BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iași
Tomul LXI (LXV), Fasc. 3, 2015
Secția
CONSTRUCȚII. ARHITECTURĂ

COMPOSITE STEEL-CONCRETE FOOTBRIDGES ON GIRDERS WITH CIRCULAR HOLES

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Received: September 17, 2015
Accepted for publication: September 30, 2015

Abstract. In this paper, some aspects concerning the design of steel-concrete composite footbridge structures are presented. The steel girders are built-up as rolled sections with circular holes in webs.

In the design of the footbridge structure besides the ULS and the SLS verifications, also the comfort criterion has to be verified which is in direct correlation with the structure frequency (risk of resonance) and acceleration.

Key words: footbridge structures; composite girders; traffic comfort; ULS and SLS verifications.

1. Introduction

Footbridges form part of the bridge category, their function being mainly to serve pedestrians to cross natural or other kinds of obstacles.

In the field of small and average sized openings, the most used constructive solutions for the footbridge superstructure concern steel girders, made of rolled profile, full web welded section girders, hollow girders and lattice girders.

In the case of pedestrian footbridges, besides the verifications for ultimate limit states and service limit states, it is necessary to check for the

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traffic comfort, which is in direct correlation with the structure frequency (risk of resonance) and acceleration.

A moderate comfort allows for the structure limited vibrations that can be achieved with slender and good appearance structures, which can be provided sometimes with vibration dampers.

In the case of simply supported beams having constant characteristics, the analytical calculus for the natural vibration modes is made according to Moga *et al*., (2014), FIB Bulletin 32 and XC PROJECT. EN 1990–EC0–Annex A2, recommends maximum values for accelerations and frequencies to provide traffic comfort and to avoid the resonance phenomenon occurrence.

The imposed footbridge loads, as defined in EC 1-2, come from the pedestrian and cyclist traffic, loads related to small and common constructions, from structure maintenance and accidental contexts.

Three mutually exclusive convoys are taken for the calculus:

a) a uniformly distributed force: q_{fk} =5 kN/m^2;

b) a concentrated force: Q_{fwk} =10 kN;

c) a Q_{serv} load representing the service vehicle, according to EC1-2.

Static calculus convoys are also taken for the horizontal forces.

If necessary, vertical loads and horizontal forces from traffic are taken with the help of loads, as defined in EC 1-2.

2. Composite Steel-Concrete Footbridges on Non-Symmetrical Steel Girders with Circular Holes

The composite steel-concrete footbridges with non-symmetrical steel girders with circular holes can provide both economically and architecturally advantageous solutions for average openings (under 30 m).

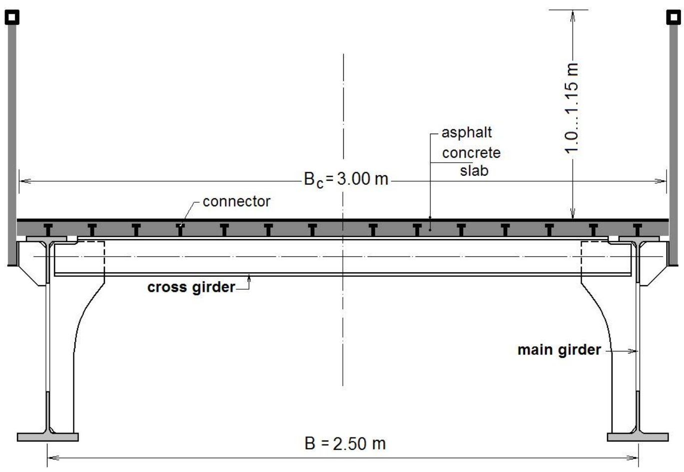


Fig. 1 – Transversal section of the footbridge deck.

The basic calculus of the steel-concrete composite deck will be further presented for a footbridge, with the following design data:

a) structure opening: L = 24.0 m; net width: B_c= 3.00 m;

b) the deck structure is made up of two main girders with composite steel-concrete section, whose cross girders are at l_a=2.40 m and work together with the precast reinforced concrete slab (Fig. 1);

c) the main steel girders, with rolled profiles with circular holes in the web are made by welding the rolled parts for the two ends;

d) the steel for the main girders: S 275 J2.

