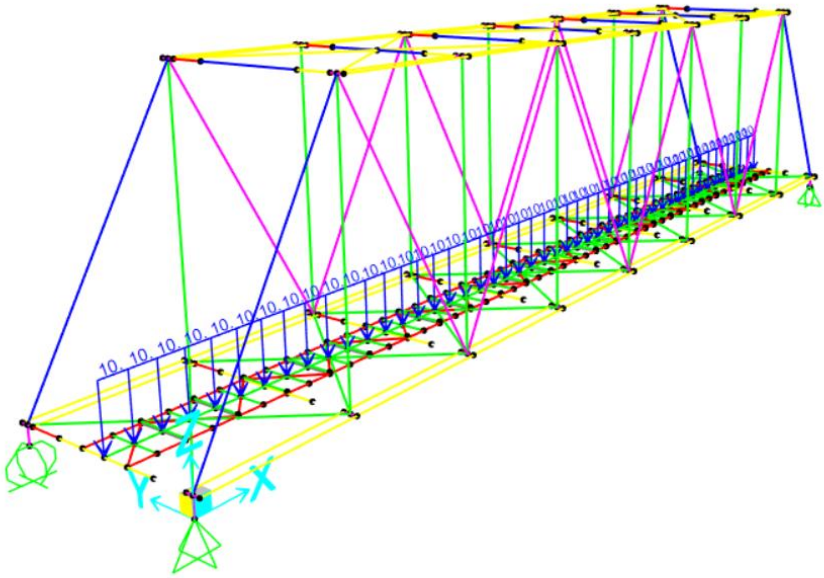


Figure 69. Unloaded train - 3D Model

However, In this analysis the unloaded train case will be neglected and not be considered, because its value itself is lower than the previous load cases (LM71, SW/0 and SW/2).

6.4 VARIABLE HORIZONTAL FORCES

According to chapter 2.5.1.4.3.2**Error! Reference source not found.**, the horizontal forces must be considered in conjunction with the vertical loads. The horizontal forces generated on railway bridges when trains either start moving or brake.

Here it is important to indicate that as the bridge is in a straight line there will not be centrifugal force acting on the bridge (see 2.5.1.4.3.1**Error! Reference source not found.**).

The horizontal action will be given by the following.

6.4.1 Lateral action (Nosing):

There will be as it is mentioned in paragraph 5.2.2.3.2 **Error! Reference source not found.**, a lateral action acting on the bridge (Nosing), the lateral force induced by the meandering is considered as a concentrated force acting horizontally, applied to the top of the highest rail, perpendicular to the axis of the track.

The characteristic value of this force will be assumed equal to Q_{sk}=100 kN. This value must be multiplied by α (if $\alpha>1$), but not by the dynamic increment coefficient Φ. This lateral force must always be combined with vertical loads.

This force will be multiplied by an adaptation coefficient equal to $\alpha= 1.1$.

6.4.2 Acceleration and braking actions:

According to paragraph 5.2.2.3.3 **Error! Reference source not found.**, The braking and starting forces act on the top of the track, in the longitudinal direction of the same. These forces are to be considered uniformly distributed over track length (L) determined to obtain the most severe effect on the structural element considered.

The characteristic values to be considered are the following.

6.4.2.1 *Acceleration:*

$$Q_{la,k} = 33\ [kN/m] \cdot\ L\ [m]\ \leq 1000\ kN \tag{44}$$

Equation (44) applies for load models LM71, SW/0 and SW/2.

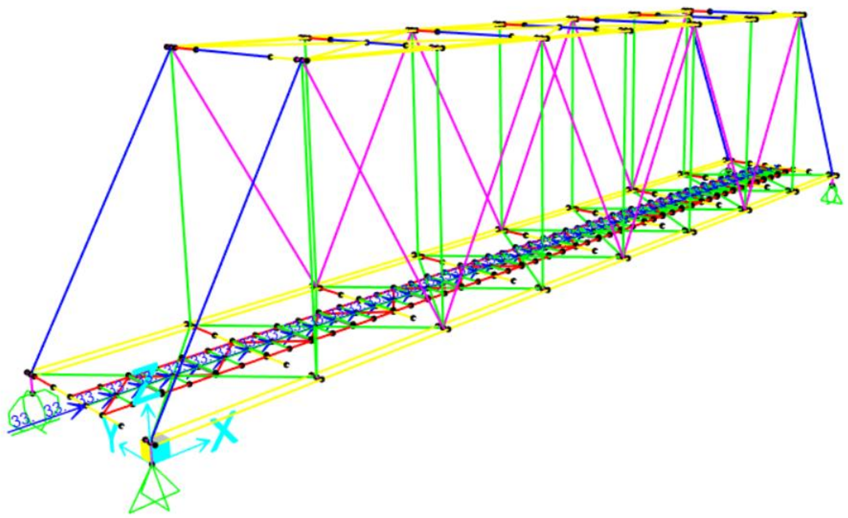


Figure 70. Acceleration - 3D Model

6.4.2.2 *Breaking:*

$$Q_{lb,k} = 20\ [kN/m] \cdot\ L\ [m]\ \leq 6000\ kN \tag{45}$$

$$Q_{lb,k} = 35\ [kN/m] \cdot\ L\ [m]\ \leq 6000\ kN \tag{46}$$

Equation (45) applies for load models LM71, SW/0, while Equation (44) applies for load model SW/2.

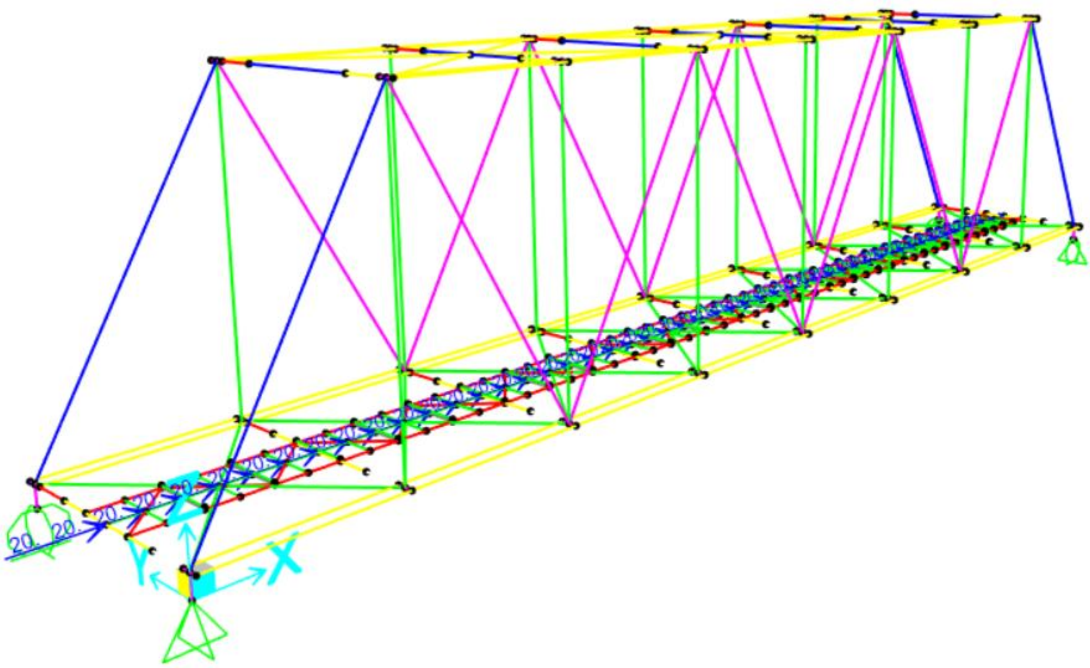


Figure 71. Braking force LM71 and SW/0 - 3D Model

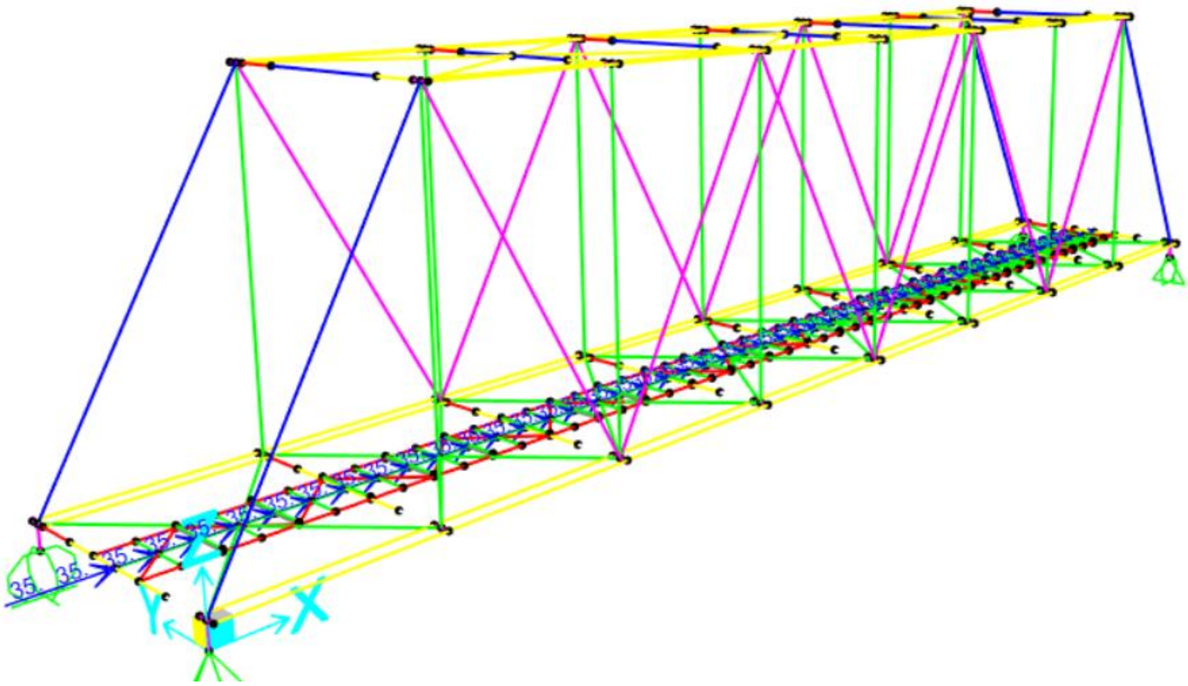


Figure 72. Braking force SW/2 - 3D Model

6.5 Characteristic values of actions combined in load groups

According to paragraph 2.5.1.8.2.3**Error! Reference source not found.**, The effects of vertical loads due to the presence of trains shall always be combined with the other actions resulting from railway traffic, using the factors indicated in Table 42.

Table 42. Traffic load combinations

TIPO DI CARICO	Azioni verticali		Azioni orizzontali			Commenti
Gruppi di carico	Carico verticale (1)	Treno scarico	Frenatura e avviamento	Centrifuga	Serpeggio	
Gruppo 1 (2)	1,0	-	0,5 (0,0)	1,0 (0,0)	1,0 (0,0)	massima azione verticale e laterale
Gruppo 2 (2)	-	1,0	0,0	1,0 (0,0)	1,0 (0,0)	stabilità laterale
Gruppo 3 (2)	1,0 (0,5)	-	1,0	0,5 (0,0)	0,5 (0,0)	massima azione longitudinale
Gruppo 4	0,8 (0,6;0,4)	-	0,8 (0,6;0,4)	0,8 (0,6;0,4)	0,8 (0,6;0,4)	Fessurazione

(1) Includendo tutti i valori (F; a; etc..)

(2) La simultaneità di due o tre valori caratteristici interi (assunzione di diversi coefficienti pari ad 1.0), sebbene improbabile, è stata considerata come semplificazione per i gruppi di carico 1,2 e 3 senza che ciò abbia significative conseguenze progettuali

I valori campiti in grigio rappresentano l'azione dominante.

In addition to the information in this table (see Table 42.), it is necessary to also verify with paragraph 5.2.3.1.2**Error! Reference source not found.** the number of trains that can be placed in the bridge at the same time according to the following table.

Table 43. Moving loads as a function of the number of tracks on the bridge

Numero di binari	Binari Carichi	Traffico normale caso a(1)	Traffico normale caso b(1)	Traffico pesante(2)
1	Primo	1,0 (LM 71"+"SW/0)	-	1,0 SW/2
2	Primo	1,0 (LM 71"+"SW/0)	-	1,0 SW/2
	secondo	1,0 (LM 71"+"SW/0)	-	1,0 (LM 71"+"SW/0)
≥3	Primo	1,0 (LM 71"+"SW/0)	0,75 (LM 71"+"SW/0)	1,0 SW/2
	secondo	1,0 (LM 71"+"SW/0)	0,75 (LM 71"+"SW/0)	1,0 (LM 71"+"SW/0)
	Altri	-	0,75 (LM 71"+"SW/0)	-

(1) LM71 "+" SW/0 significa considerare il più sfavorevole fra i treni LM 71, SW/0

(2) Salvo i casi in cui sia esplicitamente escluso

For this analysis, it will be assumed that the bridge has just one-track meaning that it will only be possible to have one train at a time.

According to paragraph 5.2.2.2.1.2**Error! Reference source not found.**, it is important to remark that the SW/0 load model schematizes the static effects produced by normal railway traffic for continuous beams (it should only be used for continuous beams if it is more unfavorable than LM71).

The following are the different cases considered based on Table 42 and Table 43, according to the bridge static scheme.

- Light traffic in simple supported bridge
- Light traffic in continuous bridge

Table 44.Light traffic load combinations (simple supported)

Load Group	LM71 model	Unloaded train	Acceleration / Braking	Nosing	Name
G1	1		0.5	1	G1_LM71_1
G1	1			1	G1_LM71_2
G1	1		0.5		G1_LM71_3
G2		1		1	G2_1
G3	1		1	0.5	G3_LM71_1
G3	1		1		G3_LM71_2
G3	0.5		1	0.5	G3_LM71_3
G3	0.5		1		G3_LM71_4

Table 45.Light traffic load combinations (continuous bridge)

Load Group	LM71 model	SW/0 model	Unloaded train	Acceleration / Braking	Nosing	Name
G1		1		0.5	1	G1_SW/0_1
G1		1			1	G1_SW/0_2
G1		1		0.5		G1_SW/0_3
G1	1			0.5	1	G1_LM71_1
G1	1				1	G1_LM71_2
G1	1			0.5		G1_LM71_3
G2			1		1	G2_1
G3		1		1	0.5	G3_SW/0_1
G3		1		1		G3_SW/0_2
G3		0.5		1	0.5	G3_SW/0_3
G3		0.5		1		G3_SW/0_4
G3	1			1	0.5	G3_LM71_1
G3	1			1		G3_LM71_2
G3	0.5			1	0.5	G3_LM71_3
G3	0.5			1		G3_LM71_4

In these two cases the acceleration, braking and nosing actions must be amplified by a factor α, according to the following

Table 46.Coefficient "α"

MODELLO DI CARICO	COEFFICIENTE "α"
LM71	1,10
SW/0	1,10
SW/2	1,00

To consider the action due to the heavy traffic, it will be used the SW/2 load model according to Table 41. The heavy traffic combinations apply for both cases, simple support (1) and continuous (2) bridge (see Table 47).

Table 47.Heavy traffic load combinations

Load Group	SW/2 model	Unloaded train	Acceleration / Braking	Nosing	Name
G1	1		0.5	1	G1_SW/2_1
G1	1		0.0	1	G1_SW/2_2
G1	1		0.5	0.0	G1_SW/2_3
G2		1		1	G2_1
G3	1		1	0.5	G3_SW/2_1
G3	1		1	0.0	G3_SW/2_2
G3	0.5		1	0.5	G3_SW/2_3
G3	0.5		1	0.0	G3_SW/2_4

As group 4 (see Table 42.) are a group of combinations used primarily to check the behavior of concrete in the event of cracking, and since this is a steel structure, this group will not be used in this analysis.

For each of these traffic cases (light or heavy), it will be created an envelope to group and look for the higher effect caused in the bridge due to these loads.

Table 48. Envelope Light and Heavy traffic load combinations

Name	Traffic case
ENV_G1_LTH_01	Light traffic in simple supported bridge
ENV_G1_LTH_02	Light traffic in continuous bridge
ENV_G1_HVY_01	Heavy traffic

As was mentioned above, the bridge in analysis will have just one track, therefore, reference is made to the configuration with load model LM71, in the case of normal traffic, and to the configuration with load model SW/2, in the case of heavy traffic.

7

|LIMIT STATE VERIFICATION (ULS and SLS)

For the purposes of structural verifications, it will be referred to the fundamental combination for ultimate limit state (ULS), while for the purposes of deformability verifications, it will be referred to the characteristic combination for serviceability limit state (SLS).

7.1 ULTIMATE LIMIT STATE (ULS):

For the ultimate limit state checks (ULS), the values of the partial coefficients γ in Table 49 and the combination coefficients ψ in Table 50 are adopted.

Table 49. Partial safety factors for load combinations at ULS

		Coefficiente	EQU	A1	A2
Azioni permanenti g_1 e g_3	favorevoli sfavorevoli	γ_{G1} e γ_{G3}	0,9 1,1	1,00 1,35	1,00 1,00
Azioni permanenti non strutturali [2] g_2	favorevoli sfavorevoli	γ_{G2}	0,0 1,5	0,00 1,50	0,00 1,30
Azioni variabili da traffico	favorevoli sfavorevoli	γ_Q	0,0 1,3	0,00 1,35	0,00 1,15
Azioni variabili	favorevoli sfavorevoli	γ_{Qi}	0,0 1,5	0,00 1,50	0,00 1,30
Distorsioni e presollecita-zioni di progetto	favorevoli sfavorevoli	$\gamma_{\varepsilon 1}$	0,9 1,00	1,00 1,00[4]	1,00 1,00
Ritiro e viscosità, Cedimenti vincolari	favorevoli sfavorevoli	$\gamma_{\varepsilon 2}$, $\gamma_{\varepsilon 3}$, $\gamma_{\varepsilon 4}$	0,0 1,2	0,00 1,20	0,00 1,00

Table 50. Combination coefficients of actions

Azioni		ψ_0	ψ_1	ψ_2
Azioni singole	Carico sul rilevato a tergo delle spalle	0,80	0,50	0,0
da traffico	Azioni aerodinamiche generate dal transito dei convogli	0,80	0,50	0,0
	gr_1	0,80[2]	0,80[1]	0,0
Gruppi di	gr_2	0,80[2]	0,80[1]	-
carico	gr_3	0,80[2]	0,80[1]	0,0
	gr_4	1,00	1,00[1]	0,0
Azioni del vento	F_{Wk}	0,60	0,50	0,0
Azioni da	in fase di esecuzione	0,80	0,0	0,0
neve	SLU e SLE	0,0	0,0	0,0
Azioni termiche	T_k	0,60	0,60	0,50

(1) 0,80 se è carico solo un binario, 0,60 se sono carichi due binari e 0,40 se sono carichi tre o più binari.

For the serviceability limit state checks (SLS), the values of the partial coefficients ψ in Table 51 are adopted.

Table 51. Partial safety factors for load combinations at SLS

	Azioni	ψ_0	ψ_1	ψ_2
Azioni singole da traffico	Treno di carico LM 71	0,80[3]	[1]	0,0
	Treno di carico SW /0	0,80[3]	0,80	0,0
	Treno di carico SW/2	0,00[3]	0,80	0,0
	Treno scarico	1,00[3]	-	-
	Centrifuga	[2] [3]	[2]	[2]
	Azione laterale (serpeggio)	1,00[3]	0,80	0,0

(1) 0,80 se è carico solo un binario, 0,60 se sono carichi due binari e 0,40 se sono carichi tre o più binari.
(2) Si usano gli stessi coefficienti ψ adottati per i carichi che provocano dette azioni.

