BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iaşi
Volumul 66 (70), Numărul 2, 2020
Secţia
CONSTRUCŢII. ARHITECTURĂ

TECHNICAL ASPECTS AND STRUCTURAL VERIFICATION OF A FOOTBRIDGE WITH COMPOSITE STEEL-CONCRETE BOX GIRDER

BY

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Received: April 21, 2020
Accepted for publication: May 5, 2020

Abstract. Footbridges, due to small loads, have very light structures in the case of small and medium size spans, when the footbridge is made of steel or composite steel concrete.

Due to a low rigidity in the horizontal and vertical plane, footbridge superstructures dynamic behaviour must be verified, so that the resonance should be avoided, and the traffic comfort ensured.

In order to have the dynamic parameters, frequencies and accelerations, within the acceptable range, a slight over dimensioning of the superstructure can be made.

In the case of composite structures, the designer must consider the stresses derived from thermal expansion and contraction of concrete that could have a high impact in the overall behaviour, even though according to EC4, for cross-sections in the classes 1 and 2, these types of stresses can be neglected.

This paper presents some aspects related to the structural design and dynamic behaviour of a composite superstructure footbridge of 31.50 m span.

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Keywords: composite structures; box girder; Eurocode; shear lag; dynamic behaviour; traffic comfort; thermal stresses; concrete shrinkage.

1. Introduction

In the area of a park near the Someș River in the city of Cluj-Napoca, Romania, two footbridge superstructures were required. The superstructures must correspond architecturally to the framing of the site area. For both footbridges the adopted solution was similar, simple, with a reduced construction height.

The superstructure uses steel S235 beams, with a span of 31.50 m; the cross-section of the beams is a box girder with a constant height of 1200mm, with a concrete slab C25/30 at the top flange with the maximum thickness of 120mm. The concrete slab with the surface concrete, ensures a 2% transverse slope for the evacuation of rain waters in marginal troughs with a longitudinal slope of 1%.

In order to ensure a reduced construction height, the box girder is an orthotropic steel deck that also serves as formwork for concrete pouring of the deck. The webs have a cross-section of 12x1150 mm, the bottom flange has a thickness of 40 mm in the central area and 30mm along the marginal areas. The bottom flange has holes of 300x600 mm for the execution of the interior welds and in order to ensure maintenance works during the lifetime of the superstructure.

In the transverse direction the box girder has transverse semi frames and diaphragms with holes near the lateral cantilevers.

The deck is made of three sections, a central one and two marginals, with lengths required by the road traffic conditions; the two mounting connections are welded, therefor two holes are required that allow access to the interior of the box girder, holes that will be covered after the connection of the three sections is finalized.

It is worth mentioning that the dimensions of the box girder resulted mainly from the dynamic and traffic comfort requirements of the footbridge.

In Fig. 1 the cross-section of the deck in the central area is presented. Central area is defined by the thickness of the inferior flange equal to 40 mm.

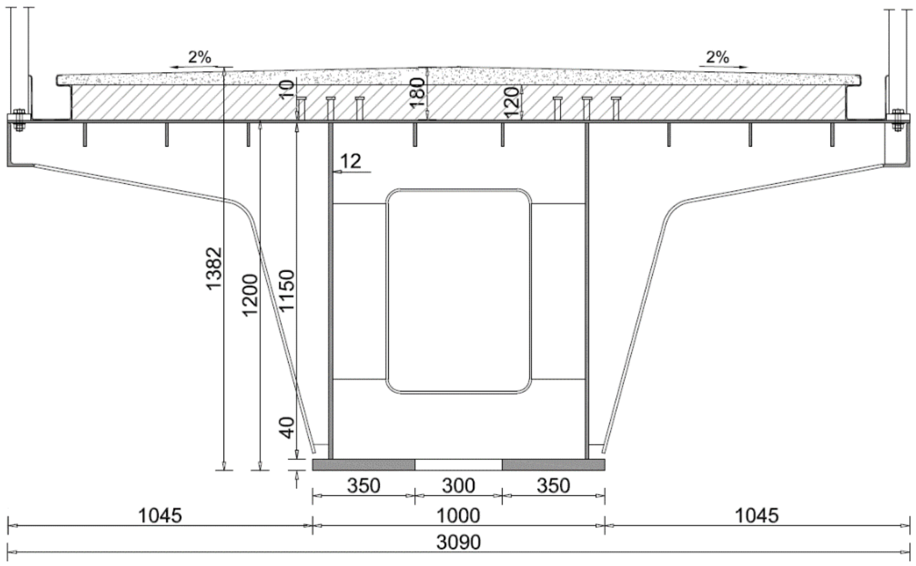

Fig. 1 – Cross-section of the deck. Central area.

2. Calculation of the Steel-Concrete Composite Deck

The superstructure of the footbridge was calculated according to the execution phases:

– the assembly on the shore by welded joints of the metallic structure made of three sections;

– mounting of the steel structure on abutments, using temporary support devices;

– mounting the reinforcement and pouring of the concrete deck;

– lifting of the superstructure using hydraulic presses and mounting the definitive support devices;

– mounting the expansion joints, the wear layer and pedestrian parapet.