2.1. Calculus Parameters

Active slab width. It yielded: b_{eff} = 150 cm.

Equivalence coefficient:

For slab concrete Class 30/37, E_{cm} = 33 GPa.

It yields: $n_0 = E_a/E_{cm} = 6.36$.

The unique equivalence coefficient is taken: $n = 2n_0 = 12.72$.

Evaluation of normal forces

Phase 1: (steel structure):

The permanent weight, during Phase 1, resulting from predimensioning, has an estimate value of: g_t =1440 daN/m.

For the main girder: g_1 = 7.2 kN/m.

It yields $M_{g1} \approx 518$ kN.m; $V_{g1} \approx 86$ kN.

Phase 2: (composite steel-concrete structure):

Additional permanent loading in Phase 1: g_{t2} = 370 daN/m.

Additional permanent loading for one girder: $g_2 \approx 2$ kN/m.

It gives: $M_{g2} \approx 144$ kN.m; $V_{g2} \approx 24$ kN.

Net load with people: 3 × 500 daN/m^2 = 1,500 daN/m.

Net load for a girder: p = 7.5 kN/m.

It results: $M_p \approx 540$ kN.m; $V_p \approx 90$ kN.

Load from a vehicle accidentally found on the footbridge; V_V = 57.5 kN.

Wind action. The result is w = 2.3 kN/m^2.

The direct wind action is taken by the concrete slab from the upper flange of the main girders which has the role of horizontal bracing.

Indirect wind action: $M_{w.\mathrm{ind}}$ = 130 kN.m.

2.2. Verification of Main Girders Bending Strength

The main steel girders are made with circular holes in the girders web, as shown in Fig. 2:

a) the upper part – of rolled section IPE 600;

b) the lower part – of rolled section HEB 800;

c) holes of diameter: D = 640 mm;

d) distance between hole axes: S = 960 mm.

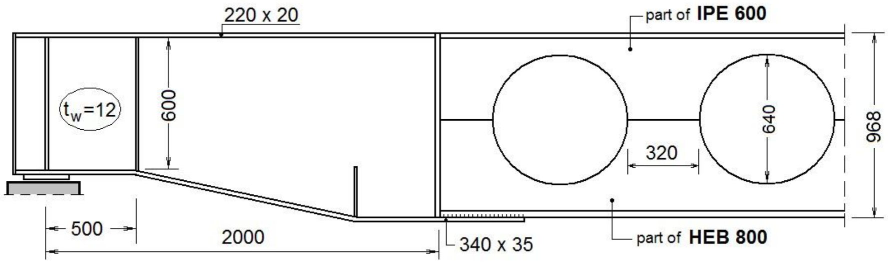


Fig. 2 – Main girder.

The main girders are calculated for the following hypotheses of structural members behaviour:

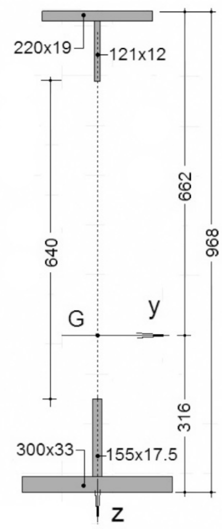
Hypothesis 1

a) Hollow girders behave in the same manner as full web girders; as the material is absent in the hole, the hollow part is obviously not considered in the calculus of the section characteristics;

b) The normal unit stresses observe the Navier law and Bernoulli hypothesis.

Phase 1: Steel girder not compounding with the concrete slab

The features of the steel girder in the circular hole are presented in Fig. 3. As the girder cross section is Class 1, the plastic strength modulus will be used.



Ax[cm²] =	182.44
Iy[cm⁴] =	321223.8
Iz[cm⁴] =	9119.6
Wy,pl[cm³] =	5485.7
Wz,pl[cm³] =	988.6
Wy,el,t[cm³] =	4924.4
Wy,el,b[cm³] =	10175.1
Wz,el,t[cm³] =	608.0
Wz,el,b[cm³] =	608.0

Fig. 3 – Main girder characteristics.