For the purposes of limit state checks, the following combinations of actions based on chapter 2.5.3**Error! Reference source not found.** are defined.

- Combinazione fondamentale, generalmente impiegata per gli stati limite ultimi (SLU):
  $\gamma_{G1} \cdot G_1 + \gamma_{G2} \cdot G_2 + \gamma_P \cdot P + \gamma_{Q1} \cdot Q_{k1} + \gamma_{Q2} \cdot \psi_{02} \cdot Q_{k2} + \gamma_{Q3} \cdot \psi_{03} \cdot Q_{k3} + \ldots$ [2.5.1]
- Combinazione caratteristica, cosiddetta rara, generalmente impiegata per gli stati limite di esercizio (SLE) irreversibili:
  $G_1 + G_2 + P + Q_{k1} + \psi_{02} \cdot Q_{k2} + \psi_{03} \cdot Q_{k3} + \ldots$ [2.5.2]
- Combinazione frequente, generalmente impiegata per gli stati limite di esercizio (SLE) reversibili:
  $G_1 + G_2 + P + \psi_{11} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{23} \cdot Q_{k3} + \ldots$ [2.5.3]
- Combinazione quasi permanente (SLE), generalmente impiegata per gli effetti a lungo termine:
  $G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{23} \cdot Q_{k3} + \ldots$ [2.5.4]
- Combinazione sismica, impiegata per gli stati limite ultimi e di esercizio connessi all'azione sismica E:
  $E + G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \ldots$ [2.5.5]
- Combinazione eccezionale, impiegata per gli stati limite ultimi connessi alle azioni eccezionali A:
  $G_1 + G_2 + P + A_d + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \ldots$ [2.5.6]

Gli effetti dell'azione sismica saranno valutati tenendo conto delle masse associate ai seguenti carichi gravitazionali:

$$G_1 + G_2 + \sum_j \psi_{2j} Q_{kj} \,. \qquad [2.5.7]$$

The following table will show the different set of combinations of actions for CASE 2.5.1 based on chapter 2.5.3**Error! Reference source not found.**, this will show the different factor considering the dynamic factor for Bottom and Top chords.7

Table 52. Fundamental combination, ultimate limit states (ULS), for Bottom and Top chords.

		COMBINATION NAME	**Permanent**	**Variable (Traffic)**	**Variable Thermal**	**Variable Wind**	**Settlements**
			G	Q (GR1,GR3)	Tk	FWk	ST
GROUP	**Permanent actions**						
1	1	SLU01-1-ALL	1.35	0	0	0	0
	2	SLU01-2-ALL	1.35	0	0	0	1.2
WITHOUT SETTLEMENTS							
TRAFFIC							
	G min						
2	1	1A01-SLU02-1-BT	1.00	1.57	0.00	0.00	0.00
	2	1A02-SLU02-2-BT	1.00	1.57	0.90	0.00	0.00
	3	1A03-SLU02-3-BT	1.00	1.57	0.00	0.90	0.00
	4	1A04-SLU02-4-BT	1.00	1.57	0.90	0.90	0.00
	G max						
3	1	1A05-SLU03-1-BT	1.35	1.57	0.00	0.00	0.00
	2	1A06-SLU03-2-BT	1.35	1.57	0.90	0.00	0.00
	3	1A07-SLU03-3-BT	1.35	1.57	0.00	0.90	0.00
	4	1A08-SLU03-4-BT	1.35	1.57	0.90	0.90	0.00
THERMAL							
	G min						
4	1	1A09-SLU04-1-BT	1.00	0.00	1.50	0.00	0.00
	2	1A10-SLU04-2-BT	1.00	1.47	1.50	0.00	0.00
	3	1A11-SLU04-3-BT	1.00	0.00	1.50	0.90	0.00
	4	1A12-SLU04-4-BT	1.00	1.47	1.50	0.90	0.00
	G max						
5	1	1A13-SLU04-1-BT	1.35	0.00	1.50	0.00	0.00
	2	1A14-SLU04-2-BT	1.35	1.47	1.50	0.00	0.00
	3	1A15-SLU04-3-BT	1.35	0.00	1.50	0.90	0.00
	4	1A16-SLU04-4-BT	1.35	1.47	1.50	0.90	0.00
WIND							
	G min						
6	1	1A17-SLU06-1-BT	1.00	0.00	0.00	1.50	0.00
	2	1A18-SLU06-2-BT	1.00	0.00	0.90	1.50	0.00
	3	1A19-SLU06-3-BT	1.00	1.47	0.00	1.50	0.00
	4	1A20-SLU06-4-BT	1.00	1.47	0.90	1.50	0.00
	G max						
7	1	1A21-SLU07-1-BT	1.35	0.00	0.00	1.50	0.00
	2	1A22-SLU07-2-BT	1.35	0.00	0.90	1.50	0.00
	3	1A23-SLU07-3-BT	1.35	1.47	0.00	1.50	0.00
	4	1A24-SLU07-4-BT	1.35	1.47	0.90	1.50	0.00

WITH SETTLEMENTS								
TRAFFIC								
	G min							
8	1	1A25-SLU08-1-BT	1.00	1.57	0.00	0.00	1.20
	2	1A26-SLU08-2-BT	1.00	1.57	0.90	0.00	1.20
	3	1A27-SLU08-3-BT	1.00	1.57	0.00	0.90	1.20
	4	1A28-SLU08-4-BT	1.00	1.57	0.90	0.90	1.20
	G max							
9	1	1A29-SLU09-1-BT	1.35	1.57	0.00	0.00	1.20
	2	1A30-SLU09-2-BT	1.35	1.57	0.90	0.00	1.20
	3	1A31-SLU09-3-BT	1.35	1.57	0.00	0.90	1.20
	4	1A32-SLU09-4-BT	1.35	1.57	0.90	0.90	1.20
THERMAL								
	G min							
10	1	1A33-SLU10-1-BT	1.00	0.00	1.50	0.00	1.20
	2	1A34-SLU10-2-BT	1.00	1.47	1.50	0.00	1.20
	3	1A35-SLU10-3-BT	1.00	0.00	1.50	0.90	1.20
	4	1A36-SLU10-4-BT	1.00	1.47	1.50	0.90	1.20
	G max							
11	1	1A37-SLU11-1-BT	1.35	0.00	1.50	0.00	1.20
	2	1A38-SLU11-2-BT	1.35	1.47	1.50	0.00	1.20
	3	1A39-SLU11-3-BT	1.35	0.00	1.50	0.90	1.20
	4	1A40-SLU11-4-BT	1.35	1.47	1.50	0.90	1.20
WIND								
	G min							
12	1	1A41-SLU12-1-BT	1.00	0.00	0.00	1.50	1.20
	2	1A42-SLU12-2-BT	1.00	1.47	0.90	1.50	1.20
	3	1A43-SLU12-3-BT	1.00	0.00	0.00	1.50	1.20
	4	1A44-SLU12-4-BT	1.00	1.47	0.90	1.50	1.20
	G max							
13	1	1A45-SLU13-1-BT	1.35	0.00	0.00	1.50	1.20
	2	1A46-SLU13-2-BT	1.35	1.47	0.90	1.50	1.20
	3	1A47-SLU13-3-BT	1.35	0.00	0.00	1.50	1.20
	4	1A48-SLU13-4-BT	1.35	1.47	0.90	1.50	1.20

In Table 52, there are a total of 13 different groups of combinations taken into account the different partial coefficients (γ) given in Table 49 and the combination factor (ψ) given in Table 50 and Table 51 depending on the case of analysis.

The 13 groups are divided into two different categories according to the analysis of the settlements **(without settlements or with settlements)**, in each of these categories the combinations considered the simultaneously of the different variable actions such as Traffic, Wind or Thermal, each of which are multiplied by its own combination factor (ψ).

As an example for the category "**without settlements**", there are a total of 7 groups, ad for each group there are a total of 4 different combinations, with this into the model it will be loaded a total of 28 combinations for case 2.5.1 given in chapter 2.5.3**Error! Reference source not found.**.

However it is important to remark that Traffic loads depend also on the analysis with the type of traffic (LM71, SW/0 or SW/2) according to chapter 6.5 and on analysis made for the dynamic factor in chapter 5, the Traffic loads combinations used will be the one given in Table 48.

For each type of element indicated in Table 40 it will be created a set of 28 combinations for case 2.5.1 and so on for each case.

All the combinations used in the analysis are shown in ANNEX-04_Load Combinations.

7.2 LIMIT STATE (SLS):

According to 2.5.1.8.3.2**Error! Reference source not found.**, the serviceability limit states to be verified concern at least the following aspects:

- Stress state (for reinforced concrete, prestressed concrete, and composite structures)
- Deformations and vibrations
- Cracking (for reinforced concrete, prestressed concrete, and composite structures)
- Slip of joints (for steel structures with bolted and riveted connections)

The most important aspect to consider as a steel structure will be the deformations and vibrations and the slip of joints.

7.2.1 Deformations and vibrations:

Based on 2.5.1.8.3.2.2**Error! Reference source not found.**, excessive deformations and/or vibrations of the bridge can cause unacceptable changes in the geometry of the track. These may have repercussions on railway convoys and reduce passenger comfort.

The assessment of the deformation parameters of the decks, to be carried out using the characteristic (Case 2.5.2, rare) combination of the Serviceability Limit States (SLS), is required for the reasons and in the manner outlined in the following points A and B:

A. For railway traffic safety reasons (to ensure the stability and continuity of the track and to maintain wheel-rail contact), it is necessary to verify that the limits for the following parameters are not exceeded:

- Vertical acceleration of the deck
- Torsion of the deck (track skew)
- Bending of the deck in the horizontal plane
- Bending of the deck in the vertical plane

B. For passenger comfort, it must be verified that the limits of vertical deflection of the deck are not exceeded.

7.2.1.1 Vertical acceleration of the deck:

According to 5.2.3.2.2.1**Error! Reference source not found.**, this verification is required for structures where the operating speed exceeds 200 km/h or when the natural frequency of the structure is not within the limits indicated in Figure 51.

In this case, it is considered that the speed will not exceed 200 km/h, and its natural frequency is on the limits indicated in Figure 51, the verification can be neglected.

7.2.1.2 Torsion of the deck (track skew):

The torsion of the bridge deck is calculated by considering the LM71 load train increased with the corresponding dynamic factor.

The maximum skew, measured over a length of 3 meters and considered the rails as integral with the deck (Figure 73).

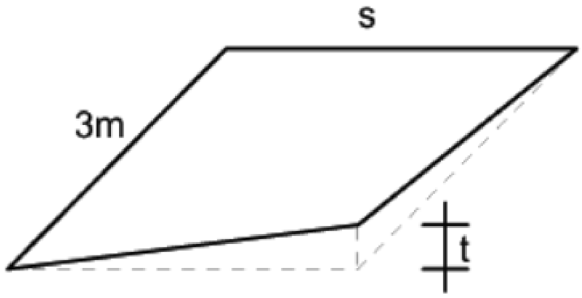


Figure 73. Maximum skew

Must not exceed the following values:

for V ≤120 km/h; t ≤ 4,5 mm/3m

for 120 < V ≤ 200 km/h; t ≤3,0 mm/3m

for V > 200 km/h; t ≤1,5 mm/3m

For speeds V > 200 km/h, it must also be verified that for real trains, multiplied by the corresponding dynamic factor, the skew does not exceed t ≤1.2 mm/ 3m.

7.2.1.3 Bending of the deck in the horizontal plane:

Considering the presence of the LM71 load train, increased with the corresponding dynamic factor, wind action, lateral force (hunting motion), centrifugal force, and the effects of linear temperature variation between the two sides of the deck, the horizontal bending of the deck must not result in:

- An angular variation greater than that given in Table 53.
- A horizontal radius of curvature smaller than the values specified in Table 53.

Table 53.Maximum angular variation and minimum radius of curvature

Velocità [km/h]	Variazione Angolare massima	Raggio minimo di curvatura	
		Singola campata	Più campate
V ≤ 120	0,0035 rd	1700 m	3500 m
120 < V ≤ 200	0,0020 rd	6000 m	9500 m
200 < V	0,0015 rd	14000 m	17500 m

The radius of curvature, in the case of simply supported decks, is given by the following expression:

$$R = \frac{L^2}{8\delta_h} \tag{47}$$

Where, δ_h represents the horizontal displacement.

7.2.1.4 *Bending of the deck in the vertical plane:*

Considering the presence of the LM71, SW/0, and SW/2 trains, increased with the corresponding dynamic factor and the coefficient α, the maximum deflection value due to these railway loads must not exceed the value:

$$\delta = \frac{L}{600} \tag{48}$$

where L is the span length.

7.2.2 ANALYSIS AND RESULTS:

7.2.2.1 Torsion of the deck (track skew):

According to 10.2.1.2 from this report, the torsion of the deck will be calculated considering the displacements on the rails given by the following labels.

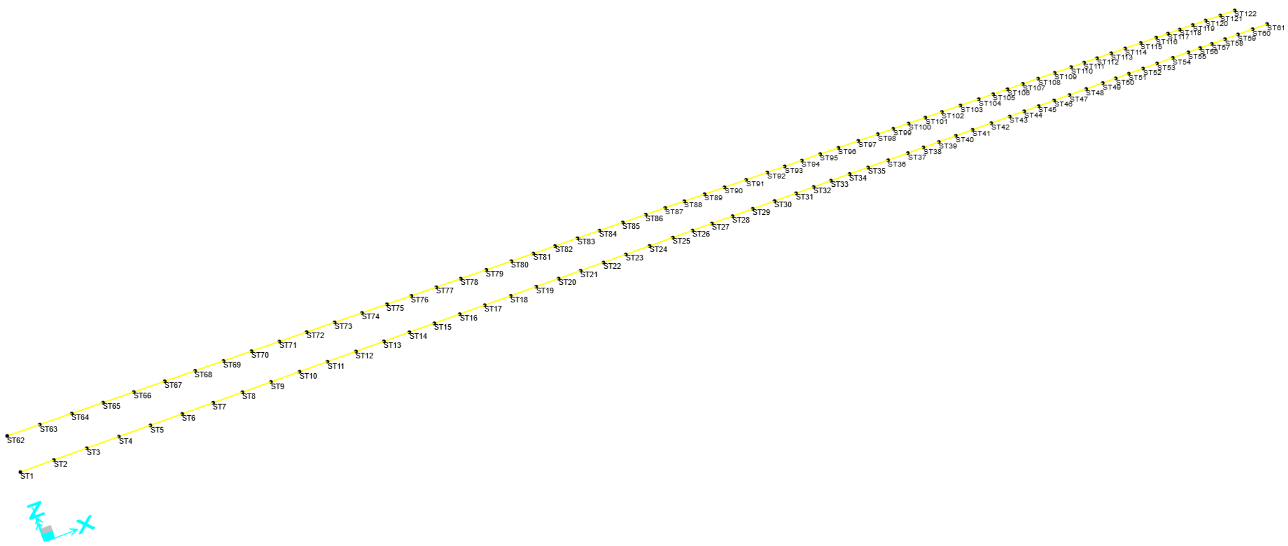


Figure 74. Rail labels

Following Figure 73, we get a distribution as the following:

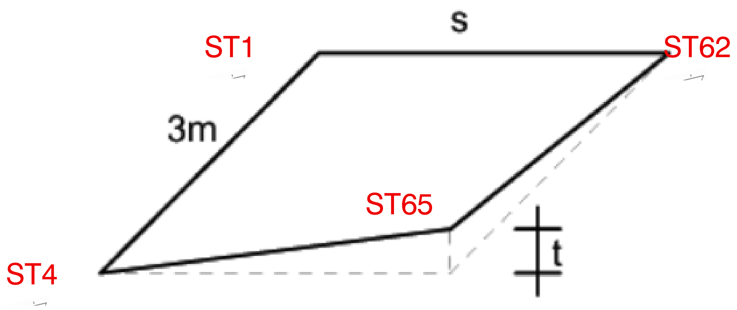


Figure 75. Rail points of analysis each 3m

The distance between points ST1 and ST4 is approximately 2.84 m. Since this length may vary slightly, and to facilitate the automation of the verification process, the following premise is applied: as the values of *t* extend beyond 3 m, the limit for this distance will be scaled proportionally to the actual spacing between the points. This approach simplifies the procedure while maintaining consistency with the real values. The verification will therefore be carried out for each point, as shown in

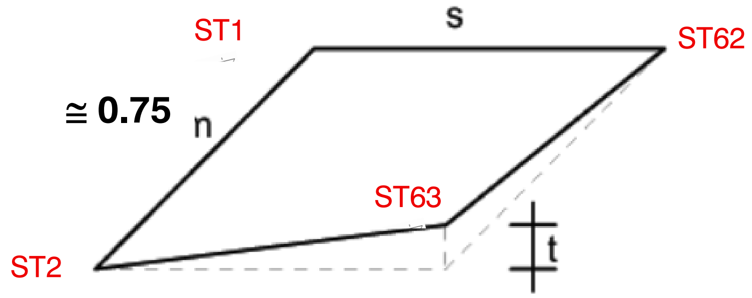


Figure 76.Rail points of analysis

With this the scale limit will be:

for 120 < V ≤ 200 km/h; t ≤0.75mm/0.75m

According to 7.2.1 the verifications must be made using the characteristic (Case 2.5.2, rare) combination of the Serviceability Limit States (SLS), with this and according to the set of combination given in ANNEX-04_Load Combinations and the results shown in ANNEX-05_Track Skew Verification, it is possible to get that the maximum value of **"t"** is equal to **t= 0.77,** in this case the value is slightly higher but the difference can be neglected and with this it is fulfill the verification.

1.1.1.1 Bending of the deck in the horizontal plane:

For the bending in the horizontal plane the displacements are going to be taken in the points where there is rigid connection between the rails and the transverse beam as it is shown in

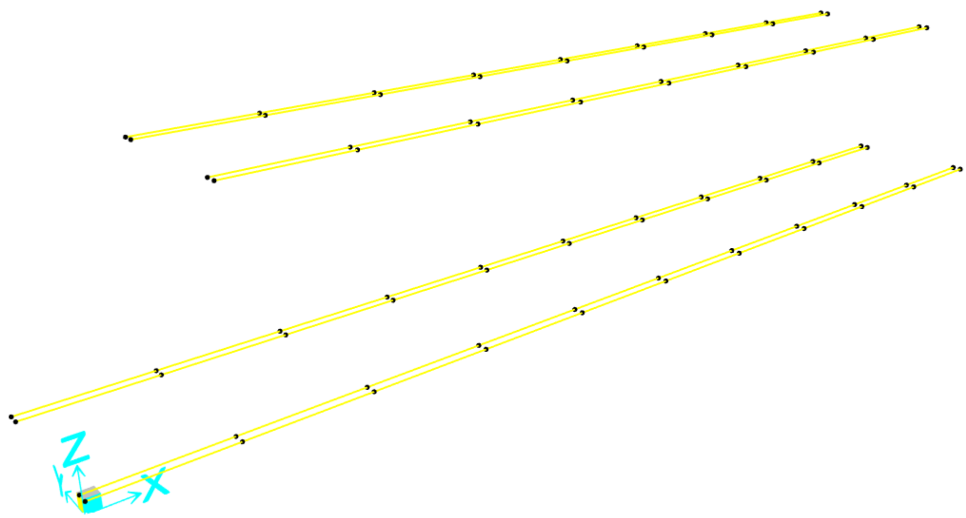


Figure 77. Points of analysis, bending of the deck in horizontal plane

As it was made for the track skew verification, for bending of the deck in horizontal plane it is also used the characteristic combination Case 2.5.2, the set of combinations used is given in ANNEX-06_Track Horizontal Verification with the results of the analysis. For this case, the maximum horizontal displacement is equal to **|U2max|=0.026 m,** according to 7.2.1.3, the radius of curvature in the case of simply supported decks, is given by equation (48), as a result:

$$R = \frac{(43m)^2}{8 \cdot 0.026m} = 8829.6\ m$$

According to Table 53, the minimum value for a bridge where the speed is:

$$120 < V \leq 200 \text{ km/h then } R \geq 6000 \text{ mm}$$

With the value obtained **R=8829.6 m**, it fulfills the verification.