1) Ultimate Limit State Design (ULS)

– bending verification of the steel structure during the mounting of the superstructure, considering the weight of the steel structure and the weight of the freshly poured concrete,

– verification of the steel-concrete composite structure considering pedestrian loads or service vehicle and wind action:

- bending resistance of the cross-section;
- shear resistance of the cross-section;
- shear buckling resistance;
- torsion verification of the cross-section;

- stability verification of the orthotropic deck;
- stiffening rigidity and welded joints check.

2) Serviceability limit state design (SLS)
– elastic deformation verification;
– dynamic response correlated to the traffic comfort verification.

3) The effects the thermal expansion and contraction of concrete have upon the stresses in the deck.

4) The effect of thermal variation over the composite steel-concrete cross-section.

Some aspects related to the calculation of a steel-concrete composite deck are presented in summary.

Effective section due to shear lag ang the class of the cross-section

In the case of wide flanges, the bending stresses do not have a uniform distribution across the width of the flanges. Their maximum values are reached near the web and the stresses decrease towards the extremities of the flange; the phenomenon is known as shear lag caused by the deformations due to unit shear stresses.

In order to simplify the resistance and stability calculations, the real width of the plate with a non-uniform stress distribution is replaced by a reduced width that is considered to have a uniform stress distribution, known as bending active width of the plate, Fig. 2, where Eq. (1) is true:

$$\int_{o}^{b} \sigma_x(y)dy = b_{eff} \cdot \sigma_{max} \tag{1}$$

According to (SR EN 1993 – 1 – 5 §3.1), the shear lag phenomenon can be neglected if the condition $b_o \leq L_e/50$ is fulfilled, where L_e is the length between the null bending moment points.

In this case the condition $b_o \leq \frac{L_e}{50} = \frac{31500}{50} = 630$ mm is fulfilled for the panel between the webs and it is not for the cantilever areas.
For the panels that are on the sides, cantilever panels, the active width will be evaluated using the methodology given by SR EN 1993-1-3:

$$b_{eff} = \beta \cdot b_0;\ \beta \text{ – } \textit{factor that gives the plate contribution} \tag{2}$$

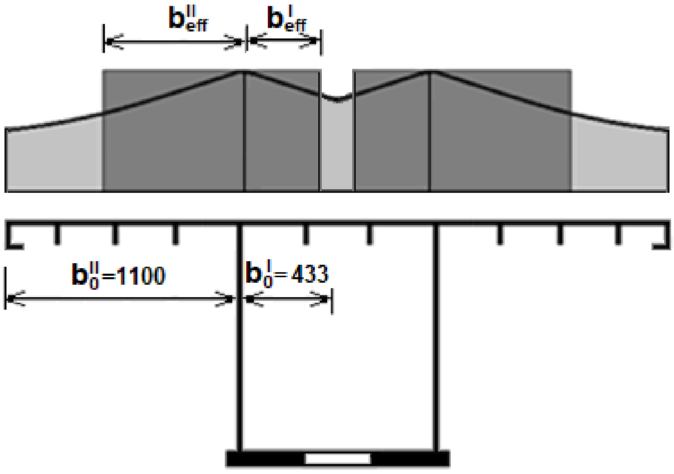

Fig. 2 – Shear lag phenomenon and active width of the plate.

Area of the longitudinal ribs: $\sum A_{sl} = 3 \cdot 1 \cdot 0.8 + 24 \cdot 2 = 48.2$ cm^2

The stiffness coefficient: $\alpha_0 = \sqrt{1 + \frac{\sum A_{sl}}{b_0 \cdot t}} = \sqrt{1 + \frac{48.2}{110 \cdot 1}} = 1.2$

$$k = \frac{\alpha_0 \cdot b_0}{L_e} = \frac{1.2 \cdot 110}{3150} = 0.042 \in [0.02 - 0.7]$$

$$\beta = \beta_1 = \frac{1}{1 + 6.4 \cdot k^2} = \frac{1}{1 + 6.4 \cdot 0.042^2} = 0.99 \approx 1$$

Considering that $\beta \approx 1$ the whole cross-section is active for both ULS and SLS.

The same result can be obtained by using the simplified relation:

$$b_{eff} = \min\left(b_0; \frac{L_e}{8}\right) = \min\left(1100; \frac{31500}{8}\right) = 1100$$

The concrete plate is active, the condition: $b_{0.c} \leq \frac{L_e}{8} = \frac{31500}{8} = 3973$ mm is fulfilled.

The deck cross-section is of Class 3, given by the dimensions of the web, therefor the strength of the beam will be evaluated considering an elastic behaviour of the cross-section under bending and shear.