The resistance to bending of the steel girder section will be:

$$M_{c.Rd}^{\text{Steel}} = \frac{W_{\text{pl.}} f_y}{\gamma_{M0}} = 1{,}508 \text{ kN.m}$$

The calculus bending moment during the mounting Phase 1 is:

$$M_{Ed}^{F1} = \gamma_G M_{g1} = 700\,\text{kN.m}.$$

It yields: $M_{Ed}^{F1} / M_{c.Rd}^{\text{Steel}} < 1$.

The normal unit stresses in the extreme fibres of girder section are:

a) in the lower fibre: $\sigma_{ai}^{(1)} = M_{Ed}^{F1} / W_{y.el.b} = 688\,\text{daN/cm}^2$;

b) in the upper fibre: $\sigma_{as}^{(1)} = M_{Ed}^{F1} / W_{y.el.t} = 1{,}422\,\text{daN/cm}^2$.

The diagram of the normal unit stresses occurring in Phase 1 – a technological and mechanical girder behaviour phase, is presented in Fig. 4.

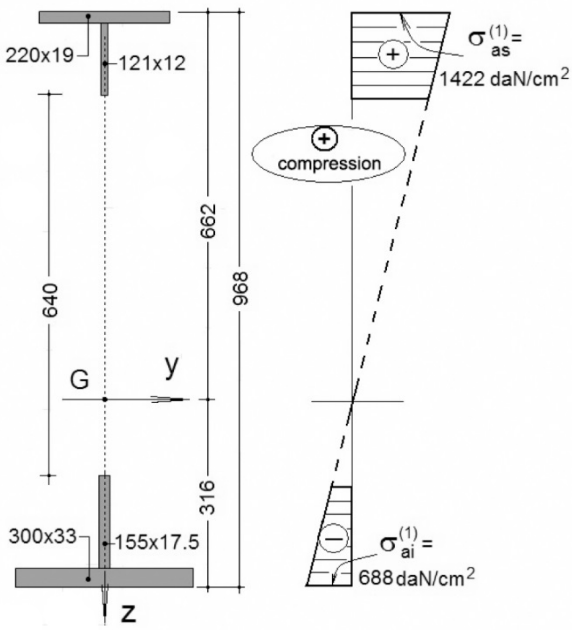


Fig. 4 – Stresses in Phase 1.

Phase 2: Steel girder compound with the concrete slab

In Phase 2 – a technological and behavioural girder related phase - the metal girder works with the reinforced concrete slab through the connectors that take over the sliding between the two components of the composite steel-concrete section.

The features of the composite girder are given in Fig. 5.

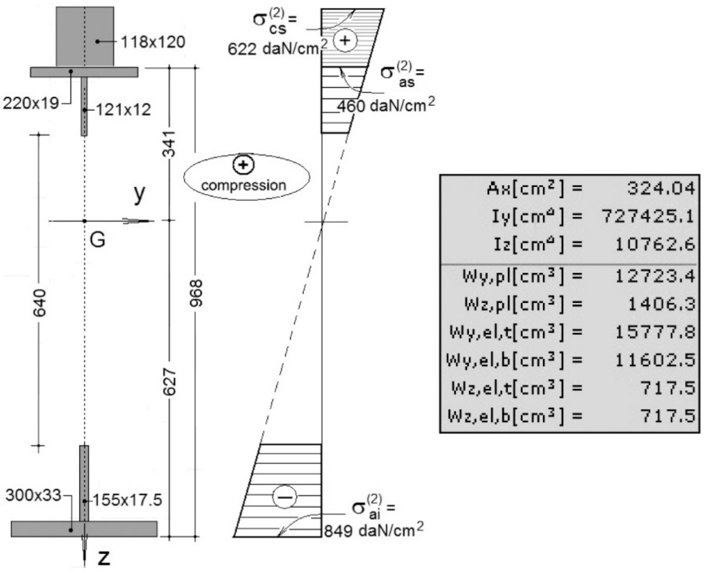


Fig. 5 – Stresses in Phase 2.