Additionally, this is necessary to get the value of the maximum angular variation, for this case it is obtained a value equal to θ **= 0.0009 rad**, lower than the one indicated in Table 53, θ **= 0.0020 rad.**

1.1.1.1 Bending of the deck in the vertical plane:

According to what is indicated in 7.2.1.4, the vertical displacement with the presence of the different train loads must not exceed the limit indicated above.

$$\delta = \frac{43m}{600} = 0.071\ m$$

According to the results given in ANNEX-07_Track Vertical Verification, the maximum value considering the presence of the LM71, SW/0, and SW/2 trains, is equal to δ**=0.048,** with this is fulfill the vertical verification

8

|FATIGUE VERIFICATION OF RAILWAY BRIDGES

According to chapter 2.7**Error! Reference source not found.** and more specifically in 2.7.1.2**Error! Reference source not found.**, It is possible to reduce the fatigue verification to a conventional strength verification by comparing the conventional design stress range, $\Delta\sigma_{E,d}$, described below, with the category of the detail $\Delta\sigma_c$. The conventional design stress range $\Delta\sigma_{E,d}$ is given by:

$$\Delta\sigma_{E,d} = \lambda \cdot \phi_2 \cdot \Delta\sigma_{71} \quad (49)$$

To ensure safety with respect to fatigue, the following condition must be satisfied:

$$\Delta\sigma_{E,d} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \quad (50)$$

Where:

- λ: It is a correction factor whose numerical values are defined in 8.2.
- $\Delta\sigma_{71}$: It is the stress range between the extreme values σ_{max} and σ_{min} due to the theoretical design overload adopted for the bridge (LM71) placed in the most unfavorable position.
- $\Delta\sigma_c$: It is the fatigue strength corresponding to 2×10^6 cycles, to be obtained from the SN curves corresponding to the structural detail under examination 8.3.
- Φ_2: It is the dynamic amplification factor of the theoretical overload, evaluated using equation (38)
- γ_{Mf}: It is the partial safety coefficient.

8.1 PARTIAL SAFETY COEFFICIENT (γ_{Mf}):

The partial safety coefficient depends both on the possibility of detecting and repairing potential fatigue cracks and on the extent of the consequences of a fatigue failure of the element or structure, this is given in chapter 2.7.1.1.4**Error! Reference source not found.** for the following expression:

$$\gamma_{Mf} = \gamma_f \cdot \gamma_m \tag{51}$$

Where:

- γ_f is the partial factor related to fatigue actions
- γ_m is the partial factor related to resistance

This value of γ_{Mf} is given by Table 54.

Table 54.Partial coefficients γ_{Mf} for fatigue checks

Criteri di valutazione della resistenza a fatica	Conseguenza della rottura per fatica	
	Moderate	Significative
Danneggiamento accettabile (strutture poco sensibili alla rottura per fatica)	$\gamma_{Mf} = 1{,}00$	$\gamma_{Mf} = 1{,}15$
Vita utile (strutture sensibili alla rottura per fatica)	$\gamma_{Mf} = 1{,}15$	$\gamma_{Mf} = 1{,}35$

It should be noted that steel railway structures are structures sensitive to fatigue failure.

Unless otherwise established by railways authorities, the partial coefficient for fatigue (γ_{Mf}) loads for steel railway bridges, since these are "sensitive to fatigue phenomena", must be assumed to be equal to:

$$\gamma_{Mf} = 1.35$$

8.2 NUMERICAL VALUES OF THE CORRECTION FACTOR "λ":

According to 2.7.1.2.1**Error! Reference source not found.** the value of "λ" is given by the following expression:

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \text{ with } \lambda \leq \lambda_{max} \quad (52)$$

Where:

- λ_1 is a factor that, for different types of girders, accounts for the damaging effect due to traffic and depends on the characteristic influence length of the element to be verified.
- λ_2 is a factor that accounts for the traffic volume.
- λ_3 is a factor that accounts for the design life of the bridge.
- λ_4 is a factor to be applied when the structural element is loaded by more than one track, however as the analysis for a in single track this factor can be neglected.

8.2.1 λ_1 Values:

The value of the span factor λ_1, corresponding to the "standard" load combination, can be obtained from Table 55.

Table 55. Values of λ depending on the length L(m)

L [m]	λ_1	L [m]	λ_1
0,5	1,60	12,5	0,82
1,0	1,60	15,0	0,76
1,5	1,60	17,5	0,70
2,0	1,46	20,0	0,67
2,5	1,38	25,0	0,66
3,0	1,35	30,0	0,65
3,5	1,17	35,0	0,64
4,0	1,07	40,0	0,64
4,5	1,02	45,0	0,64
5,0	1,03	50,0	0,63
6,0	1,03	60,0	0,63
7,0	0,97	70,0	0,62
8,0	0,92	80,0	0,61
9,0	0,88	90,0	0,61
10,0	0,85	100,0	0,60

To determine λ_1, the span length should be assumed by referring to Table 56 and Table 57, respectively for stress states resulting from bending and shear actions.

Table 56. Length L for stress states arising from bending stresses

PER STATI TENSIONALI PROVENIENTI DA SOLLECITAZIONI DI FLESSIONE:			
		Per:	Luce L da considerare
1		una campata semplicemente appoggiata	luce della campata L
2		campate continue nelle sezioni di mezzeria (vedere Figura 3.1-1)	la luce della campata Li considerata
3		campate continue nelle sezioni sugli appoggi (vedere Figura 3.1-1)	la media delle campate Li e Lj adiacenti a quell'appoggio
4		travature trasversali facenti da appoggio a supporti delle rotaie (o a irrigidimenti)	la somma delle due campate dei supporti delle rotaie (o degli irrigidimenti) immediatamente adiacenti alla travatura trasversale
5	a	piastra di impalcato sostenuta solo da travature o irrigidenti trasversali (senza elementi longitudinali) e per quelle che sostengono elementi trasversali	lunghezza della linea di influenza dello spostamento (trascurando le parti con lo spostamento verso l'alto), prendendo in dovuto conto la rigidezza delle rotaie nella distribuzione del carico
	b	elementi trasversali distanti tra di loro non più di 750 mm	2 volte la distanza tra gli elementi trasversali + 3 m

Table 57. Length L for stress states arising from shear stresses

PER STATI TENSIONALI PROVENIENTI DA SOLLECITAZIONI DI TAGLIO:			
		Per:	Luce L da considerare
6		una campata semplicemente appoggiata nelle sezioni di mezzeria (vedere Figura 2.7.1.2.1-1)	la luce della campata L
7		una campata semplicemente appoggiata nelle sezioni sugli appoggi (vedere Figura 2.7.1.2.1-1)	0,4 × la luce della campata L
8		campate continue nelle sezioni di mezzeria (vedere Figura 2.7.1.2.1-1)	la luce della campata Li considerata
9		campate continue nelle sezioni sugli appoggi (vedere Figura 2.7.1.2.1-1)	0,4 × la luce della campata Li considerata

8.2.2 Influence of traffic volume (λ_2):

Based on 2.7.1.2.2**Error! Reference source not found.**, for bridges located on lines for which a future annual traffic (T) different in terms of tonnage from the reference value (24.95×10^6 t/year) is expected, the stress values must be corrected using the coefficient λ_2, which is a function of the reference traffic volume.

The table below shows the values of the correction factor λ_2 for different traffic volume value:

Table 58. Values of λ_2 in terms of annual traffic volume

Traffico annuo [10^6 t/ binario]	*5*	*10*	*15*	*20*	*25*	*30*	*35*	*40*	*50*
λ_2	*0,72*	*0,83*	*0,90*	*0,96*	*1,00*	*1,04*	*1,07*	*1,10*	*1,15*

Unless otherwise specified by railway authorities, λ2 should be assumed to correspond to a traffic volume of 25×10^6 t/year/route.

8.2.3 Influence of fatigue in the design life(λ_3):

Based on 2.7.1.2.3**Error! Reference source not found.**,to carry out the verification of bridges for which a fatigue life different from 100 years is specified, the tabulated values of the correction factor λ3 are provided as a function of the service life 'N

Table 59. Values of λ3 in terms of the design life of the structure

Vita utile a fatica [anni]	50	60	70	80	90	100	120
λ_3	0,87	0,90	0,93	0,96	0,98	1,00	1,04

Unless otherwise specified by railway authorities, the value of λ_3 corresponding to a service life of 100 years must be assumed

As for each member, the value of ϕ_2 depends on the characteristic length L_ϕ given in chapter 5 of this document, the value of ϕ_2 for each element are taken from Table 40.

ITEM	Real Length (m)	Characteristic Length (L_ϕ)(m)
Bottom and Top chords	43	43
Stringers	4.3	7.30
Intermediate cross beams	5.6	8.60
End cross beams	5.6	3.60

ITEM	ϕ_2	L_ϕ(m)	λ_1	λ_2	λ_3	λ
Bottom and Top chords	1.04	43	0.64	1.0	1.0	0.64
Stringers	1.39	4.3	1.07	1.0	1.0	1.07

Intermediate cross beams	1.34	8.60	0.92	1.0	1.0	0.92
End cross beams	1.66	4.3	1.07	1.0	1.0	1.07

8.3 Fatigue resistance $\Delta\sigma_c$:

As is mentioned at the beginning of this chapter it is possible to reduce fatigue verification to conventional strength verification. To do this it is necessary to determine the values of the fatigue resistance ($\Delta\sigma_c$) corresponding to 2×10^6 cycles, this could be obtained based on the structural detail under analysis.

To simplify the analysis, the main structure is considered as consisting of two element types, classified into two categories: *rolled or extruded* sections and *welded built-up* sections. Since fatigue verification will be carried out only for the sections, without considering the joints to simplify the analysis, the following details from EN 1993-1-9**Error! Reference source not found.** are considered in this verification.

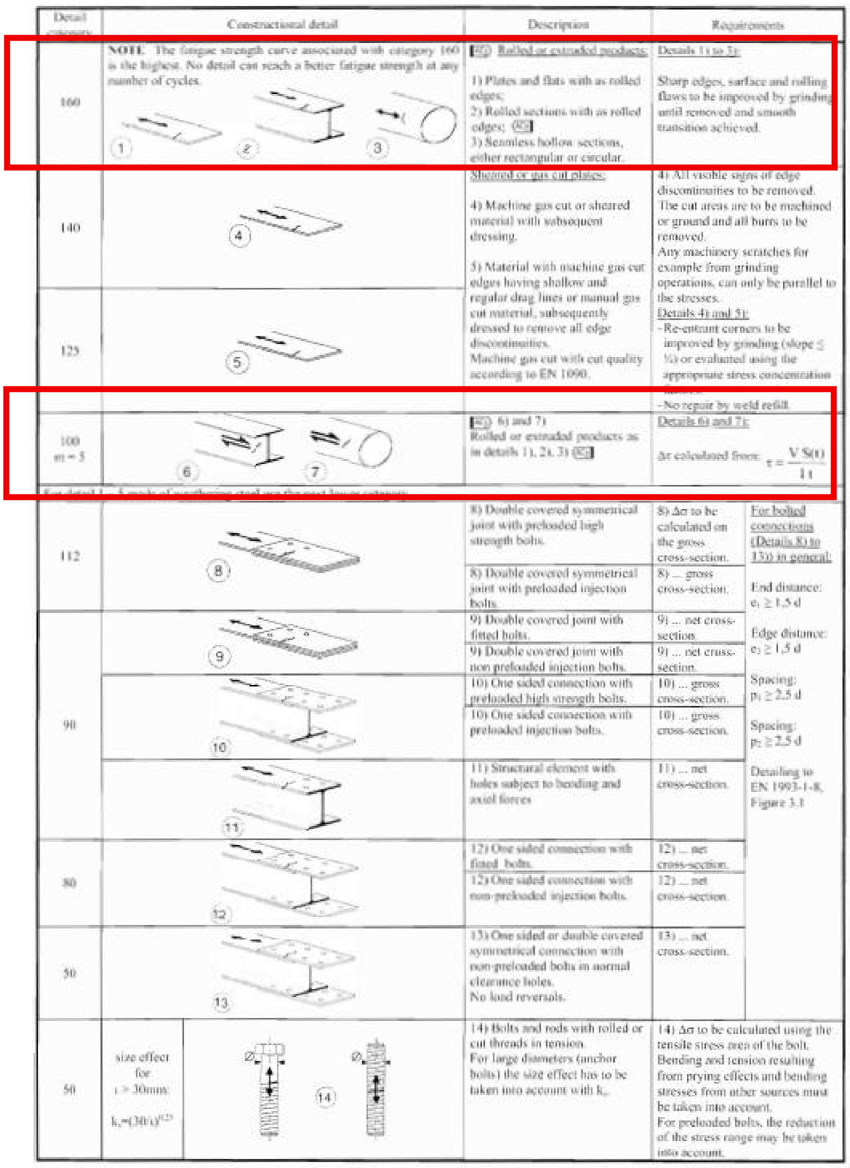
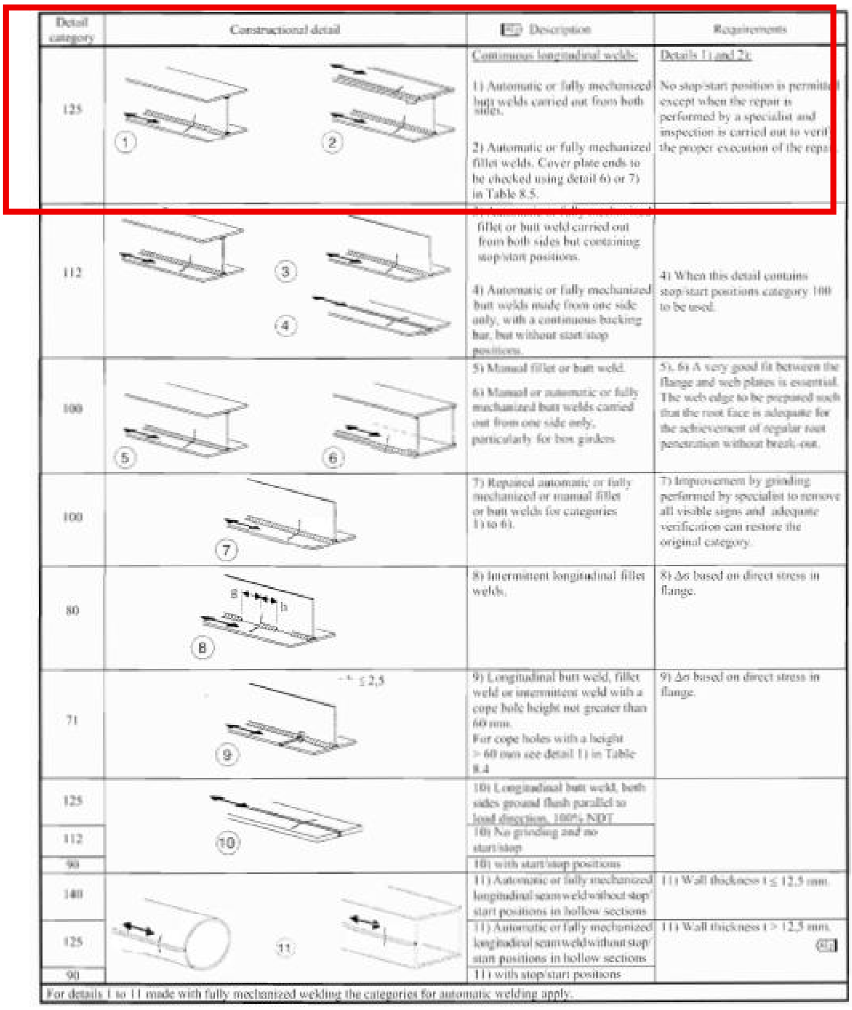


Figure 78. Plain members and mechanically fastened joints.



Detail category	Constructional detail	Description	Requirements
125	1, 2	Continuous longitudinal welds: 1) Automatic or fully mechanized butt welds carried out from both sides. 2) Automatic or fully mechanized fillet welds. Cover plate ends to be checked using detail 6) or 7) in Table 8.5.	Details 1) and 2): No stop/start position is permitted except when the repair is performed by a specialist and inspection is carried out to verify the proper execution of the repair.
112	3, 4	3) Automatic or fully mechanized fillet or butt weld carried out from both sides but containing stop/start positions. 4) Automatic or fully mechanized butt welds made from one side only, with a continuous backing bar, but without start/stop positions.	4) When this detail contains stop/start positions category 100 to be used.
100	5, 6	5) Manual fillet or butt weld. 6) Manual or automatic or fully mechanized butt welds carried out from one side only, particularly for box girders	5), 6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out.
100	7	7) Repaired automatic or fully mechanized or manual fillet or butt welds for categories 1) to 6).	7) Improvement by grinding performed by specialist to remove all visible signs and adequate verification can restore the original category.
80	8	8) Intermittent longitudinal fillet welds.	8) $\Delta\sigma$ based on direct stress in flange.
71	9	9) Longitudinal butt weld, fillet weld or intermittent weld with a cope hole height not greater than 60 mm. For cope holes with a height > 60 mm see detail 1) in Table 8.4	9) $\Delta\sigma$ based on direct stress in flange.
125	10	10) Longitudinal butt weld, both sides ground flush parallel to load direction, 100% NDT	
112		10) No grinding and no start/stop	
90		10) with start/stop positions	
140	11	11) Automatic or fully mechanized longitudinal seam weld without stop/start positions in hollow sections	11) Wall thickness $t \leq 12{,}5$ mm.
125		11) Automatic or fully mechanized longitudinal seam weld without stop/start positions in hollow sections	11) Wall thickness $t > 12{,}5$ mm.
90		11) with stop/start positions	
For details 1 to 11 made with fully mechanized welding the categories for automatic welding apply.			

Figure 79.Welded built-up sections.

Now, consider the following information about which are going to be the forces acting on each element, it is possible to define $\Delta\sigma$ or $\Delta\tau$:

Table 60. Acting internal forces by element

Element	Axial	Bending	Shear
Rails	✓	✓	✓
Stringers	-	✓	✓
Transversal beams	-	✓	✓
Main truss chords	✓	-	–
Main truss diagonals/verticals	✓	-	–

From the 3D model it is possible to get the values of σ_{max} obtained from the load model LM71, while the value of σ_{min} will be assumed prudently as zero.

To simplify the analysis, it will be taken the maximum value of σ for the elements mentioned in Table 60.

Element	$\Delta\sigma_{7L}$(MPa)	ϕ_2	λ	$\Delta\sigma_c$(MPa)	γ_{Mf}	$\Delta_{E,d}<\Delta\sigma_c/\gamma_{Mf}$	OK/NO
Stringers	41	1.39	1.07	125	1.35	0.7	OK
Top Floor beams Intermediate (transversals)	16	1.34	0.92	160	1.35	0.2	OK
Top Floor beams End (transversals)	21	1.66	1.07	160	1.35	0.3	OK
Bottom Floor beams Intermediate (transversals)	42	1.34	0.92	125	1.35	0.6	OK
Bottom Floor beams End (transversals)	43	1.66	1.07	125	1.35	0.8	OK
Main truss chords	74	1.04	0.64	125	1.35	0.5	OK
Main truss diagonals/verticals	19.5	1.04	0.64	160	1.35	0.1	OK

9

|STRUCTURAL ELEMENT'S VERIFICATION

In this chapter, the theory of verification of the structural elements will be explained, however it is important to say that the verification will be done using the SAP2000 design modules.