Strength characteristics of the deck

For the pre-dimensioning an equivalence coefficient between concrete and steel $n = 2 \cdot n_0$ will be considered, given by (SR EN 1994-1-1:2004 §

5.4.2.2). For an exact evaluation of the equivalence coefficient the recommendations given in (SR EN 1994-1-1:2004 § 5.4.2.2) and (SR EN 1994-2:2006) will be applied.

The concrete used on the deck is C25/30 with the characteristic modulus of elasticity $E_{cm} = 31\,\mathrm{GPa}$. The equivalence coefficient will be: $n_0 = \frac{E_a}{E_{cm}} = \frac{210}{31} = 6.77$ and $n = 2 \cdot n_0 = 13.54$.

The concrete deck, with a minimum thickness of 110 mm, will be equivalent with a steel plate in the median plane of the plate deck, having the thickness: $t_c^{ech} = 110/13.54 \approx 8\,\mathrm{mm}$.

Such a model of the concrete plate is closer to reality with respect to the dynamic behaviour of the deck, considering that the steel and concrete are working together, with respect to the model where the width of the concrete plate is reduced by the equivalence coefficient.

The strength characteristics of the steel beam and the composite beam with the concrete plate modelled as expressed above are presented in Fig. 3.

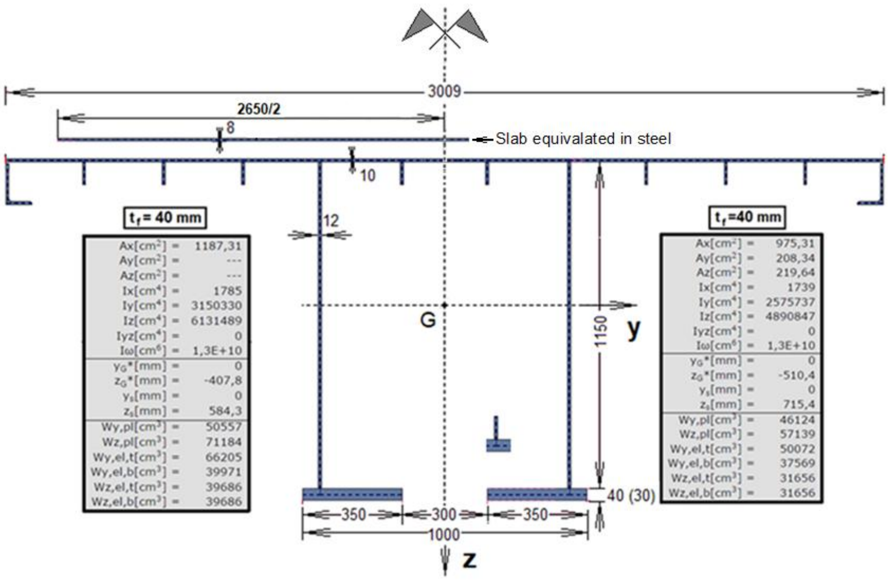

Fig. 3 – Strength characteristics of the steel girder and the composite girder equivalated as a steel girder.

Bi-axial bending with axial force

Following the evaluation of the actions, taking into account the action coefficients ($\gamma_G = \gamma_Q = 1.35$, $\gamma_{Q.w} = 1.5$ and $\gamma_{Q.\Delta\mathrm{T}} = 1.5$), the following maximum stresses at the middle of the beam were found:

– during mounting, after the pouring of the concrete deck (to be considered the weight of the steel box girder and the fresh concrete in the deck) $M_{Ed.g} = 3100$ kNm.

– after the hardening of the concrete the following are to be added: the wear layer, fencer, throughs and the weight of the pedestrians $M_{Ed.p} = 4100$ kNm, $N_{Ed.p} = 65$ kN.

– wind action in the horizontal plane: $M_{z.Ed.w} = 1300$ kNm.

Fig. 4 shows the normal stresses diagrams due to bending.

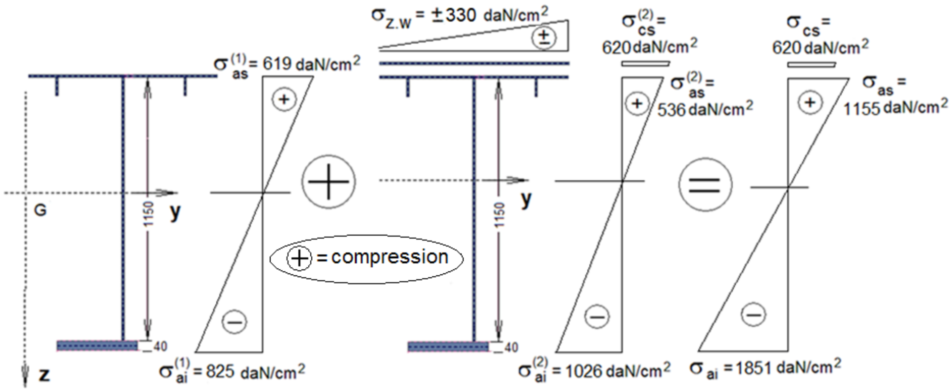

Fig. 4 – Normal stresses diagrams due to bending.