The ideal bending moment, in the technological and work Phase 2, results from the load group 1 (net load with people):

$$M_{Ed}^{F2} = \gamma_G M_{g2} + \gamma_{Q1} M_p + \psi_{0.w}\gamma_w M_{w.\text{ind}} = 982\,\text{kN.m}.$$

The normal unit stresses in the extreme ale fibres of the steel component and the concrete slab will be:

a) in the lower fibre of the steel girder: $\sigma_{ai}^{(2)} = M_{Ed}^{F2} \big/ W_{y.el.b}^{\text{compus}} =$ $= 849\,\text{daN/cm}^2$;

b) in the upper fibre: $\sigma_{as}^{(2)} = \left(M_{Ed}^{F2} \big/ I_{y.el}^{\text{compus}}\right) z_s = 460\,\text{daN/cm}^2$;

c) in the upper fibre of the concrete slab (steel equivalent): $\sigma_{cs}^{(2)} =$ $= M_{Ed}^{F2} \big/ W_{y.el.t}^{\text{compus}} = 622\,\text{daN/cm}^2$.

The total (final) unit normal stresses in the extreme fibres of the steel component and the concrete slab will be:

a) in the lower fibre of the steel girder:

$$\sigma_{ai} = \sigma_{ai}^{(1)} + \sigma_{ai}^{(2)} = 688 + 849 = 1{,}537\,\text{daN/cm}^2 < \frac{f_y}{\gamma_{M0}} = 2{,}750\,\text{daN/cm}^2$$

b) in the upper fibre:

$$\sigma_{as} = \sigma_{as}^{(1)} + \sigma_{as}^{(2)} = 1,422 + 460 = 1,882\,\text{daN/cm}^2 < \frac{f_y}{\gamma_{M0}};$$

c) in the upper fibre of the concrete slab (steel equivalent):

$$\sigma_{cs} = \sigma_{cs}^{(2)} = 622\,\text{daN/cm}^2 < n\frac{0.85 f_{ck}}{\gamma_c} = 12.72\frac{0.85\times 300}{1.5} = 2,162\,\text{daN/cm}^2.$$

Hypothesis 2

The hollow girders behave in the hollow area similarly to the flange of lattice girders, the axial stress being calculated from the M/h_0 ratio.

Phase 1: Steel girder not compounding with the concrete slab

In Fig. 6, the strength features of the upper and lower flange are given.

The section bending moment is taken over by a couple of two equal and opposite forces, $C = -I$, where: $C = -I = N_{0.Ed}^{F1} = M_{Ed}^{F1}/h_0$.

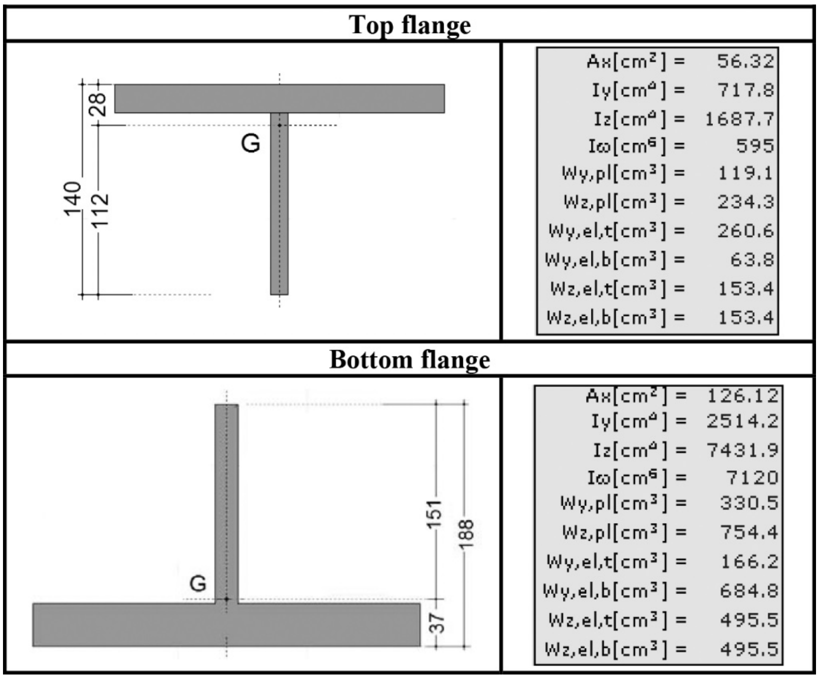


Fig. 6 – Flanges characteristics of the steel girder.