According to the NTC18**Error! Reference source not found.** the structural validation is based on the Design Limit State, the performance of the structure must be verified under both conditions ULS (Ultimate Limit State) and SLS (Serviceability Limit State), the first ensure safety against collapse, while the second case ensure functionality and comfort.

For the ULS's checks it is necessary to verify the strength, stability and ductility of the elements, according to the NTC18**Error! Reference source not found.**, the following condition must fulfill:

$$R_d \geq E_d \tag{53}$$

Where:

- E_d: is the most unfavorable design value of the effect of the actions, evaluated based on the design values of the actions
- R_d: is design resistance, considering material partial factors and section properties.

To fulfil this requirement, it is necessary to verify the following conditions:

9.1 TENSILE STRENGTH RESISTANCE

Based on chapter 6.2.3 EC3[4], to verify the tensile strength resistance in elements subjected to axial tensile forces it is necessary to fulfill the following condition:

$$N_{Ed} \geq N_{t,Rd} \tag{54}$$

Or

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \tag{55}$$

Where:

- N_{Ed}: is the design axial tensile force from load combinations.
- $N_{t,Rd}$: is the design tensile resistance, based on the limit states below.

To fulfill this condition two failure modes must be checked.

9.1.1 YIELDING OF THE GROSS SECTION ($N_{pl,Rd}$):

This first mode of failure assumes the entire cross-section yields without localized failures.

$$N_{pl,Rd} = A \frac{f_y}{\gamma_{M0}} \tag{56}$$

Where:

- A= gross cross-sectional area
- f_y= yield strength of steel
- γ_{M0} = partial safety factor

9.1.2 FRACTURE OF THE NET SECTION ($N_{U,Rd}$):

This second mode of failure is brittle and typically is the most critical when the elements are bolted (fasteners).

$$N_{U,Rd} = 0.9 A_{net} \frac{f_u}{\gamma_{M2}} \tag{57}$$

Where:

- A_{net}= net cross-sectional area (reduced for bolt holes)
- f_U=ultimate tensile strength of steel
- γ_{M2} = partial safety factor

Finally, for sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller from 9.1.1 and 9.1.2.

$$N_{t,Rd} = \min\,(N_{U,Rd}, N_{pl,Rd}) \tag{58}$$

9.2 COMPRESSION RESISTANCE

According to chapter 6.2.4 EC3[4], to verify compression resistance, the force N_{Ed} at each cross-section shall satisfy:

$$N_{Ed} \geq N_{b,Rd} \tag{59}$$

Or

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1 \tag{60}$$

Where:

$$N_{c,Rd} = A \frac{f_y}{\gamma_{M0}} \tag{61}$$

9.3 BUCKLING RESISTANCE OF MEMBERS:

According to chapter 6.3 EC3[4] to ensure a steel member under axial compression does not buckle or yield before reaching its design resistance it is necessary to fulfill the following condition:

$$N_{Ed} \geq N_{b,Rd} \tag{62}$$

Or,

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \tag{63}$$

Where:

- N_{Ed}: is the design axial tensile force from load combinations.
- $N_{b,Rd}$: is the design buckling resistance, based on the limit states below.

To fulfill this condition, it is necessary to determine del bucking resistance $N_{b,Rd}$ given by:

$$N_{b,Rd} = \chi \cdot A \frac{f_y}{\gamma_{M1}} \tag{64}$$

Where:

- χ = reduction factor due to buckling
- A= gross cross-sectional area
- f_y= yield strength of steel
- γ_{M1} = partial safety factor for stability

9.3.1 *Buckling curves:*

The reduction factor (χ) depends on the member slenderness and buckling curve given by the following equation:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 + \lambda^2}} \tag{65}$$

But $\chi \leq 1$.

Where:

$$\phi = 0.5\,[1 + \alpha(\lambda - 0.2) + \lambda^2] \tag{66}$$

$$\lambda = \sqrt{\frac{A \cdot f_y}{N_{cr}}} \tag{67}$$

With,

$$N_{cr} = \frac{\pi^2 E\, I}{{L_{cr}}^2} \tag{68}$$

Where:

- α = Imperfection factor
- N_{cr}= is the elastic critical force for the relevant buckling mode based on the gross cross-sectional properties
- E = Elastic modulus of steel.
- I= Moment of inertia of the section in the direction under consideration.
- L_{cr}= Is the buckling length in the buckling plane considered (Effective length)

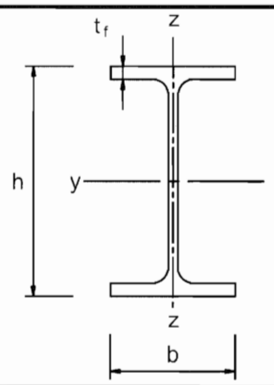
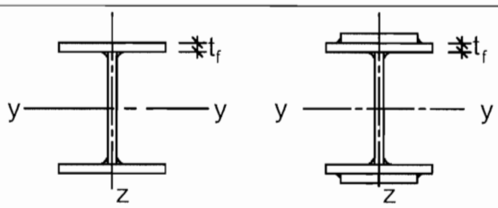
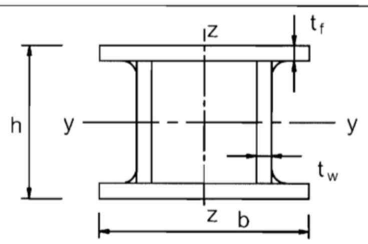
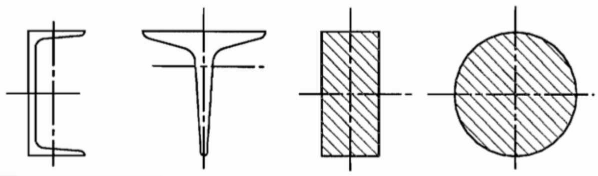
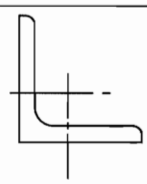
According to 6.3.1.2 (2) EC3 [4], the imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 61.

Table 61. Imperfection Factors for Buckling Curves

Buckling Curve	Imperfection Factor α
a0	0.13
a	0.21
b	0.34
c	0.49
d	0.76

In this case, as the profiles used are considered as rolled or extruded sections, it will be selected the **A curve** and as a result of this the imperfection factor will be α **= 0.21.,** however there are also elements as welded built-up it must be verify according to Table 62.

Table 62. Selection of buckling curve for a cross-section

Cross section		Limits		Buckling about axis	Buckling curve: S 235, S 275, S 355, S 420	Buckling curve: S 460
Rolled sections		$h/b > 1,2$	$t_f \leq 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm $< t_f \leq 100$	y – y z – z	b c	a a
		$h/b \leq 1,2$	$t_f \leq 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \leq 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0,5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

According to 6.3.1.2 (3) EC3 [4], Values of the reduction factor χ for the appropriate non-dimensional slenderness λmay be obtained from Figure 80.

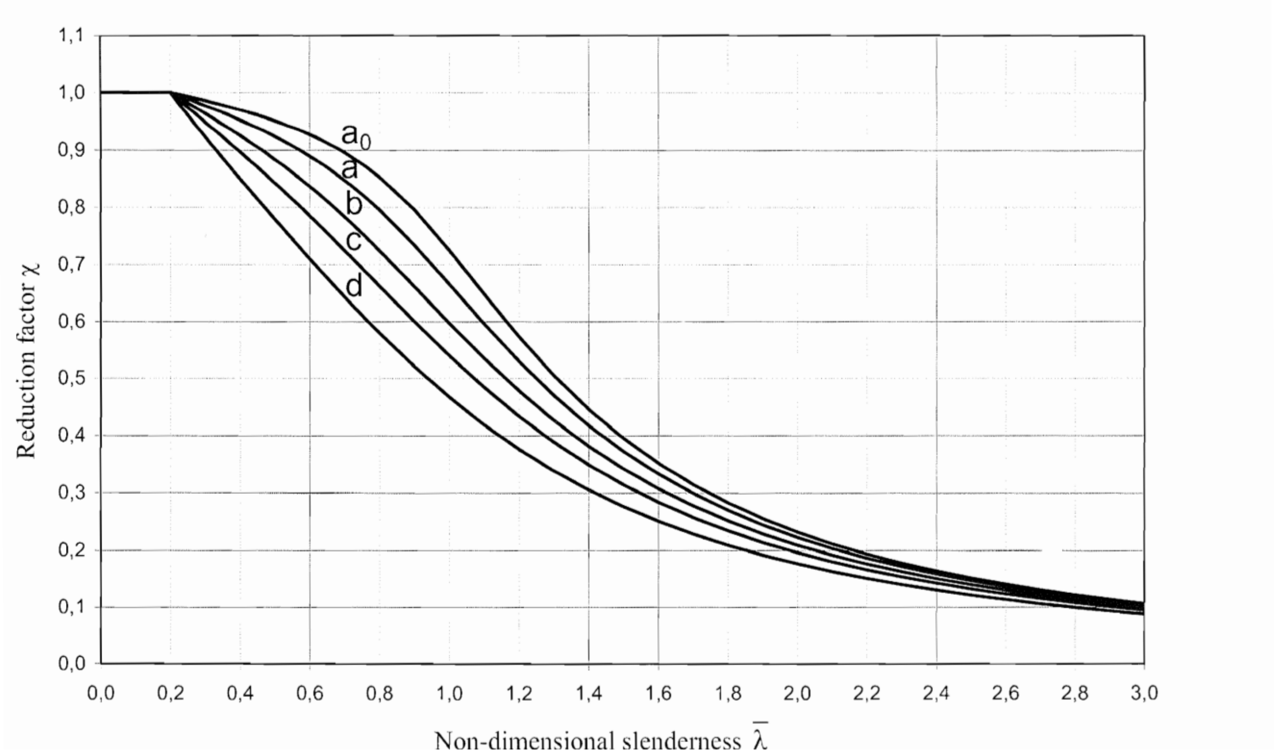


Figure 80.Buckling curves

However, it is possible according to 6.3.1.2 (4) EC3 [4], For slenderness $\lambda \leq$ 0,2 or for $\frac{N_{Ed}}{N_{cr}} \leq 0.04$ the buckling effects may be ignored and only cross-sectional checks apply.

9.4 BENDING MOMENT RESISTANCE

According to chapter 6.2.5 EC3[4], to verify that the design bending moment does not exceed the plastic bending resistance of the cross-section it is necessary to fulfill the following condition:

$$M_{Ed} \leq M_{Rd} \tag{69}$$

Or

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1 \tag{70}$$

Where:

$$M_{c,Rd} = M_{p,Rd} = \frac{W_{pl\cdot} \cdot f_{y\cdot}}{\gamma_{M0}} \tag{71}$$

Where:

- $W_{pl\cdot}$ = plastic section modulus
- $f_{y\cdot}$= yield strength of steel
- γ_{M0}= yield strength of steel

In general, for class 1 or 2 sections, while for class 3 sections $W_{pl\cdot} = W_{el,min\cdot}$, where $W_{el,}$ represents the elastic section modulus, typically smaller than $W_{pl\cdot}$.

According to chapter 6.2.8 (1) EC3[4], if the design value of the shear force V_{Ed} does not exceed 50% of the design plastic shear resistance $V_{pl,Rd}$(see chapter)the design resistance bending moment will not be reduced, otherwise the reduced moment resistance should be taken as the design resistance of the cross-section calculated using a reduced yield strength.

$$f_{y,red} = (1-\rho) \cdot f_{y\cdot} \tag{72}$$

Where, ρ is the reduction factor given by,

$$\rho = \cdot \left(\frac{2V_{Ed\cdot}}{V_{pl,Rd\cdot}} - 1\right)^2 \tag{73}$$

9.5 SHEAR RESISTANCE

According to chapter 6.2.6 EC3[4], to ensure the design, the shear force V_{Ed} in a steel member does not exceed the design shear resistance $V_{pl,Rd}$, according to the following condition:

$$V_{Ed} \leq V_{c,Rd} \tag{74}$$

Or

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1 \tag{75}$$

For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in the following equation:

$$V_{pl,Td} = \frac{A_v \cdot (\frac{f_y}{\sqrt{3}})}{\gamma_{M0}} \tag{76}$$

Where A_v is the shear area, this depends on the type of steel profile used (I or H, channel o hollow section)

Based on 6.2.7 (9) EC3[4], for combined shear force and torsional moment the plastic shear resistance accounting for torsional effects should be reduced from $V_{pl,Rd}$ to $V_{pl,T\,Rd}$ and the design shear force should satisfy:

$$\frac{V_{Ed}}{V_{pl,T\,Rd}} \leq 1 \tag{77}$$

In which $V_{pl,T\,Rd}$ depends on the type of steel profile used.

9.6 UNIFORM MEMBERS IN BENDING:

According to 6.3.2 EC3[4], to verify the lateral-torsional buckling resistance of slender steel beams (I – H sections) subjected to bending about their major axis (y-y), it is necessary to fulfill the following requirement:

$$M_{Ed} \leq M_{b,Rd} \tag{78}$$

Or

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1 \tag{79}$$

The design buckling resistance moment of a laterally unrestrained beam should be taken as:

$$M_{b,Rd} = \chi_{LT}. \frac{W_y . f_y .}{\gamma_{M1}} \tag{80}$$

Where, $W_y .$ is the appropriate section modulus, $W_y . = W_{pl,y}$ for class 1 or 2, and $W_y . = W_{el,y}$ for class 3 sections.

According to 6.3.2.2 (1)EC3[4], unless otherwise specified, for bending members of constant cross-section, the value of $\chi_{LT}.$ for the appropriate non-dimensional slenderness $\lambda_{LT}.$ should be determined from:

$$\chi_{LT}. = \frac{1}{\phi_{LT} + \sqrt{\phi^2{}_{LT} + \lambda^2{}_{LT}}} \leq 1 \tag{81}$$

Where,

$$\phi_{LT} = 0.5\,[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}{}^2] \tag{82}$$

$$\lambda = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} \tag{83}$$

Where, M_{cr} is the elastic moment for lateral torsional buckling.

According to 6.3.2.2 EC3 [4], the imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 63

Table 63.: Recommended values for imperfection factors for lateral torsional

Buckling Curve	**Imperfection Factor α**
a	0.21
b	0.34
c	0.49
d	0.76

The recommendations for buckling curves are given in Table 64

Table 64.: Recommended values for lateral torsional buckling curves for cross sections

Cross-section	Limits	Buckling curve
Rolled I-sections	h/b ≤ 2 h/b > 2	**a** **b**
Welded I-sections	h/b ≤ 2 h/b > 2	**c** **d**
Other cross-sections	-	**d**

According to 6.3.2.2 (4)EC3 [4], for slenderness $\lambda_{LT} \leq \lambda_{LT,0}$ or for $\frac{M_{Ed}}{M_{cr}} \leq \lambda_{LT,0}{}^2$lateral torsional buckling effects may be ignored and only cross sectional checks apply.

9.7 Results:

The design outcomes are shown per element group, with each group assessed using its corresponding dynamic factor. The 4 groups are:

Group 1. All-BT:

- Bottom-chord, [BC]
- Top-chord, [TC]
- Vertical elements, [VER]
- Diagonals elements, [DGA and DGB]
- Top braces, [CTT]
- Bottom braces, [CTB]

Group 2. All ST:

- Stringers, [LG]
- Transversal beam (at stringer level), [R-TR-B]
- Bottom braces (at stringer level), [ST-CT-B]

Group 3. All IB:

- Top transversal beams (interior), [TRS]
- Bottom transversal beams (interior), [TRB]

Group 4. All EB:

- Top transversal beams (at the end), [TRS]
- Bottom transversal beams (at the end), [TRB]

Using the four element groups, a set of load combinations is generated in the 3D model for each group (see Table 52). Combinations are labeled [G][T][nn] (1A01–1A48), where G = 1–4 denotes the element group, T = {A, B, C} denotes the train type (A = LM71, B = SW/0, C = SW/2), and nn is a sequential index. This naming scheme lets the project team quickly select the required combination sets for design and deformation checks

According to this information, now the design results for the most critical load C=SW/2 will be shown for each of the 4 groups of elements, the results of the design are presented in the attachment ANNEX-09_Design result BT-ST-IB-EB, in this is shown the different values of ratios (D/C) that cannot be shown correctly in the following images.