The effect of thermal expansion and concrete shrinkage

According to SR EN 1992, the contraction of concrete can be evaluated as:

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \tag{3}$$

where: ε_{cs} – final strain due to contraction, ε_{cd} – strain due to contraction in time, ε_{ca} – strain due to initial elastic contraction.

Concrete elasticity modulus:

$$E_c = \frac{n_0}{n_S} E_{cm} \tag{4}$$

The equivalence coefficient n_s that considers the contraction effect can be evaluated using Eq. (5):

$$n_S = n_{L(\Psi=0.55)} = n_0 \cdot (1 + 0.55 \cdot \varphi(t, t_0)) \tag{5}$$

$$\varphi(t, t_0) = \varphi_0 \cdot \beta_{c(t,t_0)};\ n_0 = E_a/E_{cm} \tag{6}$$

The efforts that act upon the composite cross-section are:

$$N_m = -N_c = \varepsilon_c \cdot E_c \cdot A_c \text{ – compression force} \tag{7}$$

$$M_m = N_m \cdot z_{cm} \text{ – positive bending moment} \tag{8}$$

The calculation diagram for stresses that come from concrete contraction is presented in Fig. 5 (Moga, 2020).

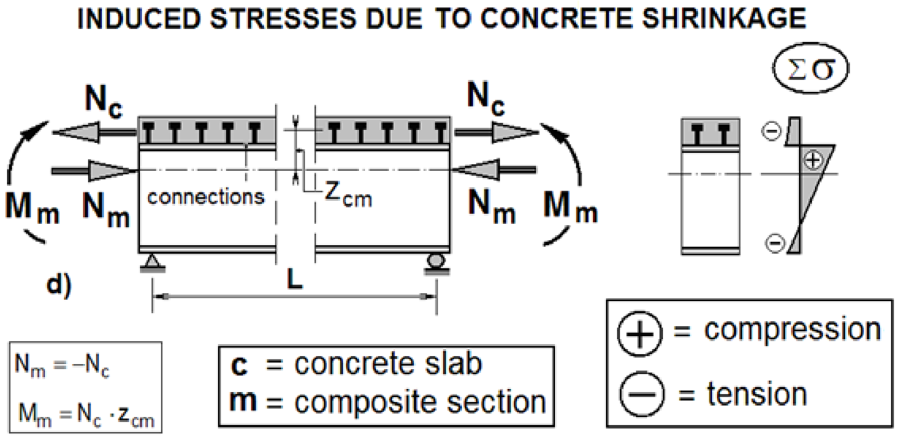

Fig. 5 – Development of stresses due to concrete shrinkage.

Considering that the box girder has a variable cross-section, due to the use of two different thickness at the inferior flange (30 mm and 40 mm) and that it has holes for maintenance, the calculations will be performed using the strength characteristics of the central section with hole with the following calculation parameters:

$$\varepsilon_c = \varepsilon_{cs} = 0.25 \cdot 10^{-3};\ E_c = E_c(t = \infty) \approx 0.5 \cdot E_{cm} = 0.5 \cdot 31 = 15.5 \text{ MPa};$$
$$n_0 = 6.77;\ n = 2 \cdot n_0 = 13.54$$

The forces obtained are:

Axial stress:

$$N_m = -N_c = \varepsilon_c \cdot E_c \cdot A_c = 0.25 \cdot 10^{-3} \cdot 15.5 \cdot 10^4 \cdot 3180 \cdot 10^{-2} = 1232 \text{ kN}$$

Bending moment: $M_m = N_m \cdot z_{cm} = 1232 \cdot 0.47 = 579$ kNm

The stresses due to thermal expansion and the concrete shrinkage, at the centre of the plate and the extreme fibres of the steel box girder have the following values:

$$\sigma_c = -\frac{N_c}{A_c} + \frac{1}{n} \cdot \left(\frac{N_m}{A_m} + \frac{M_m}{I_m} \cdot z_{cm}\right)$$
$$= -\frac{1232 \cdot 10^2}{3180} + \frac{1}{13.54} \cdot \left(\frac{1232 \cdot 10^2}{1187} + \frac{579 \cdot 10^4}{3.15 \cdot 10^6} \cdot 47\right)$$
$$= -31 \text{ daN/cm}^2$$

$$\sigma_{a.sup} = \frac{N_m}{A_m} + \frac{M_m}{I_m} \cdot z_{as} = \frac{1232 \cdot 10^2}{1187} + \frac{579 \cdot 10^4}{3.15 \cdot 10^6} \cdot 41.2 = 180 \text{ daN/cm}^2$$

$$\sigma_{a.inf} = \frac{N_m}{A_m} - \frac{M_m}{I_m} \cdot z_{ai} = \frac{1232 \cdot 10^2}{1187} + \frac{579 \cdot 10^4}{3.15 \cdot 10^6} \cdot 78.8 = -41 \text{ daN/cm}^2$$

Stresses due to temperature variation across the composite cross-section depth

For a given period, the heating and cooling of the superior part of the deck leads to a temperature variation that can lead to a maximum heating (the superior face is hotter) and a maximum cooling (the inferior face is cooler).