The condition is verified: $N_{0.Ed}^{F1}/N_{b.Rd} \le 1.0$.

The flange resistance to compression is that of a unit section compressed bar:

$$N_{b.Rd}=\begin{cases}\chi_{LT}\dfrac{A_f f_y}{\gamma_{M1}} & - \text{ for class 1, 2 or 3 cross-section;}\\ \chi_{LT}\dfrac{A_{f\cdot\text{eff}} f_y}{\gamma_{M1}} & - \text{ for class 4 cross-section.}\end{cases}$$

For sections of classes 1, 2 and 3 it results: $N_{b.Rd}=\chi_{LT}\left(A_0 f_y/\gamma_{M1}\right)$.

The reduction coefficient χ_{LT} considers the possibility to that the compresses flange loses its stability through lateral buckling, during Phase 1; it is determined with the reduced slender coefficient $\overline{\lambda}$:

$$\overline{\lambda}=\overline{\lambda}_{LT}=\overline{\lambda}_{TF}=\sqrt{\frac{A_0 f_y}{N_{0.cr.TF}}}\Rightarrow\chi \quad (\text{curve } \boldsymbol{d}).$$

The critical buckling force is determined with the relationship:

$$N_{0.cr.TF}=\frac{I_{0.0}}{2\left(I_{0.y}+I_{0.z}\right)}\times$$

$$\times\left[\left(N_{0.cr.z}+N_{0.cr.T}\right)-\sqrt{\left(N_{0.cr.z}+N_{0.cr.T}\right)^2-4\frac{\left(I_{0.y}+I_{0.z}\right)}{I_{0.0}}N_{0.cr.z}N_{0.cr.T}}\right],$$

where:

$$N_{0.cr.z}=\frac{\pi^2 EI_{0.z}}{L_{cr.z}^{\ 2}};\ N_{0.cr.T}=\frac{A_0}{I_{0.0}}\left(GI_{0.t}+\frac{\pi^2 EI_{0.\omega}}{L_{cr.T}^2}\right);\ I_{0.0}=I_{0.y}+I_{0.z}+A_0 z_s^2.$$

In this case, the critical length will be equal to the distance between the cross girders, respectively $L_{cr.z}$ 2.40 m, if measures are taken against the chord buckling until cross girders are fixed.

It yields:

$$I_{0.0}=I_{0.y}+I_{0.z}+A_0 z_s^2=2{,}588\,\text{cm}^4;\ I_{0.t}=57.3\,\text{cm}^4;$$

$$N_{0.cr.z}=0.607\times10^6\,\text{daN};\ N_{0.cr.T}=1.01\times10^6\,\text{daN};\ N_{0.cr.TF}=5{,}500\,\text{kN};$$

$$\overline{\lambda}_{LT}=\sqrt{\frac{A_0 f_y}{N_{0.cr.TF}}}=0.53;\ \chi_{LT}=0.76;$$

$$N_{b.Rd}=\chi_{LT}\frac{A_0 f_y}{\gamma_{M1}}=1{,}070\,\text{kN};\ N_{0.Ed}^{F1}=\frac{M_{Ed}^{F1}}{h_0}=775\,\text{kN}.$$

It gives: $N_{0.Ed}^{F1}/N_{b.Rd}=775/1{,}070=0.72<1.0$ – the relationship is verified.

Phase 2: Steel girder compound with the concrete slab

In Phase 2 – a technological and behavioural girder related phase - the metal girder works with the reinforced concrete slab through the connectors that take over the sliding between the two components of the composite steel-concrete section. In this phase, when concrete is hard, the issue of losing compressed flange stability is no more present, as the concrete slab prevents the occurrence of the phenomenon. Taking into consideration, similarly to Phase 1, the fact that the two flange are subjected to axial stresses, the bearing capacity of the girder shall be given by the minimal value of flange strength.