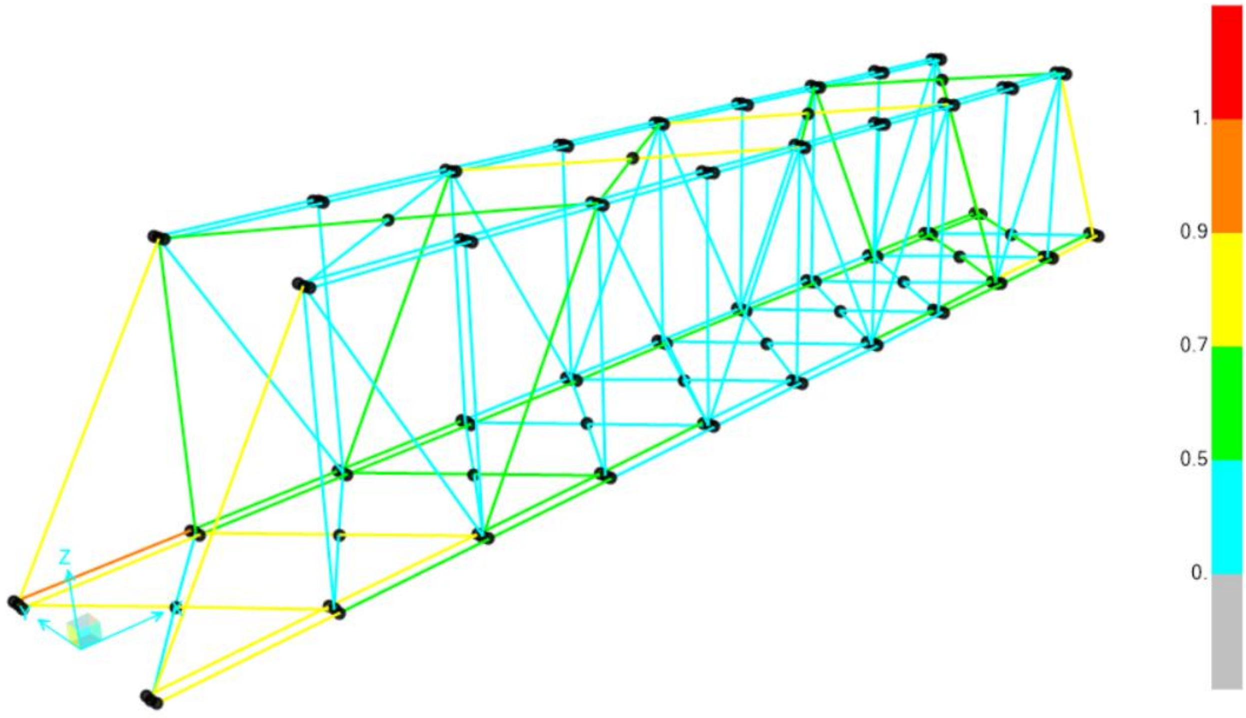


Figure 81. Design results group 1

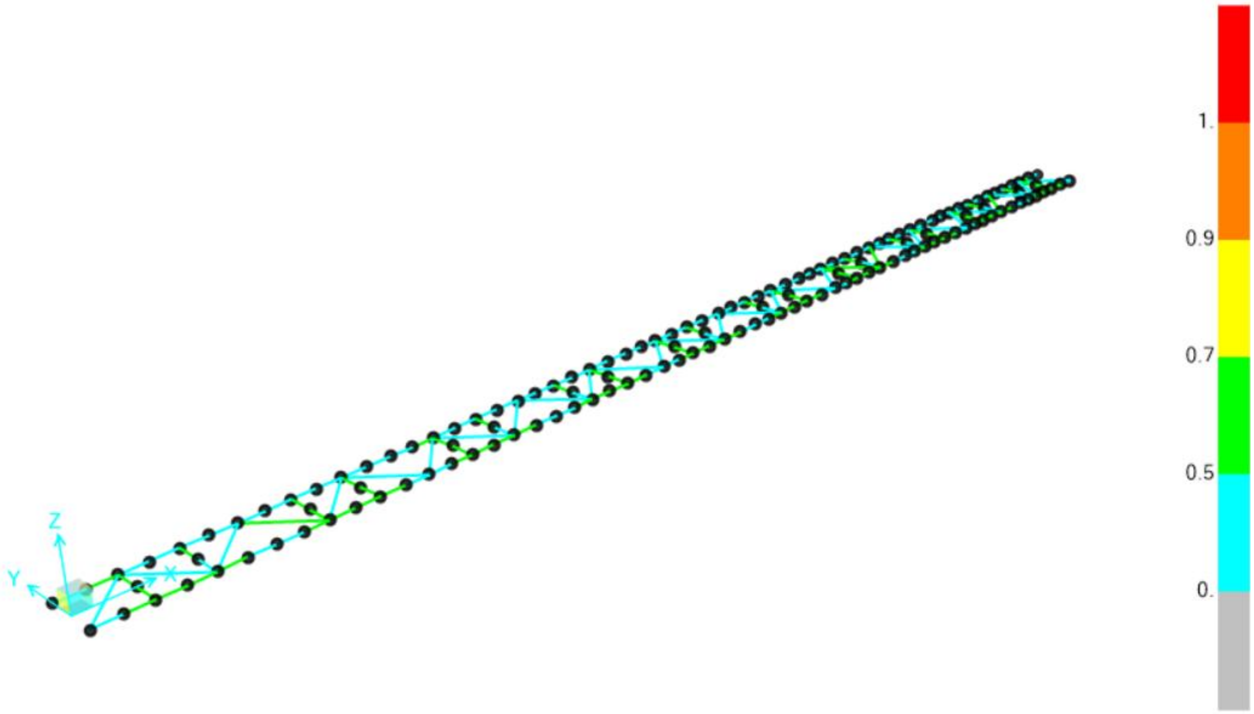


Figure 82. Design results group 2

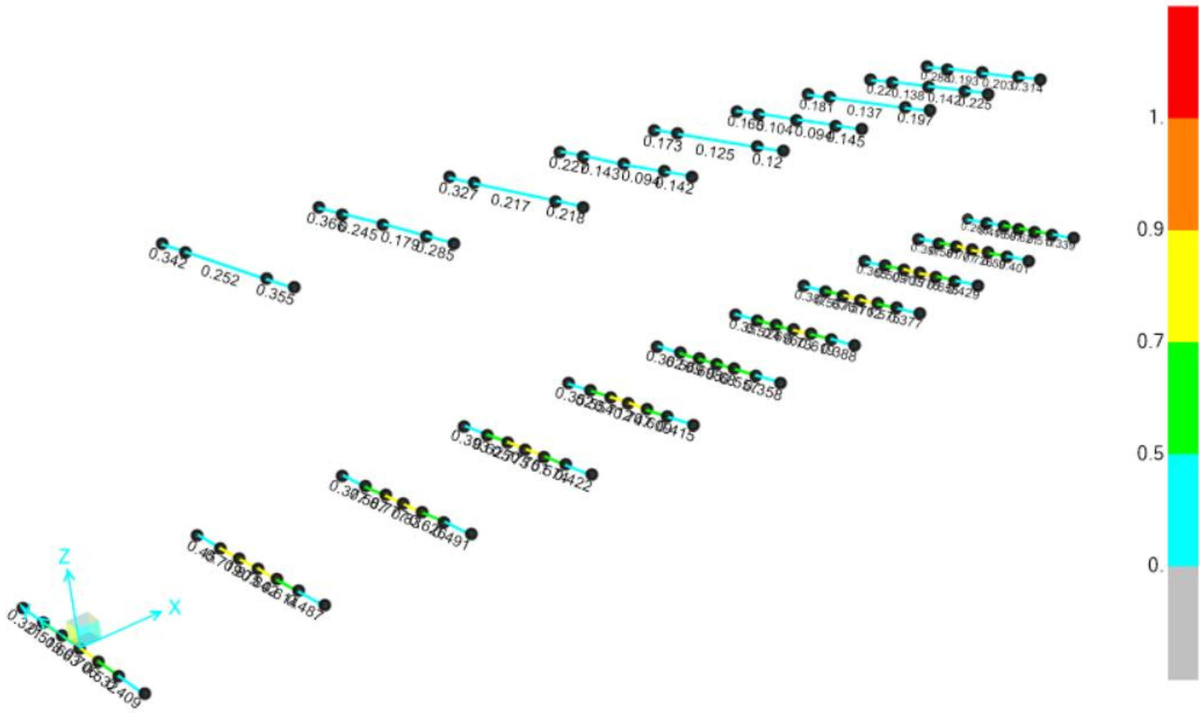


Figure 83. Design results group 3 and 4

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|OPTIMIZATION

The optimization will be carried out through an iterative process using Matlab and SAP2000, through the definition of decision variables, an objective function, and constraints, optimization offers a formal framework for choosing the "best" design from a range of workable alternatives. In structural engineering, it is crucial to precisely define the decision variables, including their permitted ranges and types (continuous parametric dimensions versus discrete catalog choices). This determines the search space's size and shape, how easy it is to solve, and finally the solution's quality and dependability. An explicit recognition of which variables truly drive performance enables more efficient exploration and better management, and interpretation of the results.

This study seeks the optimal combination of 29 structural design variables, which include both standard catalog shapes and parametric dimensions. The primary objective is to minimize total steel weight.

To solve this, it is developed a hybrid optimization framework:

- MATLAB is used to orchestrate the search process.
- SAP2000 provides the finite-element analysis engine to validate the engineering performance of each design.

It is employed a Genetic Algorithm (GA) to efficiently navigate the large design space, which contains a “cost” assigned to the candidate designs. This approach allows us to find an optimal or near-optimal solution that would be practically impossible to obtain through manual trial-and-error.

As is mentioned above, a total of 29 structural design variables will be defined and then they will be optimized in each iteration. The primary objective of this analysis is to identify the section properties that satisfy the requirements for both strength and deformability, while minimizing the key parameter: the total weight (W).

As outlined in ANNEX 10 – SAP Variables, some variables will remain fixed throughout the analysis, whereas others will be allowed to vary within defined upper and lower bounds.

Table 65. Input data, general parameters

GENERAL PARAMETERS					
Inpunt data			Lower Bound	Upper Bound	Δ
Bridge Length	L	43			
Gantry height (Recommended)	d_g	6.85			
Type of line		O Bridge x 25000V AC			
Width of train gantry	G_W	4.5			
Free width (minimum)**	N_{bw}	0.30	0.25	0.50	0.05
Track height	$d_{L,Track}$	0.35			
Vertical elements longitudinal interaxis	d_v	4.3			
Total Internal Bridge height	d_i	7.20			
Total Interaxis Bridge height**	d_e	7.80	7.50	9.60	0.10
Total Interaxis Bridge Width**	b_i	5.60	5.50	6.30	0.10

*Units (m)

The user will be required to provide the input data specified in Table 65, where the following values could be changed according to the project demands:

Table 66. Values that must be selected by the user

Bridge Length	In this analysis, the bridge length is defined as 43 m; however, this parameter should be determined specifically for each individual analysis to ensure accuracy and applicability
Type of line	The type of line must be selected by the user, according to the different option in the code, according to the option selected there is a minimum value of the gantry height.
Free width (minimum)	These values can vary between the lower and upper bounds indicated, it must be selected specifically for each individual analysis.
Total Interaxis Bridge height	These values can vary between the lower and upper bounds indicated, it must be selected specifically for each individual analysis.
Total Interaxis Bridge Width	These values can vary between the lower and upper bounds indicated, it must be selected specifically for each individual analysis.

The other values in Table 65, are fixed in the values given by the normative or depend on the values selected by the user according to their description in Table 66, for this case and as it was mentioned at the beginning of this document the objective focuses exclusively on reducing the self-weight of the bridge by decreasing the structural sections of its main members while keeping the overall geometry fixed, however it is shown that these values must be also evaluated carefully.

As is mentioned before, the optimization will be done exclusively on the section used to build the structure.

Table 67 and Table 68 present the required input values for the different profile sections used in the 3D model. The profile sections are categorized into two main groups as it is mentioned along different chapters of this document, consistent with the fatigue analysis: *rolled or extruded* sections and *welded built-up* sections. For each category, the table provides the corresponding SAP label, the relevant dimensions, and the upper and lower bounds for each parameter.

Table 67.Welded built-up sections data

		GEOMETRY			
WELDED BUILT-UP PROFILES			Lower Bound	Upper Bound	Δ
BOTTOM Chord					
SAP name		BC			
Outside stem (t_3)**	[mm]	665	500	800	50
Outside flange (t_2)	[mm]	450			
Flange thickness (t_f)**	[mm]	35	14	40	2
Stem thickness (t_f)**	[mm]	35	34	40	2
Free space between elements (s)	[mm]	450	400	700	50
TOP Chord					
SAP name		TC			
Outside stem (t_3)**	[mm]	550	400	800	50
Outside flange (t_2)	[mm]	450	300	600	50
Flange thickness (t_f)**	[mm]	35	18	40	2
Stem thickness (t_f)**	[mm]	36	36	40	2
Flange to Flange distance (s)	[mm]	0			
DIAGONAL at the support					
SAP name		DGA			
Outside height (t_3)	[mm]	450			
Top flange width (t_2)**	[mm]	450	350	450	50
Top flange thickness (t_f)**	[mm]	25	18	40	2
Web thickness (t_w)**	[mm]	15	16	22	2
Bottom flange width (t_{2b})	[mm]	450			
Bottom flange thickness (t_{fb})	[mm]	25			
Fillet Radius (r)	[mm]	0			
DIAGONAL truss elements					
SAP name		DGB			
Outside height (t_3)	[mm]	450			
Top flange width (t_2)**	[mm]	350	300	450	50
Top flange thickness (t_f)**	[mm]	25	14	40	2
Web thickness (t_w)**	[mm]	15	16	22	2
Bottom flange width (t_{2b})	[mm]	350			
Bottom flange thickness (t_{fb})	[mm]	25			
Fillet Radius (r)	[mm]	0			
BOTTOM TRANSVERSAL					
SAP name		TRB			
Outside height (t_3)**	[mm]	665	550	800	50
Top flange width (t_2)**	[mm]	450	350	650	50
Top flange thickness (t_f)**	[mm]	40	18	40	2
Web thickness (t_w)**	[mm]	25	22	30	2
Bottom flange width (t_{2b})	[mm]	450			
Bottom flange thickness (t_{fb})	[mm]	40			
Fillet Radius (r)	[mm]	0			

Table 68. Rolled or extruded sections data

ROLLED or EXTRUDED PROFILES			Lower Bound	Upper Bound	Δ
VERTICAL truss elements (Rolled Profile)					
SAP name		VER			
Type of element**		HEB	HE300B	HE600B	
Element selected**		450	300	600	
h	[mm]	450			
b	[mm]	300			
STRINGER (Rolled Profile)					
SAP name		LG			
Type of element**		HEM	HEM300	HEM600	
Element selected**		450	340	620	
Profile height (h)	[mm]	478			
TOP TRANSVERSAL (Rolled Profile)					
SAP name		TRS			
Type of element**		HEA	HEB	HEB	
Element selected**		240	200	400	
Profile height (h)	[mm]	230			
TRANSVERSAL at the stringer (Rolled Profile)					
SAP name		TRL			
Type of element**		HEA	HEB200	HEB400	
Element selected**		360	200	400	
Profile height (h)	[mm]	350			
TOP BRACE (Rolled Profile)					
SAP name		CTT			
Type of element		DA	L	L	
Element selected		125x125x12	100x100x10	180x180x12	
Profile dimension (L)**	[mm]	125	100	180	
Profile thickness (t)**	[mm]	12	10	20	
BOTTOM BRACE (Rolled Profile)					
SAP name		CTB			
Type of element		DA	L	L	
Element selected		160x160x12	100x100x10	180x180x12	
Profile dimension (L)**	[mm]	160			
Profile thickness (t)**	[mm]	12	10	20	
BRACE at the stringer (Rolled Profile)					
SAP name		CTL			
Type of element		DA	L	L	
Element selected		100x100x12	80x80x8	150x150x12	
Profile dimension (L)**	[mm]	100			
Profile thickness (t)**	[mm]	12	10	20	

In order to simplify the analysis and to reduce the number of variables to introduce in MATLAB, there are some variables that as for the Input data in Table 65 are fixed and others will be allowed to vary within defined upper and lower bounds.

Now it will present the assumptions considered to define which variables are fixed or variable.

Geometry constraints:

1. **Profile section restrictions:**
    - The profile sections considered in the optimization are limited to those used in the original design. Specifically, if an element was originally defined with a rolled profile, its optimized section must also be a rolled profile. Likewise, elements with welded built-up sections in the original design will remain restricted to welded built-up sections during optimization.
    - For rolled sections, the optimization will be limited to three specific profile types: HEB, HEM, and double-angle sections.
    - The value given for ***Outside flange (t_2)*** in bottom and top chord and ***Outside height (t_3)*** in diagonal elements (at supports and for the internal truss system) will depend on the parameter ***height (h)*** of the vertical element.
    - The lower bound of the ***Flange thickness (t_f)*** and ***Stem thickness (t_f)*** in the bottom chord will be limited to the ***Outside flange (t_2)/30*** and ***Outside height (t_3)/20*** respectively, while the maximum will be set in 40 mm for both cases as it is shown in Table 67.
    - The lower bound of the ***Flange thickness (t_f)*** and ***Stem thickness (t_f)*** in the top chord will be limited to the ***Outside flange (t_2)/24*** and ***Outside height (t_3)/15*** respectively, while the maximum will be set in 40 mm for both cases as it is shown in Table 67.
    - The value of the upper bound for the diagonal elements (at the support and internal truss elements) will depend on the ***Outside height (t_3)*** selected.
    - The lower bound of the ***Top flange thickness (t_f)*** and ***Web thickness (t_w)*** in diagonal elements (at the support and internal truss elements) will be limited to the ***Top flange width (t_2)/24*** and ***Outside height (t_3)/30*** respectively, while the maximum will be set in 40 mm for both cases as it is shown in Table 67.
    - The lower and upper bound for the ***Outside height (t_3)*** in the bottom transversal elements will depend on the ***Total Interaxis Bridge Width/10*** and ***Total Interaxis Bridge Width/7*** respectively.
    - The lower and upper bound for the ***Top flange width (t_2)*** in the bottom transversal elements will depend on the ***Lower bound/1.5*** and ***Upper bound/1.2*** respectively.
    - The lower bound of the ***Top flange thickness (t_f)*** and ***Web thickness (t_w)*** in bottom transversal elements will depend on ***Top flange width (t_2)/24*** and ***Outside height (t_3)/30*** respectively, while the maximum will be set in 40 mm and 30 mm respectively as it is shown in Table 67.

The remaining boundaries not specified in this section are defined empirically.

With these conditions the number of variables that the user must introduced during the iteration process are the ones with (**) in Table 65, Table 67 and Table 68, for a total of

29 variables to be modified, however as it is mentioned along the different chapters of this document the optimization will be done exclusively for the section geometry and with this the total number of variable will be **26.**

As is mentioned at the beginning of this chapter, the optimization will be done according to a system that connects the computational power of MATLAB with the finite element analysis engine of SAP2000.

The system uses a Genetic Algorithm (GA) to intelligently explore the gigantic space of possible solutions (much larger than trillions of combinations) and find an optimal or near-optimal solution that cannot be done manually.

As in the problem some variables are discrete (HEB 200, HEB 220) and others are continuous (a height that could be 500mm, 501mm, 502mm...). Traditional methods fail with this type of "mixed variables" or get stuck in "local optima" as is mentioned by MATLAB guide that basically is a good solution, but not the best one.

10.1 How does GA work?

The GA is excellent for navigating these complex problems and finding the global optimum.

The problem can be solved according to how GA works, basically the GA creates a population of designs for this analysis it was defined for a X number of individuals (population). Each individual is a set of 26 variables that define a complete structure.

In MATLAB the GA tell the Model3D.m function to evaluate each individual in the population to select the most useful ones, this process of selection is done through a variable called "Cost" defined and according to the MATLAB manual as:

$$\text{total\_cost} = \text{total\_weight} * (1.0 + \text{penalty\_ratio} + \text{penalty\_displ} + \ldots) \tag{84}$$

This variable "cost" will be basically the weight given to each iteration.

10.2 Constraints

In addition to the variables, it is essential to define all the constraints and verifications the model must satisfy to find the optimal solution.

The verifications that must be fulfilled are:

- Dynamic Check
- Horizontal Check (Radius R)
- Angular Check (Rotation Theta)
- Vertical Check (L/600)
- Fatigue Check
- D/C Ratios Check (Design/Capacity Ratios)
- Skew Check

Basically, these constraints are the same that were already evaluated manually for the main model in the chapters above.

There are two clearly defined options:

- If a design is lightweight and meets all 7 checks, its penalties are 0 and its cost is low, for example, the variable Cost will be assumed an initial value Cost =Weight=15000 (selected randomly) and without any penalty=0 Cost =15000*(1.0+penalty) = 15000, where 15000 is the minimum value for this example.
- If a design is super lightweight but fails (for instance, a ratio > 1.0), it receives a huge penalty, resulting in a variable "Cost" > 15000.

Following these two options GA will "kill" (discard) the individuals with high cost (those that failed) and selects the low-cost ones (lightweight and safe) to be something called "parents".

These "parents" will be mixed in a process called "crossover and mutation", basically the parents will create a new generation of "children," with random changes to explore new ideas.

Finally, the process will be repeated hundreds of times. Over time, the population "evolves" toward progressively lighter designs that still meet all the constraints, and finally, after the number of generations defined we will get the optimal model.

The following are the values defined for this analysis:

- Individuals = 50
- Generations = 10

Finally, with this, MATLAB will run a total of 500 iterations.

10.3 MATLAB and SAP2000

To run this analysis in MATLAB 2 scripts were defined:

- Optimization.m
- Model3D.m

This **Optimization.m** script is the starting point. It runs only once, and its job is to set up the entire optimization environment.

- Define global variables: Create the containers that will let Model3D.m access the configuration.
- Define the main configuration:
  - Connection to SAP2000, stores and launch SAP2000.exe and then the path to the original SAP file XXX.sdb, also connect to SAP2000 if there is already a instance open.
  - API, COM or NET connection, stores the path to the SAP2000 API.
  - Defines the height of the catalogue sections for each 'VER' profile (e.g, 'HE500B') and the properties modifiers for TRSV and TRBM.
  - Defines the lists of combinations used for the displacements verifications: skew_combinations, horizontal_combinations, angular_combinations, vertical_check_combinations, and the numeric limits displacement_limit, skew_limit, R_limit, angular_variation_limit, vertical_limit).
  - Call the function GA () and after it finishes, prints the best combination of indices found and translates it into readable values (e.g., VER: HE500B, BC_h: 550.0 mm).
- Reads the Excel file ANNEX-10_SAP Variables, in this file it is defined a total of 19 sheets: given by the following names:
  - Parametric: It has 22 continuous variables, the upper and lower bound and its step.
  - Catalogues: It has the 4 discreet variables, the minimum and the maximum catalogue section
  - Dependents: It has variables which depend on the value assumed for other variables.
  - TRSV_Modifiers:It has the property modifiers used for each TRSV beam, according to the section selected.
  - New_sections: it has information about initial assignment, ensure all the frames (VER1...VER18, BRT1...BRT8, etc.) are assigned to the correct section properties ("VER", "TC", etc.) before the optimization starts
  - FramesForResults: It has the name of the frames used to get design ratios.
  - FatigueParams:It has limits, ϕ factors and others used to calculate fatigue resistance.
  - FramesWBTForFatigue: It has the name of the welded built-up frames defined in BT group used to get fatigue strength.