In case of composite decks, for protection layers of 50 mm, the following values are to be considered (SR EN 1991-1-5):

$$\Delta T_{M.heating} = 15^{\mathrm{o}}\mathrm{C};\ \Delta T_{M.cooling} = 18^{\mathrm{o}}\mathrm{C}$$

The loading due to temperature is considered as a short-term load, the calculation cross-section is to be determined using the equivalence coefficient for short term actions (the equivalence coefficient n_0).

The strain due to temperature variation is:

$$\varepsilon_{c.\Delta T} = \alpha_T \cdot \Delta T_M \tag{9}$$

where: $\alpha_T = 1 \cdot 10^{-5}/^0\mathrm{C}$ – coefficient of thermal expansion of concrete and steel in composite steel-concrete structures (SR EN 1991-1-5, Annex C, Table C.1).

The axial force in the concrete deck is:

$$N_{c.\Delta T} = -N_{m.\Delta T} = -\varepsilon_{c.\Delta T} \cdot E_{cm} \cdot A_c \tag{10}$$

Fig. 6 shows the stresses across the composite section when the concrete is cooler than the opposite side of the cross-section (with a temperature difference $\Delta T_{M.cooling} = 18^{\mathrm{o}}\mathrm{C}$). If the temperature in the deck is higher than the opposite side (with a temperature difference $\Delta T_{M.heating} = 15^{\mathrm{o}}\mathrm{C}$), the stresses will have opposite signs compared to the previous case.

The stresses in the concrete deck and the steel beam:

$$\sigma_c = -\frac{N_{c.\Delta T}}{A_c} + \frac{1}{n_0} \cdot \left(\frac{N_{m.\Delta T}}{A_m} + \frac{M_{m.\Delta T}}{I_m} \cdot z_c \right) \tag{11}$$

$$\sigma_a = \frac{N_{m..\Delta T}}{A_m} + \frac{M_{m..\Delta T}}{I_m} \cdot z_a \tag{12}$$

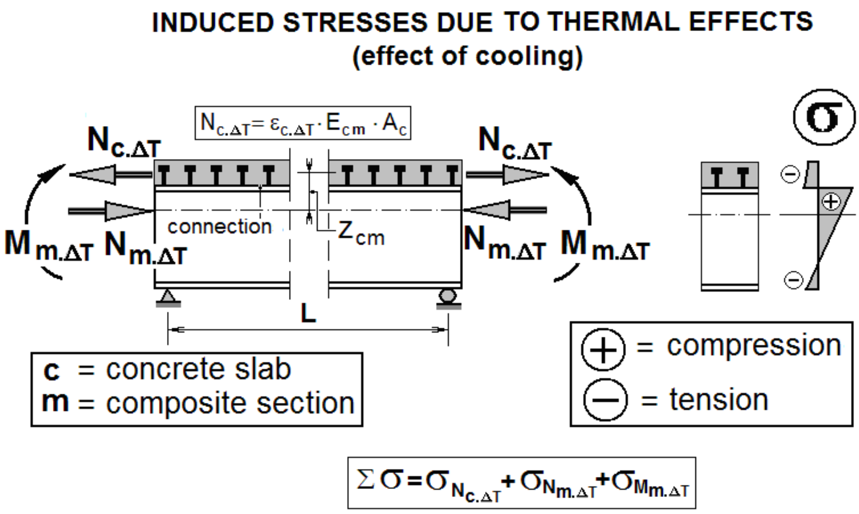

Fig. 6 – Stress distribution due to thermal effect of cooling and heating.

Stress calculation when the deck is cooler $\Delta T_{M.cooling} = 18^{\circ}\mathrm{C}$

Specific deformation due to temperature variation: $\varepsilon_{c.\Delta T} = \alpha_T \cdot \Delta T_M = 1 \cdot 10^{-5} \cdot 18 = 1.8 \cdot 10^{-4}$

The equivalence coefficient for short-term loads: $n_0 = 6.77$

The axial stress in the concrete deck: $N_{c.\Delta T} = -N_{m.\Delta T} = -\varepsilon_{c.\Delta T} \cdot E_{cm} \cdot A_c = 1.8 \cdot 10^{-4} \cdot 31 \cdot 10^4 \cdot 3180 = 177 \cdot 10^3\ \mathrm{daN} = 1770\ \mathrm{kN}$

The bending moment: $M_{m.\Delta T} = N_{m.\Delta T} \cdot z_{cm} = 177 \cdot 10^3 \cdot 40 = 70.8 \cdot 10^5\ \mathrm{daNcm} = 708\ \mathrm{kNm}$

The equivalent thickness in steel of the concrete deck $t_c^{ech} = \frac{110}{6.77} \approx 16\ \mathrm{mm}$, thickness for which the strength characteristics of the cross-section were evaluated.