- FramesRBTForFatigue: It has the name of the rolled or extruded frames defined in BT group used to get fatigue strength
- FramesWSTForFatigue: It has the name of the welded built-up frames defined in ST group used to get fatigue strength.
- FramesRSTForFatigue: It has the name of the rolled or extruded frames defined in ST group used to get fatigue strength
- FramesIBBForFatigue: It has the name of the bottom welded built-up frames defined in IB group used to get fatigue strength.
- FramesEBBForFatigue: It has the name of the bottom welded built-up frames defined in EB group used to get fatigue strength.
- FramesIBTForFatigue: It has the name of the top rolled or extruded frames defined in IB group used to get fatigue strength
- FramesEBTForFatigue: It has the name of the top rolled or extruded frames defined in EB group used to get fatigue strength
- JointsForSkew: It has the list of joints used to calculate the skew verification.
- JointsForHorizontal: It has the list of joints used to calculate the horizontal verification.
- JointsForAngular: It has a list of joints used to calculate the angular verification.

This **Model3D.m** script does all the heavy lifting. It's called hundreds of times, once for each design the GA () decides to try.

The main input to these functions is a vector un numbers e.g., [7, 5, 2, ...] from Optimization.m, and its output is basically the value of equation [(84], total_cost (a single number).

- Prints to the console the geometry selected for this
- Assigns catalog profiles through the API functions (PropFrame.ImportProp, FrameObj.SetSection, etc.).
- Calls the API functions to create the welded built-up sections through the API functions (PropFrame.SetISection, SetTee, SetDblAngle, etc.)
- Run the analysis through the API functions (Analyze.RunAnalysis).
- Runs all post-analysis checks:
    - **Weight:** Calls API function AnalysisResults.JointReact for the 4 support joints (SP1-A, etc.) and sums the |FZ| of 'DEAD'.
    - **Vertical Displacement:** Calls API function JointDispl for the middle point (F60) and the combo of permanent loads (COMB01-PER-LOAD), to verify the dynamic behavior.
    - **Skew Displacement:** Calls API function JointDispl, used to call the displacement in the stringers, a total of 16 times (once per combo), and within each, iterates 122 times (once per joint) to get all U3, the value must be:

| (U3_j+1 - U3_k+1) - (U3_j - U3_k) | < 75mm.

- **Horizontal Displacement:** Calls API funciton JointDispl, 24 times (once per combo), and within each, iterates 80 times (once per joint) to get the max U2.

R > 6000000mm

- **Angular Displacement:** Calls API function JointDispl, used to call the horizontal displacement 24 times (once per combo), and within each, iterates 2 times (once per joint) to get the U2.

|ϕ| = |(U2_J1 - U2_J2) / L| < 0.0020

- **Vertical Displacement (F60):** Calls API function JointDispl 96 times (once per combo) to get the max U3 at F60.

|U3_max| < (L/600)

- **Fatigue:** This verification is more complex due to there is not a direct way to as for the stresses acting on a element defined as a Frame, for this it was analyzed in the following way; for each frame, the script retrieves its geometric properties area(A), section moduli (S_{22}) and (S_{33}) and the internal forces, axial force $P$and bending moments $M_2$and M_3.for the selected fatigue combination.

  With this data the normal stress $S_{11}$is approximated as the sum of axial and bending contributions about the two principal axes:

$$S_{11}^{\text{approx}} = \frac{\mid P \mid}{A} + \frac{\mid M_2 \mid}{S_{22}} + \frac{\mid M_3 \mid}{S_{33}}. \tag{85}$$

  Finally, it is selected the highest S_{11} found in the frame, it is adjusted using reliability factors read from the main Excel file.

$$S_{11}^{\text{final}} = S_{11}^{\text{approx}} \times 1000 \times \phi_2 \times \lambda \tag{86}$$

  The maximum S_{11}^{final}across all frames in the current worksheet is compared to the corresponding fatigue limit for that worksheet:

  - $S_{11}^{\text{final}} >$ fatigue_limit

- Runs the design, calls API function SteelDesign.StartDesign().
- Calls GetSummaryResults in a loop for the 166 frames in the FramesForResults list.

- Computes the ratios (D/C) for each of the 166 frames and then classifies by ranges.
    - x >1,0
    - 1≤ x <0.9
    - 0.9 ≤ x<0.7
    - 0.7 ≤ x<0.4
    - 0.4 ≤ x<0.1
    - 0.1 ≤ x
- Calculates the 6 penalties: ratio, displacement, skew, horizontal, angular, vertical, using equation [(84]
- Prints the iteration summary.

10.4 Penalty method:

The penalty method considering the variable “cost” is the point where the genetic algorithm (GA) takes decisions, it can automatically ignore “invalid” designs by giving them a very high cost and focus the search on “valid” designs to find the one with the lowest total weight, that can be translate in a lower “cost” as is mentioned above.

For this analysis and according to how MATLAB describes a penalty function:

“If the individual is feasible, the penalty function is the fitness function. If the individual is infeasible, the penalty function is the maximum fitness among feasible members, plus a sum of the constraint violations.”[7]

There are two types of penalties according to this:

- Feasible: It adds a penalty proportional to the amount of violation. As an example of this, consider two infeasible candidates:
    - Design A: max ratio (D/C) = 2.5 (substantial violation),
    - Design B: max ratio (D/C) = 1.0 (minor violation).

  Let the penalty function be quadratic in the degree of violation:

  $$P(\text{ratio}) = 10^6\ (\text{ratio} - 1)^2 \text{for ratio} > 1.$$

  Then:

  $$P_A = 10^6(2.5 - 1)^2 = 2{,}250{,}000, P_B = 10^6(1.1 - 1)^2 = 10{,}000.$$

  Because the total objective incorporates this penalty, candidate B attains a much smaller penalized cost than candidate A. Consequently, the genetic algorithm (GA) preferentially selects B, thereby steering the search toward the boundary of feasibility even though B itself is infeasible.

  The following are the variables which have a feasible penalty:

    - **D/C Ratio check:** penalty_ratio = $1\text{x}10^6$ · (max_ratio_dc - 1.0) ^{2}
    - **Skew check:** penalty_skew = $1\text{x}10^4$ · skew_violations_count
    - **Fatigue check:** penalty_fatigue = $1\text{x}10^4$ · fatigue_violations_count

- Infeasible: It adds a huge or fixed cost; it does not depend on the amount of violations as in the analysis of the dynamic behavior, as an example if:

$$L.Inf < X < L.Sup$$

Where,
  - Lower bound = 2.54
  - Upper bound = 5.96

If the value of X is not between these boundaries, for the dynamic factor it will be assigned a fixed value, for this analysis the fixed value is:

$$Cost = 1x10^{7}$$

The following are the variables which have an infeasible penalty:

  - **Dynamic check:** penalty_displ = $1x10^7$, it applies if X_calculated is outside the allowed range.
  - **Horizontal check (Radius R):** penalty_horizontal = $1x10^7$, it applies if R_value is below the limit.
  - **Angular check (Rotation θ):** penalty_angular = $1x10^7$, it applies if max_theta_abs exceeds the limit.
  - **Vertical Check (L/600):** penalty_vertical = $1x10^7$, it applies if max_U3_vertical_abs exceeds the limit.

After the genetic algorithm (GA) framework, variable definitions, and penalty scheme were explained, the analysis for the "BT" element group was carried out. A total of 500 iterations were executed for this group. The corresponding results are presented in the next chapter.

11

|RESULTS

Following the execution of the 500 planned iterations, this chapter reports the key outcomes of the optimization.

It is important to note that, as mentioned in the previous chapters, the results presented here concern the following analysis case:

<u>Design Group:</u> Element group "ALL BT". The elements in this group are part of the main bracing system of the structure. The elements to be analyzed include:

- VER: 18 vertical elements
- DGA, DGB: 20 diagonal elements
- CTT, CTB: 56 upper and lower bracings
- BC: 40 bottom chords
- TC: 32 top chords

The total number of elements in the group is 166 frames.

<u>Combination Group:</u> The selected combination group depends on the element group to be designed. As mentioned previously, there are 4 element groups, which depend exclusively on their dynamic factor. For the selected "ALL BT" group, the "1C" group combinations will be used, where "C" represents the most critical variable load SW/2, as mentioned in paragraph 9.7 of this report.

For this group and all others, there are a total of 48 ULS combinations for checking design stresses and solicitations, and a total of 34 SLS combinations for deformation analysis.

In summary, the MATLAB code must run approximately 540 iterations. In each iteration, all elements belonging to the selected group will be designed, considering the specific combinations for the selected group and load type.

As a result of the MATLAB code, a .csv file is exported containing the results of each check, as well as the final cost value assigned as a result of the number of penalties applied to each iteration.

The results of the 540 iterations can be observed in the attached file ANNEX-11_GA Results. In this file, there are a total of 16 columns with the following information:

- **# Iteration:** Indicates the iteration number, 1-542
- **Weight (kN):** Indicates the weight calculated for each iteration (i). It is the sum of the vertical reactions at the supports for the DEAD load case.
- **Max_ratio (D/C):** Indicates only the maximum value found for the (D/C) ratio. If this value is "<1", it indicates that all elements have been designed correctly; if, on the other hand, it is ">1", it indicates that the design failed for at least one element.
- **Max_U3(mm):** Indicates the maximum vertical displacement found for the COMB01-PER-LOAD load combination in the middle point F60.
- **n0:** Indicates the calculated value for the natural frequency of the structure for the COMB01-PER-LOAD load combination.
- **Skew_infringements:** Indicates the total number of values found that exceed the limit.
- **Max_U2(mm):** Indicates the value of the maximum horizontal displacement found for the SLE-CHR group of combinations.
- **R(mm):** Indicates the value of the radius of curvature calculated for iteration (i).
- **Max_θ:** Indicates the value of the angular variation calculated for iteration (i).
- **Max_U3(mm):** Indicates the value of the vertical displacement found for the SLE-CHR group of combinations.
- **Dinamico check:** Indicates if the found n_0 value complies with the limits indicated for this verification.
- **Horizontal check:** Indicates if the found Max_U2 value complies with the limits indicated for this verification.
- **Angular check:** Indicates if the found Max_θ value complies with the limits indicated for this verification.
- **Vertical check:** Indicates if the found Max_U3 value complies with the limits indicated for this verification.
- **Fatigue check:** Indicates the total number of groups that do not comply with the fatigue verification (BT, ST, IB, or EB).
- **Costo:** Indicates the total cost value assigned to iteration (i) according to the weight and the number of verifications passed.

As mentioned in the previous chapter, this analysis proposed a total of 50 individuals (population) and 10 generations for a total of 500 iterations. With this, the results of iterations 1-50 presented in the attached file are part of the first generation, the following 51-100 are part of the second, and so on.

After the analysis of the 500 iterations, MATLAB indicates which one was the most optimal, as well as the sections used for that iteration (i). This makes it possible to compare the sections used in the original model with those indicated by MATLAB.

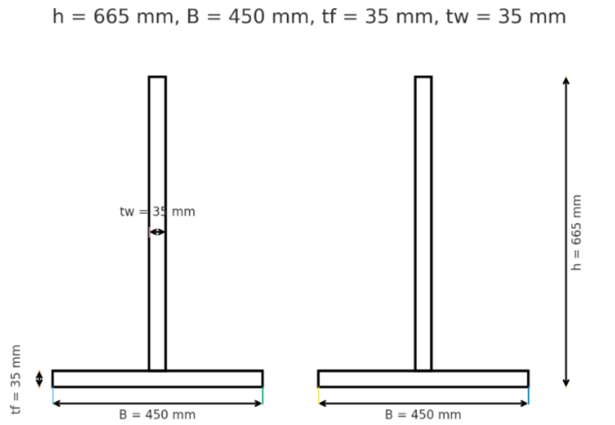
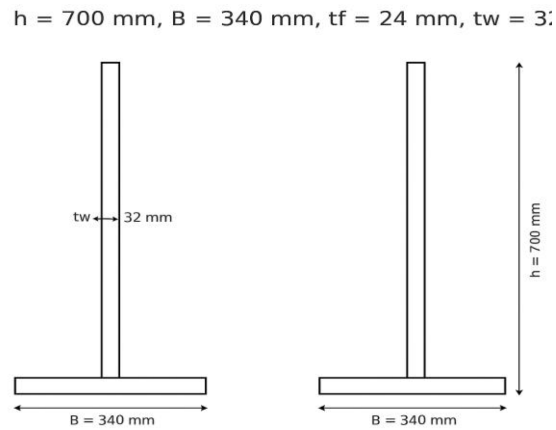
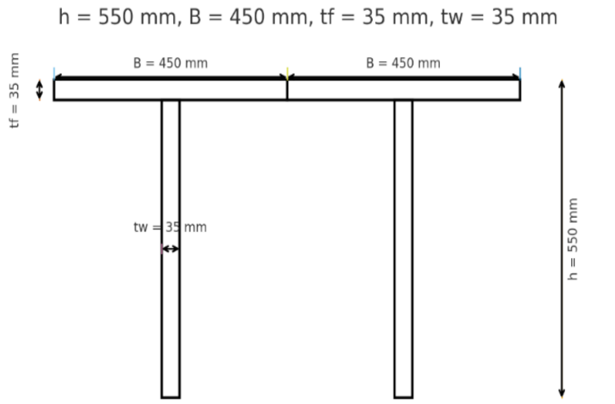
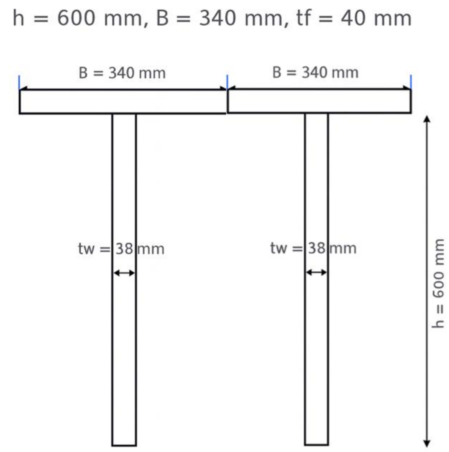
For this case and as is mentioned many times before the results of the analysis are just for the group selected "ALL-BT".

The difference between the original section and the ones that MATLAB assigned along the iterations for the elements in "ALL_BT" group are the following:

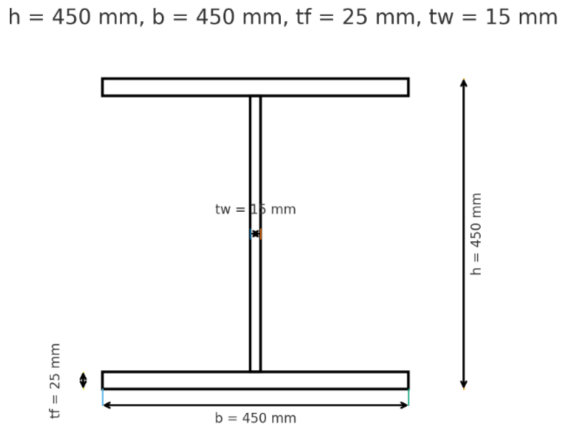
Rolled or Extruded sections

Original	**Optimal**
Vertical (VER): HE450B	Vertical (VER): HE340B
Top brace (CTT): DA125x14	Top brace (CTT): DA150x20
Bottom brace (CTB): DA180x15	Bottom brace (CTB):DA180x18

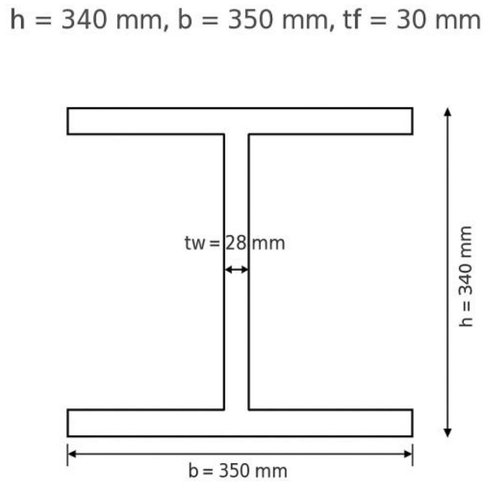
Welded built-up sections:

<table>
<tr><th>Original</th><th>Optimal</th></tr>
<tr><td>Bottom chord (BC)

</td><td>Bottom chord (BC)

</td></tr>
<tr><td>Top chord (TC)

</td><td>Top chord (TC)

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

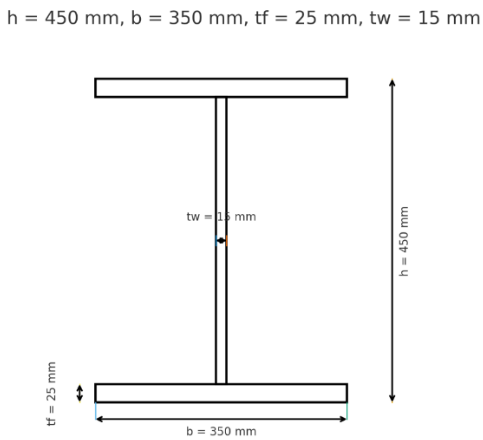
Diagonal (DGA)



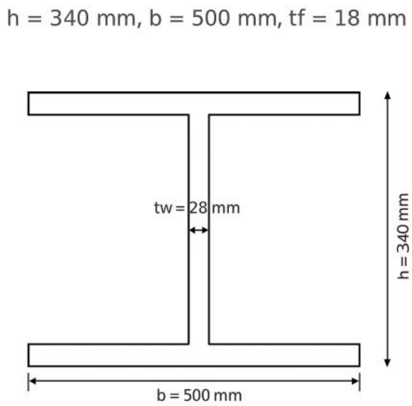
Diagonal (DGA)



Diagonal (DGB)



Diagonal (DGB)



The total time it took MATLAB to analyze, design, and verify the 500 iterations initially proposed for this thesis is approximately 715432 seconds (By Tik-Tak), around 199 hours, for a total of ≅ 8.3 days.

The time MATLAB takes to perform one iteration can be broken down, where MATLAB spends approximately:

- Analysis: ≅ 1.5 minutes -
- Verification: ≅ 7.5 minutes, around the 80% of the time is dedicted o the fatigue verification.
- Design: ≅ 13 minutes -

The total time per iteration is ≅ 22 minutes, where approximately the 60% of the time is dedicated to using the SAP2000 design module.

It is possible to track how the different penalties evolve over the optimization iterations (by generation) however, the most important ones, which drive the optimization process, are the Dynamic Effects, the D/C ratio, the skew verification, and the fatigue check.