The following stresses due to temperature are found:

- in concrete:

$$\sigma_c = -\frac{N_{c.\Delta T}}{A_c} + \frac{1}{n_0} \cdot \left(\frac{N_{m.\Delta T}}{A_m} + \frac{M_{m.\Delta T}}{I_m} \cdot z_c \right)$$
$$= -\frac{177 \cdot 10^3}{3180} + \frac{1}{6.77} \cdot \left(\frac{177 \cdot 10^3}{1400} + \frac{70.8 \cdot 10^5}{3.56 \cdot 10^6} \cdot 40 \right)$$
$$= -25\ \mathrm{daN/cm^2}$$

- in steel:

$$\sigma_{a.sup} = \frac{N_{m..\Delta T}}{A_m} + \frac{M_{m..\Delta T}}{I_m} \cdot z_{as} = \frac{177 \cdot 10^3}{1400} + \frac{70.8 \cdot 10^5}{3.56 \cdot 10^6} \cdot 34 = 194 \text{ daN/cm}^2$$

$$\sigma_{a.inf} = \frac{N_{m..\Delta T}}{A_m} - \frac{M_{m..\Delta T}}{I_m} \cdot z_{ai} = \frac{177 \cdot 10^3}{1400} - \frac{70.8 \cdot 10^5}{3.56 \cdot 10^6} \cdot 86 = -45 \text{ daN/cm}^2$$

Stress calculation when the deck is hotter $\Delta T_{M.heating} = 15^{\mathrm{o}}\mathrm{C}$

Specific deformation due to temperature variation: $\varepsilon_{c.\Delta T} = \alpha_T \cdot \Delta T_M = 1 \cdot 10^{-5} \cdot 15 = 1.5 \cdot 10^{-4}$

The equivalence coefficient for short-term loads: $n_0 = 5.96$

The axial stress in the concrete deck: $N_{c.\Delta T} = -N_{m.\Delta T} = -\varepsilon_{c.\Delta T} \cdot E_{cm} \cdot A_c = 1.5 \cdot 10^{-4} \cdot 31 \cdot 10^4 \cdot 3180 = 148 \cdot 10^3 \text{ daN} = 1480 \text{ kN}$

The bending moment: $M_{m.\Delta T} = N_{m.\Delta T} \cdot z_{cm} = 148 \cdot 10^3 \cdot 40 =\cdot 59.2 \cdot 10^5 \text{ daNcm} = 592 \text{ kNm}$

The following unit stresses due to temperature are found:

- in concrete:

$$\sigma_c = \frac{N_{c.\Delta T}}{A_c} - \frac{1}{n_0} \cdot \left(\frac{N_{m.\Delta T}}{A_m} + \frac{M_{m.\Delta T}}{I_m} \cdot z_c\right) = \frac{148 \cdot 10^3}{3180} - \frac{1}{5.96} \cdot \left(\frac{148 \cdot 10^3}{1400} + \frac{59.2 \cdot 10^5}{3.56 \cdot 10^6} \cdot 40\right) = 18 \text{ daN/cm}^2$$

- in steel:

$$\sigma_{a.sup} = -\left(\frac{N_{m..\Delta T}}{A_m} + \frac{M_{m..\Delta T}}{I_m} \cdot z_{as}\right) = -\left(\frac{148 \cdot 10^3}{1400} + \frac{59.2 \cdot 10^5}{3.56 \cdot 10^6} \cdot 34\right) = -162 \text{ daN/cm}^2$$

$$\sigma_{a.inf} = -\frac{N_{m..\Delta T}}{A_m} + \frac{M_{m..\Delta T}}{I_m} \cdot z_{ai} = \frac{148 \cdot 10^3}{1400} - \frac{59.2 \cdot 10^5}{3.56 \cdot 10^6} \cdot 86 = 37 \text{ daN/cm}^2$$

Fig. 7 presents the stresses in the two situations discussed:

– concrete deck with a lower temperature than the opposite side;

– concrete deck with a higher temperature that the opposite side.

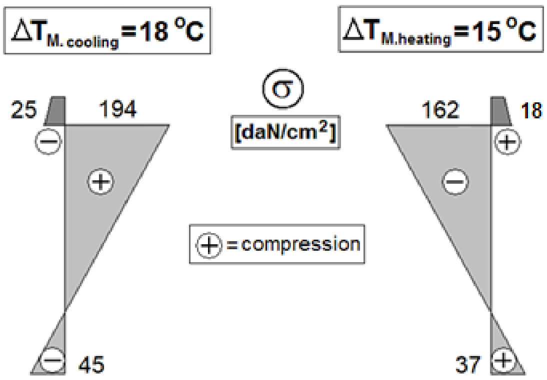

Fig. 7 – Stress distribution due to thermal effect of cooling and heating.

For the verifications at Ultimate Limit States the action coefficient and the group coefficient are: $\gamma_{Q.\Delta T} = 1.5$ and $\psi_0 = 0.6$.