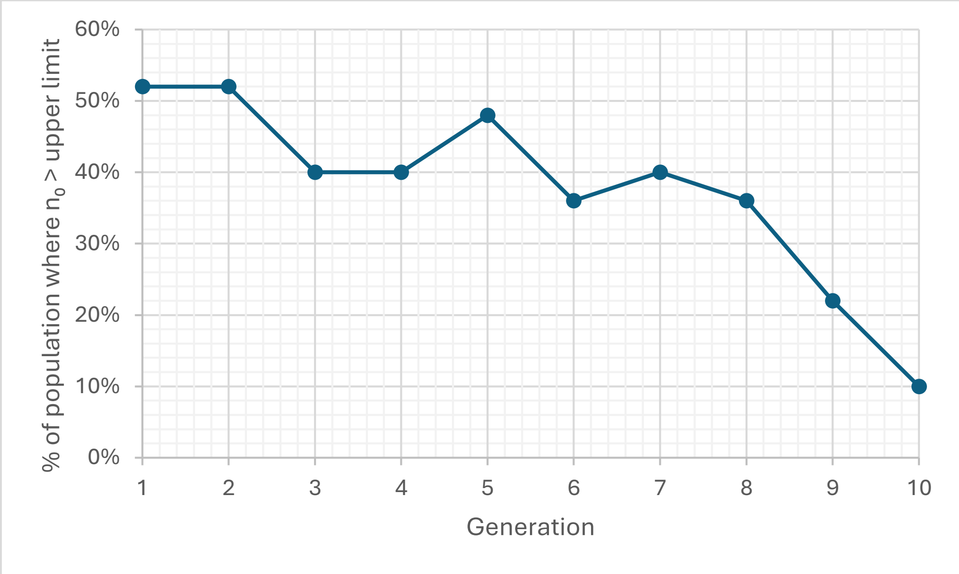


Figure 84. Behavior of dynamic effects

Figure 84 shows the n_0 values that are above the limit for a length of 43 m. Since there were no values below the lower bound, only those exceeding the upper limit are displayed. A general improvement is observed in the total percentage of values complying with the established limits.

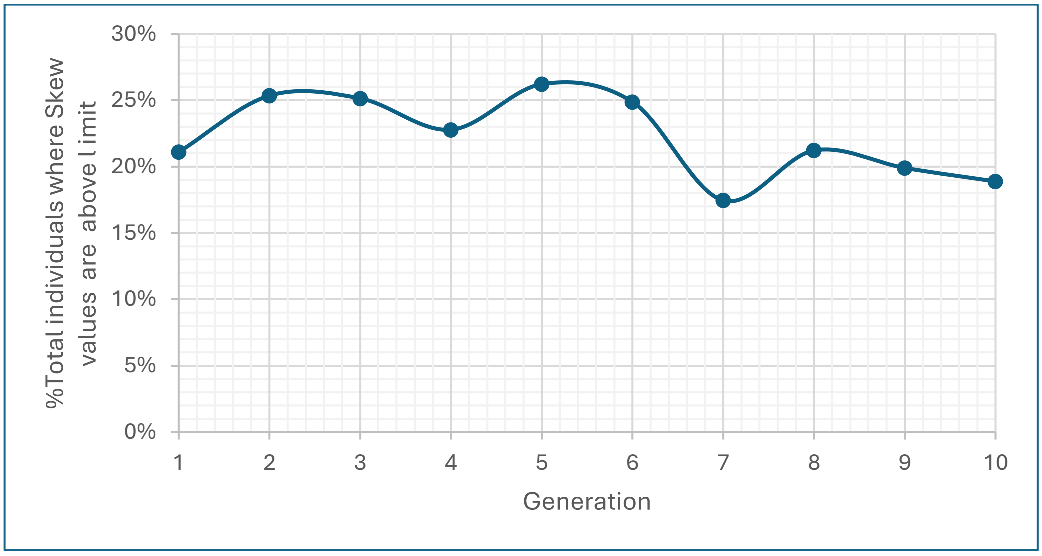


Figure 85. Behavior of skew verification

Figure 85 shows the percentage of the total amount of individuals in skew verification that are above the limit of a length of 43 m.

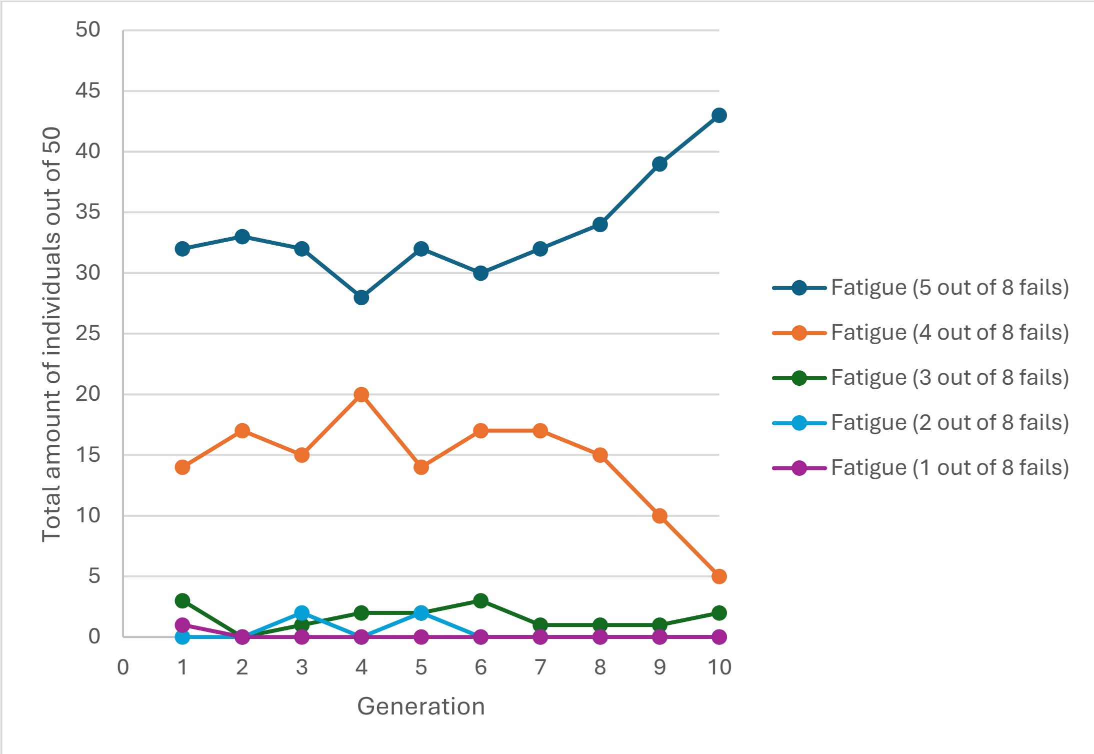


Figure 86. Behavior of Fatigue verification

Based on the 50 iterations, the variable number of fatigue failed groups per iteration was analyzed.

The results in

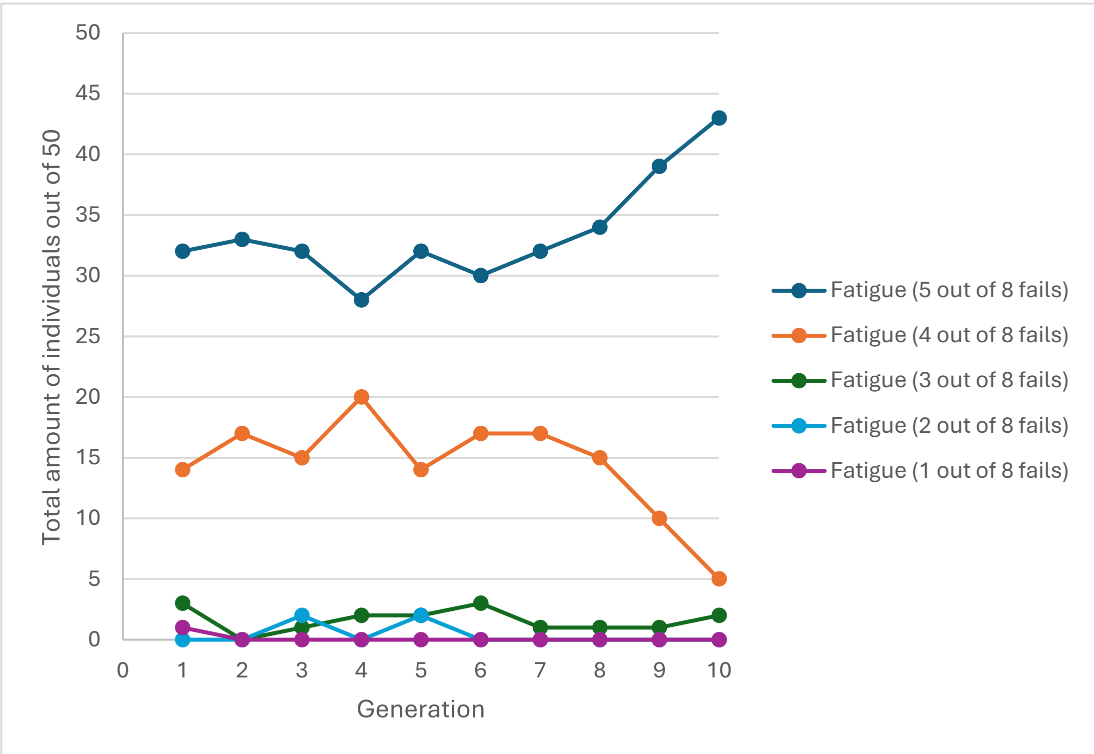


Figure 86 show that the probability of observing 5 failing groups (out of 8) is higher than 1 failing groups (out of 8) within the evaluated set. This demonstrates that non-compliance is not only frequent but also of high magnitude.

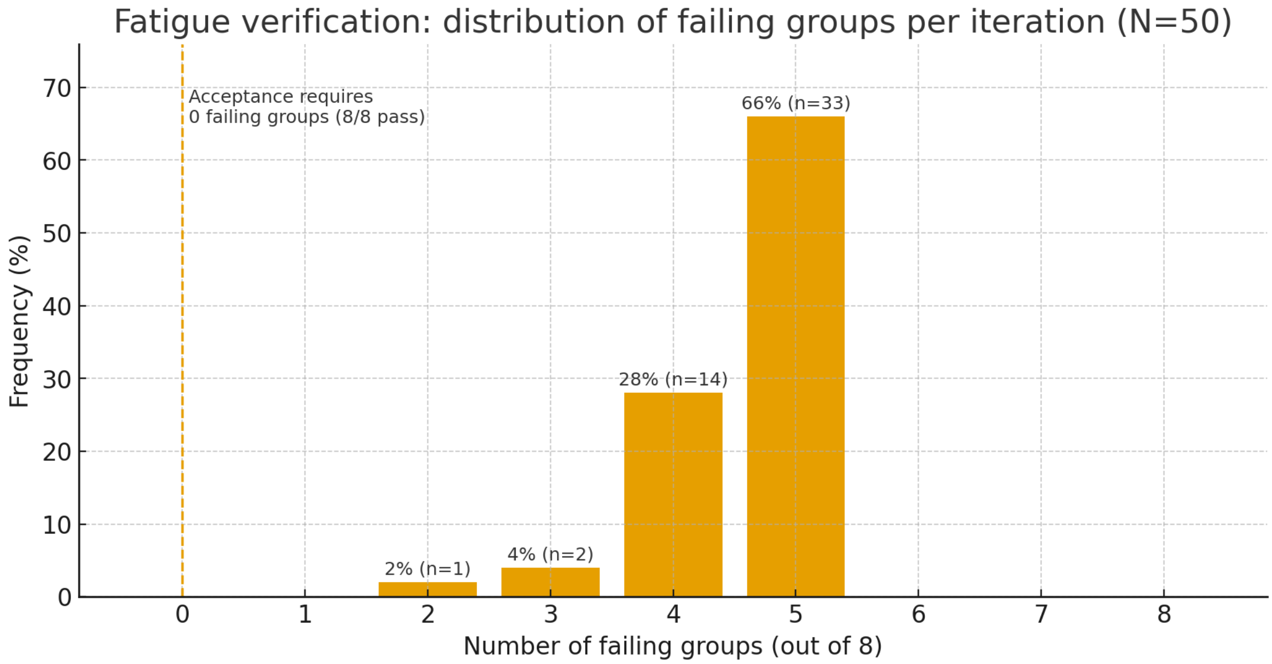


Figure 87.Distribution of failing groups per iteration (N=50)

Over 50 iterations, the scenario of 5 failing groups (out of 8) occurred with a frequency of 67% (see Figure 87), indicating a marked trend toward multiple non-compliances; therefore, the required total compliance (8/8) is not robustly achieved, this confirms that, under current conditions, consistently achieving the acceptance criterion (8/8) is unlikely

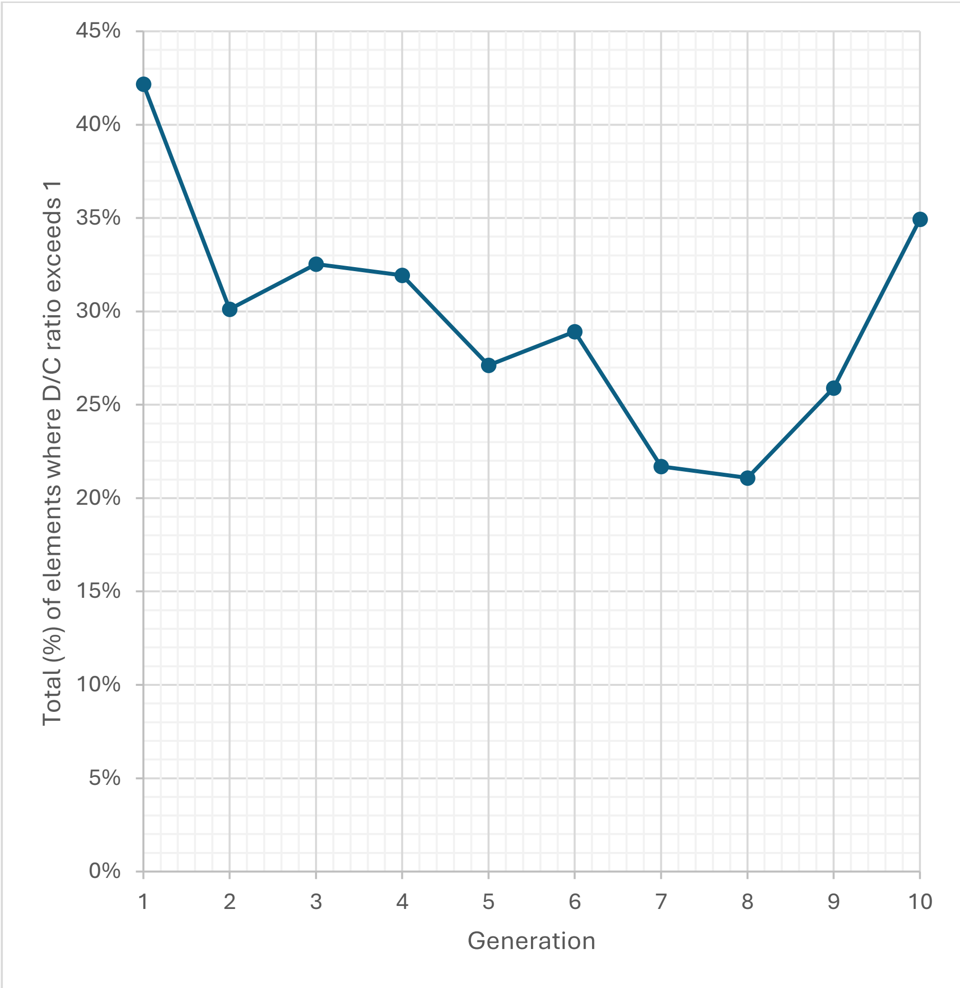


Figure 88. Behavior of Design ratio (D/C)

Figure 88 shows the percentage of the total amount of elements that their desgin ratio(D/C) is higher than 1, it is obtained with the maximun value obtained across the generations.

Since these constraints are the primary drivers of the penalty value, the results are as follows (**See Figure 89**):

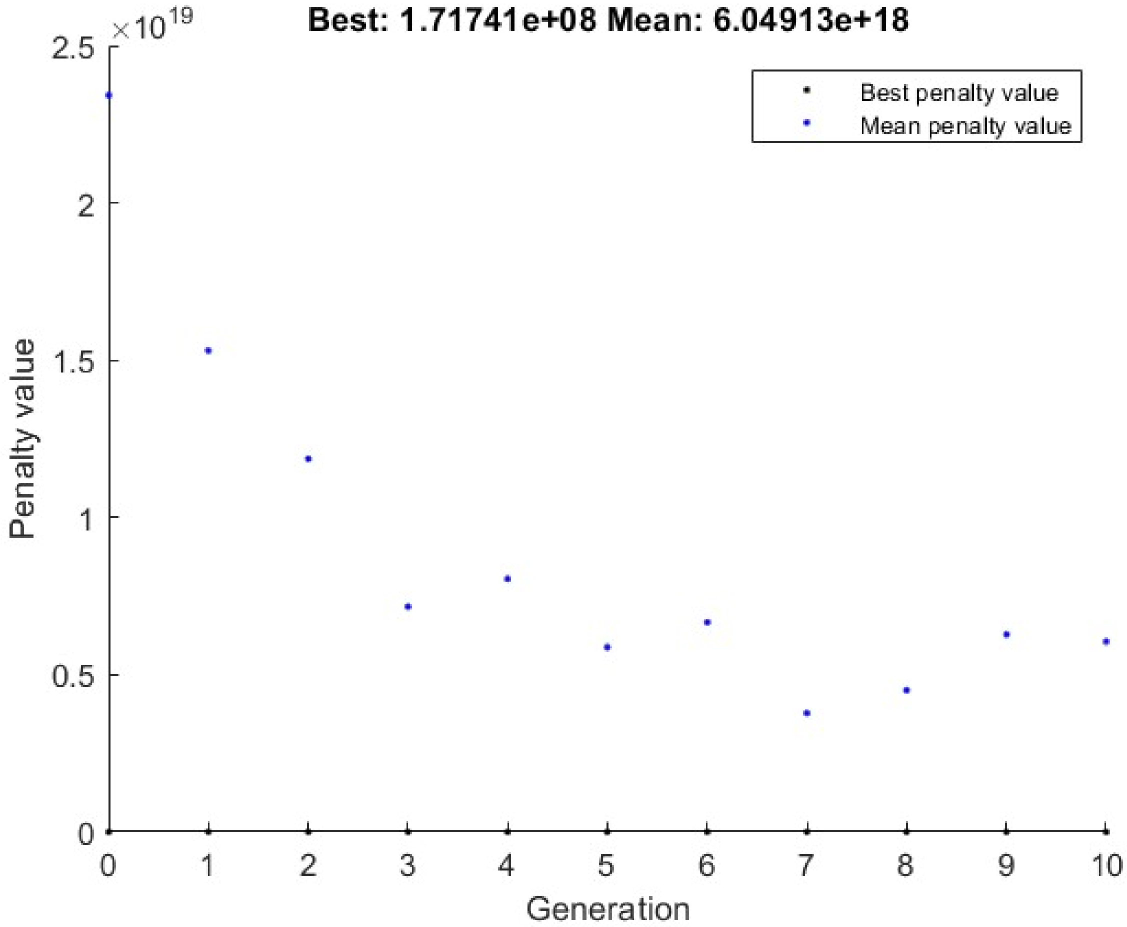


Figure 89. Behavior of Penalty value

Where the best value found for the penalty was 1.17e08, however, it suggests that even the best solution includes some penalty or is a very heavyweight structure due to the strict restrictions.

It is important to note that, although the expected results were not achieved, the code functions correctly: a progressive improvement is observed throughout the iterations starting from generation 1, until the best obtained value is identified.

Below, the maximum values achieved for the various constraints in generation 1 are compared with those obtained in the final iteration.