The stresses in the concrete deck and at the extreme fibres of the steel box girder are obtained by summation of the stresses computed above, taking into account the group coefficient ψ.

Table 1 presents in summary the unit stresses obtained in the concrete deck and in the steel box girder.

Table 1

Unit Normal Stresses in the Deck [daN/cm^2]

	Mounting phase (box girder weight and concrete plate weight)	Pedestrian loading	Wind action $\psi_0 = 0.3$ $n = 13.54$	Concrete thermal expansion and contraction	Temperature variation $\psi_0 = 0.3$	Total stresses
Concrete plate	-	$\frac{620}{n} = +46$	$\psi_0 \cdot \frac{283}{n} = +6$	-31	$\psi_0 \cdot 18 = +11$	32
Steel – top flange	+619	+536	$\psi_0 \cdot 330 = +100$	+180	$\psi_0 \cdot 194 = +116$	1551
Steel – bottom flange	-825	-1026	$\psi_0 \cdot 107 = -32$	-41	$\psi_0 \cdot 45 = -27$	1951

Traffic comfort corelated to dynamic behaviour parameters

Pedestrians traffic comfort is corelated to the acceleration of the structure, determined for different dynamic loading cases.

Four conventional domains for vertical and horizontal accelerations are defined in Fig. 8, corresponding to maximum, medium, minimum and unacceptable comfort level (Setra, 2006).

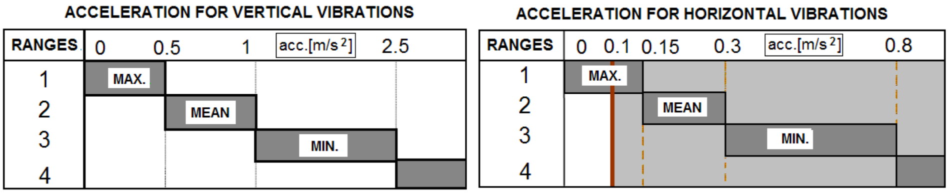

Fig. 8 – Conventional accelerations domains.

For footbridges that fall into traffic classes I, II and III it becomes necessary to evaluate the natural frequency of the structure. The frequencies are evaluated along the three directions: vertical, horizontal transverse and horizontal longitudinal (Moga, 2020; Setra, 2006).

The frequencies are determined for two mass hypotheses of the system:

- unloaded footbridge
- loaded footbridge with the value of the loading 700 N/mm^2.

The vertical and horizontal frequencies could fall within four domains about the resonance risk, Fig. 9 (Setra, 2006), where:

Domain **1**: maximum resonance risk
Domain **2**: medium resonance risk
Domain **3**: low resonance risk
Domain **4**: negligible resonance risk.

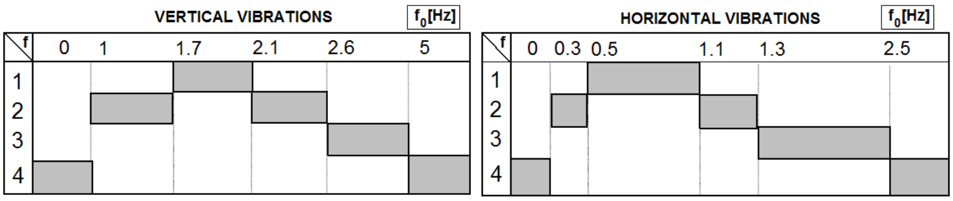

Fig. 9 – Conventional domains for the frequencies.

For the simply supported beam with constant characteristics, the analytic calculation for the natural vibration modes can be done using Table 2.

In case of the designed footbridge (XC Project, 2020), as the structure is narrow compared to the length and with a good torsional stiffness (the cross-section is a box girder), the frequencies derived from torsion and axial force are high, the analysis will be done only for bending vibrations (vertical and horizontal).

Table 2
Dynamic Characteristics Evaluation

Mode	Natural pulsation	Natural frequency	Vibration mode
Simple bending with *n* half-waves	$\omega_n = \frac{n^2\pi^2}{L^2}\sqrt{\frac{E\cdot I}{\rho S}}$	$f_n = \frac{n^2\pi}{2\cdot L^2}\sqrt{\frac{E\cdot I}{\rho S}}$	$v_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Tension – compression with *n* half-waves	$\omega_n = \frac{n\pi}{L}\sqrt{\frac{E\cdot S_N}{\rho S}}$	$f_n = \frac{n}{2\cdot L}\sqrt{\frac{E\cdot S_N}{\rho S}}$	$u_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Torsion with *n* half-waves	$\omega_n = \frac{n\pi}{L}\sqrt{\frac{G\cdot I_\omega}{\rho I_r}}$	$f_n = \frac{n}{2\cdot L}\sqrt{\frac{G\cdot I_\omega}{\rho I_r}}$	$\theta_n(x) = \sin\left(\frac{n\pi x}{L}\right)$
Maximum acceleration	$Acc_{max} = \frac{1}{2\xi_n}\frac{4F}{\pi\rho S}$		
Measure units: L [m]; E=210·10^9 N/mm^2; I [m^4]; ρS [kg/m]; m [kg/m]. **Parameters:** ρS - linear density of the structure; ρI_r - torsion moment of inertia; ES_N - axial rigidity; EI - bending rigidity; GI_ω - warping torsion rigidity.			