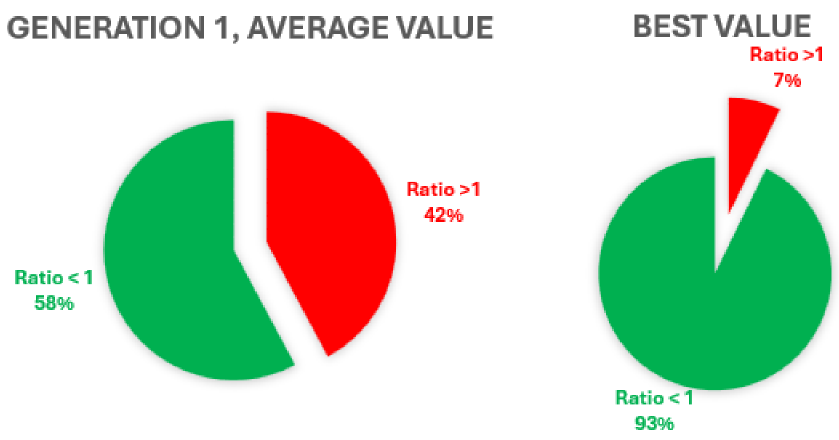


Figure 90. Design ratio values: Generation 1 vs best result

It is observed in Figure 90 that in Generation 1, where the initial alternatives are evaluated, 76 elements (about 42%) show a ratio > 1. In contrast, in the best result achieved in the seventh generation, only 12 elements (approximately 7% of the analyzed total) exceed this threshold. This reflects a significant improvement, in the order of 51%.

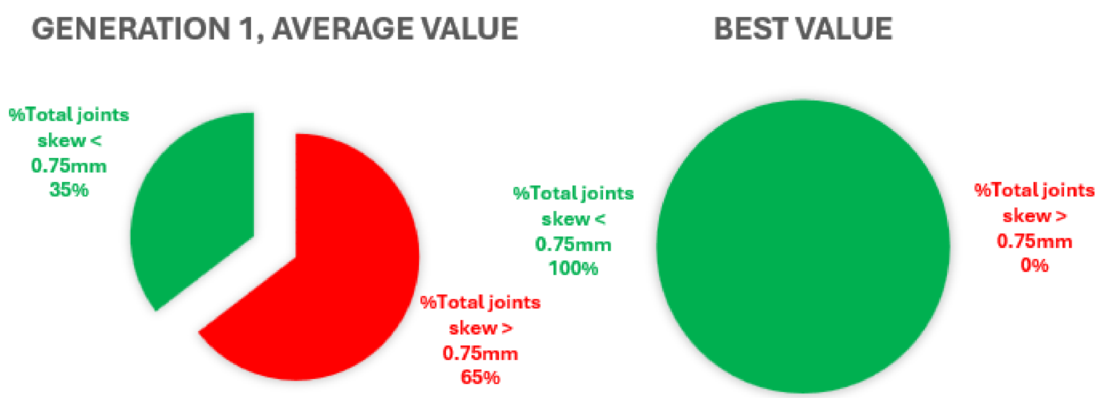


Figure 91 Skew violation values: Generation 1 (maximum) vs best result

It is observed in Figure 91 that in Generation 1, the average value of violations is about 65% show a value of skew > 0.75mm. In contrast, in the best result achieved in the seventh generation there is not any value above the limit. This reflects a total improvement.

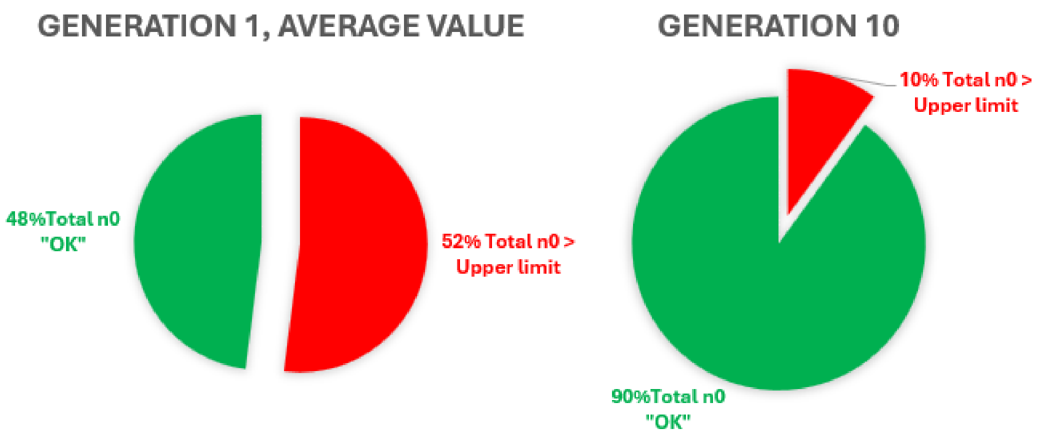


Figure 92. Dynamic effects value (n_0): Generation 1 vs. best value

In Figure 92 a general improvement is observed in the total percentage of values complying with the established limits, rising from approximately 48% in Generation 1 to nearly 90% in Generation 10.

As previously mentioned, the verifications for **horizontal displacement**, **vertical displacement**, and **angular variation** are met in all iterations, from the first to the last, with no non-compliances, meaning that there would be no variation: it would be represented as a constant line along the X-axis with Y = 0 for each generation.

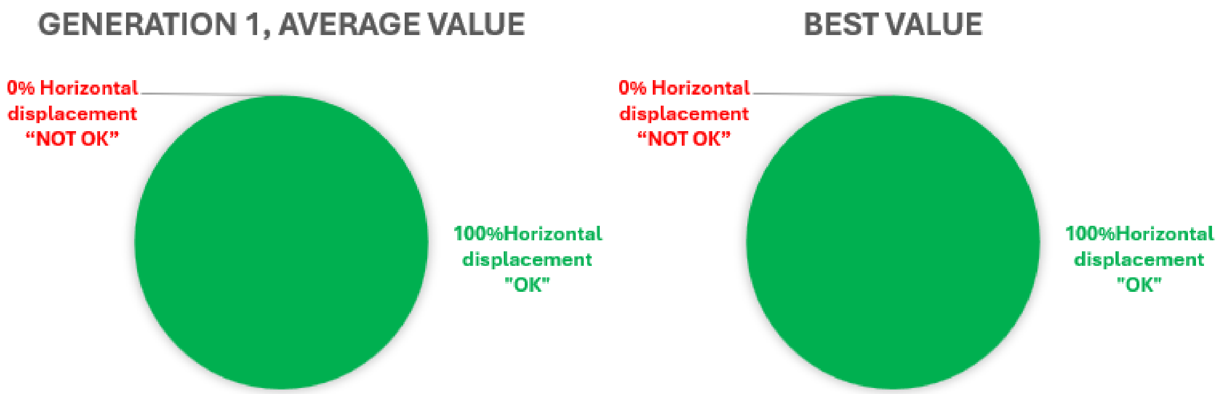


Figure 93. Horizontal displacement: Generation 1 vs. best value

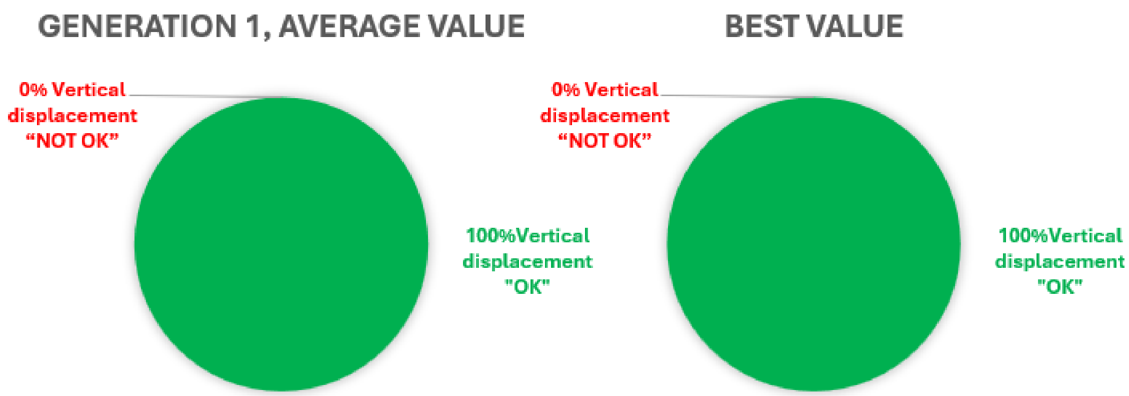


Figure 94. Vertical displacement: Generation 1 vs. best value

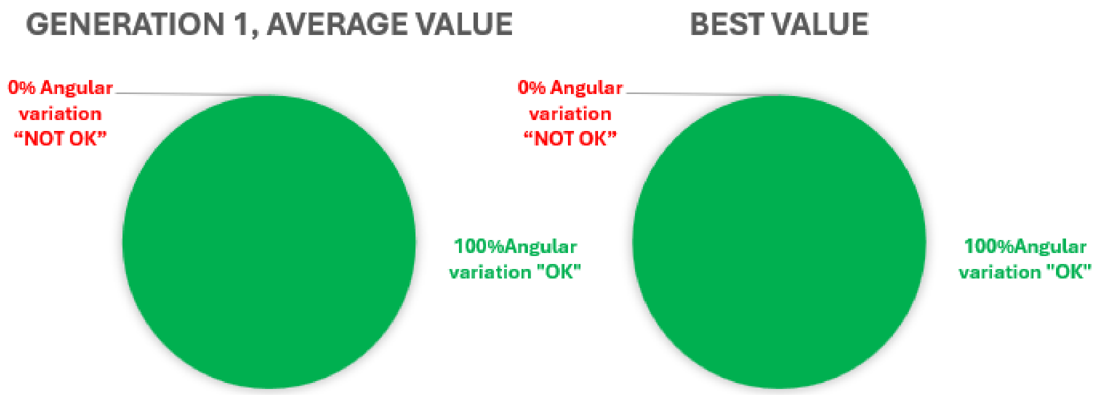


Figure 95. Angular displacement: Generation 1 vs. best value

For the **fatigue verification**, as previously mentioned, within the population (500–540 iterations), the probability of obtaining a result of 1 out of 8 failures is extremely low, and achieving 0 out of 8 is virtually null under the current limits. The best result achieved during the optimization process recorded 3 out of 8 fatigue failures; that is, although it was the best case **(see Figure 96)**, it still exhibits non-compliance.

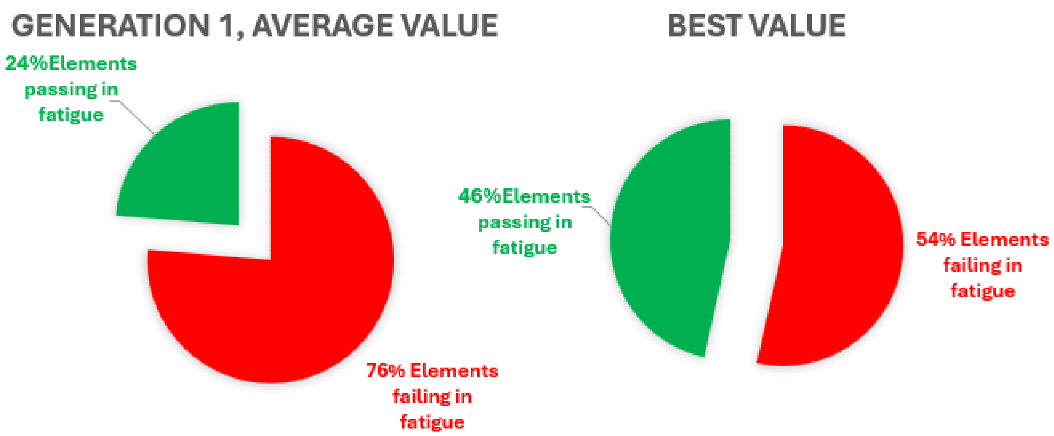


Figure 96. Fatigue: Generation 1 vs. best value

12

|CONCLUSIONS

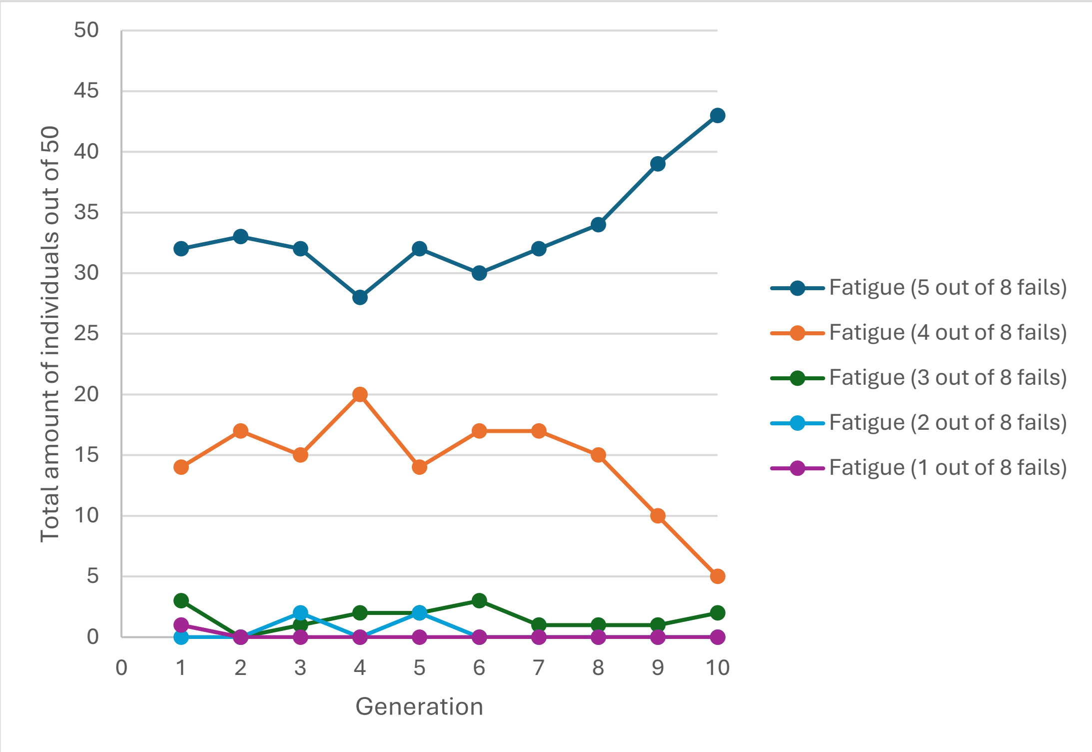
This chapter presents the key results obtained from implementing and applying the optimization framework developed for the sizing of railway trusses. The system—based on interoperability between MATLAB and SAP2000—has proven technically sound for addressing design problems with mixed variables using a Genetic Algorithm (GA).

Throughout the document, the adopted constraint scheme has been validated; it comprehensively integrates the critical ULS/SLS verifications (including fatigue, dynamics, and serviceability limits) required by the relevant design standards (Eurocode and NTC 2018). The implemented penalty methodology, combining proportional and fixed penalties, has been vital in steering the GA's search toward feasible, weight-optimized designs.

The following section presents the analysis of the results derived from the preceding chapter. The aim is to determine whether the framework achieved the thesis's primary objective: identifying lighter, code-compliant designs while substantially reducing manual iteration. The implications of the results will be discussed, along with the limitations of the defined scope (member sizing for a 43 m span) and recommendations for future work (such as the inclusion of connections and constructability).

- The scope is clear: the study fixes the global geometry (a 43 m reference span) and focuses on weight minimization through member sizing alone appropriate for demonstrating the optimization engine while explicitly leaving connections and constructability for future work.

- The computational plan requires more iterations, with 50 individuals and 10 generations (500-540 iterations), the configuration does not provide sufficient convergence depth for the total amount of variable analyzed. In Figure 85,



- Figure 86 and Figure 88, the series are not monotonic; it drops to a certain point and then rises again, even if it does not reach the higher point so there is no sustained trend towards 0 to extrapolate, and know how many iterations will be required.

- The amount of time MATLAB takes to analyze, verify, and design can be due to several variables, from the most common ones like the machine's power and capacity to process information (Hardware), to the more specific ones, such as the interoperability between the software (MATLAB / SAP2000). Despite having demonstrated a high percentage of interoperability (they are part of the same company), there are still gaps in performing certain activities automatically. For example, exporting stresses (S11, S12, and S13) from a frame element, which for this thesis could perhaps reduce the time for the fatigue verification of the elements, or the not possibility to automatically select the design combinations to use (in this particular case, according to their dynamic coefficient), making it necessary to carry this out to verify the design of the remaining groups (ST, IB, and EB). For the other element groups (ST, IB, and EB), and assuming the design time is the same as for the BT group, the total time would, mathematically speaking, increase by nearly double the current time for BT, that is, approximately 15 days more. This means that for a complete model of this magnitude, with nearly 454 frames modeled and 43 meters in length, a total of 24 uninterrupted days would be required; the time will increase proportionally to the number of elements to be analyzed.

- The expected outcome provides the penalties are well scaled and the limits are sensible, the framework should yield lighter, code-compliant designs with substantially less manual iteration, consistent with the thesis objective and abstract, however, certain aspects need to be refined because it is not performing efficiently with all checks:
    - It handled dynamic F60, deflections, rotations, and horizontals well.
    - It is struggling with Skew verification, the results are mixed solutions, both good and bad.
    - It has a clear bottleneck in S11 fatigue verification regarding the web and lower flange details, which never reach (COMPLIANT) status within these design constraints.

  It is necessary to refine and study the allowed search space, as it currently does not contain a fatigue-compliant solution for the elements within groups “IB”, “EB” (Transversal bottom beams), and “ST” (Stringer). Any solution proposed for future work must specify an adequate section margin in critical fatigue zones

- Regarding the design of the elements, the ANNEX-12_MATLAB Results file shows “bad” iterations with extremely high ratios (on the order of 104–105). However, in the more reasonable iterations with D/C ratios in the 1.1–1.8 range most elements fall within the (0.4–0.7) and (0.7–0.9) intervals, while very few appear in (0.1–0.4) and virtually none below 0.1. This suggests that, from a static perspective, the design performs adequately; however, fatigue criteria are driving the need to increase the cross-sectional area in very localized zones.

- When defining variable ranges for welded built-up sections, each element in this group generates on the order of 7,000 to 8,000 possible combinations. Therefore, it is necessary to refine the ranges used. As a suggestion, a catalog of welded sections could be pre-generated using MATLAB; this way, like the process for rolled or extruded sections, MATLAB can select the welded sections to be used in each iteration directly from the catalog.

- Moving forward, the goal is to refine and adapt this methodology to the specific geometric requirements (length, width, and height) of each individual project. However, each case must be studied in detail to properly define the variable ranges and section types to be considered

- In the future, fatigue results could be analyzed on an element-by-element basis, allowing for a more specific and detailed penalty. This contrasts with the general approach used in this project, where evaluation was performed by groups of elements without distinguishing between how many complied with the verification and how many did not.

- The workflow demonstrates that MATLAB–SAP2000 interaction is not only possible but operational: it reads a structured Excel file, assigns sections, runs analyses, and extracts data for verifications making the process reproducible.
- The problem formulation is suitable for mixed variables. Using a Genetic Algorithm (GA) is warranted given the presence of discrete variables (catalog sections) and continuous variables (parametric dimensions).
- ULS/SLS verifications include dynamic behavior, fatigue, serviceability deflections (vertical L/600), track skew, horizontal curvature (R), angular rotation, and member D/C ratios, covering the key failure and comfort modes for a railway truss.
- The penalty scheme guides the GA toward feasible, lighter designs. Proportional penalties combined with fixed penalties provide direction for improving borderline cases and discarding infeasible options.

13

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|APPENDICES

1.) ANNEX-01_ Load Analysis
2.) ANNEX-02_Temperature Analysis
3.) ANNEX-03_Wind Analysis
4.) ANNEX-04_Load Combinations
5.) ANNEX-05_Track Skew Verification
6.) ANNEX-06_Track Horizontal Verification
7.) ANNEX-07_Track Vertical Verification
8.) ANNEX-08_Fatigue
9.) ANNEX-09_Design result BT-BB-DG-VT
10.) ANNEX-10_SAP Variables
11.) ANNEX-11_GA Results
12.) ANNEX-12_MATLAB_Results
13.) ANNEX-13_MATLAB CODE
14.) ANNEX-14_SAP MODEL