Vibration mode 1 in the vertical plane

The inertia moment for the steel box girder (including the longitudinal stiffeners) with regard to the horizontal axis y-y is $I_y = 0.0315\ m^4$;

The linear natural density, resulted from the weight of the box girder, the longitudinal and transverse stiffeners, the concrete deck, cantilevers, fences: m = 2400 kg/m;

The linear density:

- unloaded footbridge: $\rho \cdot S = 2400$ kg/m
- loaded footbridge with density d: $\rho \cdot S = 2400 + 210 = 2610$ kg/m

Frequencies for vibration mode 1:

- superior frequency: $f_1 = \frac{1^2\cdot\pi}{2\cdot 31.5^2}\cdot\sqrt{\frac{210\cdot 10^9\cdot 0.0315}{2400}} = 2.63$ Hz
- inferior frequency: $f_1 = \frac{1^2\cdot\pi}{2\cdot 31.5^2}\cdot\sqrt{\frac{210\cdot 10^9\cdot 0.0315}{2610}} = 2.52$ Hz

It can be observed that Vibration Mode 1 falls within Domain 3: low resonance risk. For this domain the dynamic calculation is not necessary, in other words the calculation of the system acceleration is not required ($\psi = 0$).

Vibration mode 1 in the horizontal plane

The inertia moment about the vertical axis z-z is $I_z = 0.0613$ m^4;

Frequencies for vibration mode 1:

- superior frequency: $f_1 = \frac{1^2 \cdot \pi}{2 \cdot 31.5^2} \cdot \sqrt{\frac{210 \cdot 10^9 \cdot 0.0613}{2400}} = 3.66\ \text{Hz}$
- inferior frequency: $f_1 = \frac{1^2 \cdot \pi}{2 \cdot 31.5^2} \cdot \sqrt{\frac{210 \cdot 10^9 \cdot 0.0613}{2610}} = 3.51\ \text{Hz}$

The frequencies for Vibration Mode 1 fall within Domain 4: negligible resonance risk.

3. Conclusions

Footbridges are part of the bigger family of bridges but due to a low useful load, their superstructures are very light, in case of small and medium size spans, when the superstructure is made of steel or composite steel concrete.

Because of a relatively reduced stiffness in the vertical and horizontal plane, footbridges should be verified from the point of view of their dynamic behaviour, so that the resonance risk to be avoided and the traffic comfort to be ensured.

In order to fit the dynamic parameters - frequencies and accelerations, within the necessary limits, it is necessary, in many cases, to modify the dimensions of the constituent elements of the superstructure, resulting in a slight oversize relative to the conditions of resistance, or the use of damping devices, which may be more expensive comparatively with increasing the rigidity of the deck.

In the presented case, in the design phase the dimensions of the box girder beam have been increased so that the conditions regarding the dynamic response of the structure and the pedestrian comfort were obtained, resulting in a slight oversize.

In composite structures, the stresses resulting from the thermal expansion and contraction of the concrete, as well as the stresses from the temperature variation over the depth of the beam cross-section must be considered. Such stresses can reach about 50% of the stresses that come from pedestrian (live) loads.

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ASPECTE TEHNICE ȘI DE VERIFICARE STRUCTURALĂ A UNEI PASARELE PIETONALE CU SECȚIUNE CASETATĂ COMPUSĂ OȚEL-BETON

(Rezumat)

Pasarelele pietonale, ca urmare a încărcărilor reduse, sunt alcătuite cu structuri de rezistență rezultă ușoare, în cazul deschiderilor mici și mijlocii pe grinzi metalice sau compozite oțel-beton.

Datorită rigidităților relativ reduse în plan vertical și orizontal, structurile de pasarele trebuie verificate din punct de vedere al comportării dinamice, astfel încât să fie evitat fenomenul de rezonanță și să fie asigurat confortul de circulație al pietonilor.

Pentru încadrarea parametrilor dinamici - frecvențe și accelerații, în limitele necesare, se impune, în multe situații, modificarea dimensiunilor elementelor constitutive ale suprastructurii, uneori rezultând o ușoară supradimensionare a structurii de rezistență.

La structurile compuse trebuie avute de asemenea în vedere și eforturile rezultate din contracția și curgerea lentă a betonului, care pot avea valori destul de importante, deși conform normativului EC 4 acestea pot fi neglijate pentru clasele 1 și 2 de secțiuni transversale.

În lucrare se prezintă unele aspecte legate de calculul unui tablier cu structură compusă oțel-beton, din punct de vedere al rezistenței structurale și al comportării dinamice, pentru o pasarelă pietonală cu deschiderea de 31,5 m.