Another aspect to be considered is that a part of the bearing capacity of the metal flange was used during work Phase 1.

In Fig. 7, the strength features of the upper and lower chords are given.

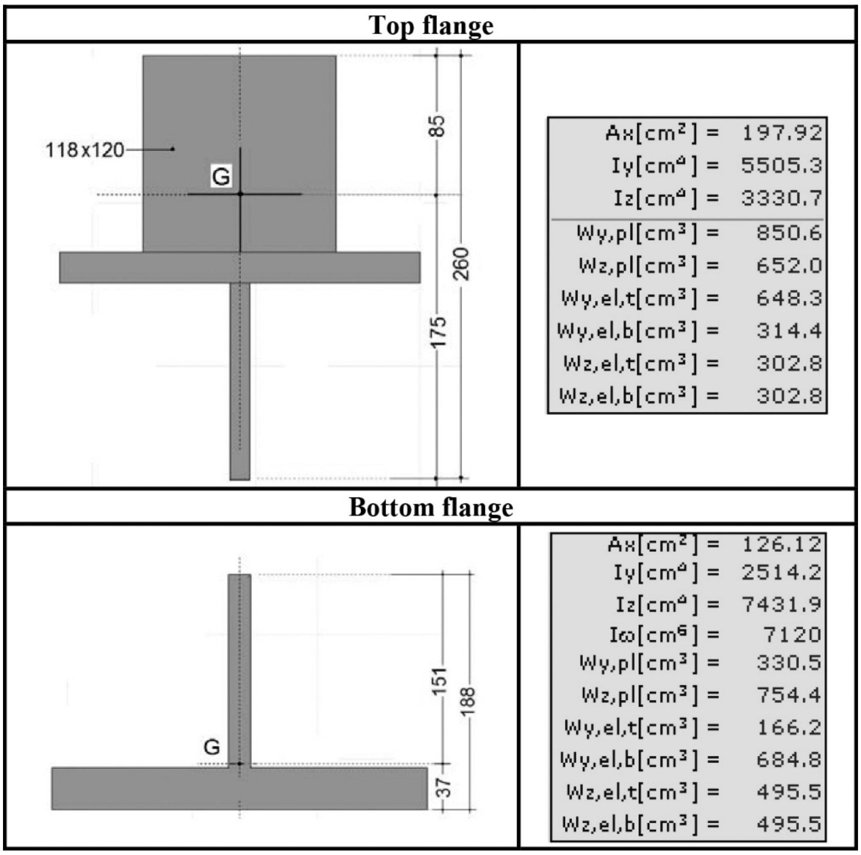


Fig. 7 – Flanges characteristics of the composite girder.

The axial chord forces in Phase 2 are found:

$$N_{0.Ed}^{F2} = \frac{M_{Ed}^{F2}}{h_{0.2}} = \frac{982}{0.966} = 1,016\,\text{kN}$$

The lower flange is calculated with relationship: $\dfrac{N_{0.Ed}^{F1} + N_{0.Ed}^{F2}}{N_{c.Rd}^{T.\text{inf.}}} \leq 1$.

The tensile flange strength is: $N_{c.Rd}^{T.\text{inf.}} = \dfrac{A_{TI} f_y}{\gamma_{M0}} = 3{,}468$ kN.

It yields: $\dfrac{N_{0.Ed}^{F1} + N_{0.Ed}^{F2}}{N_{c.Rd}^{T.\text{inf.}}} = \dfrac{775 + 1{,}016}{3{,}468} = 0.52 < 1$

For the upper chord, the minimal bearing capacity resulting from the steel strength or from the concrete slab strength is considered.

According to the stress state in Fig. 8, the relations below will be applied:

a) in the steel: $\sigma_a = N_{0.Ed}^{F1} / \chi_{LT} A_{TS}^{\text{Steel}} + N_{0.Ed}^{F2} / A_{TS}^{\text{Total}} \leq f_y / \gamma_{M0}$;

b) in the steel equivalent concrete: $\sigma_c = N_{0.Ed}^{F2} / A_{TS}^{\text{Total}} \leq n(0.85 f_{ck} / \gamma_c)$.

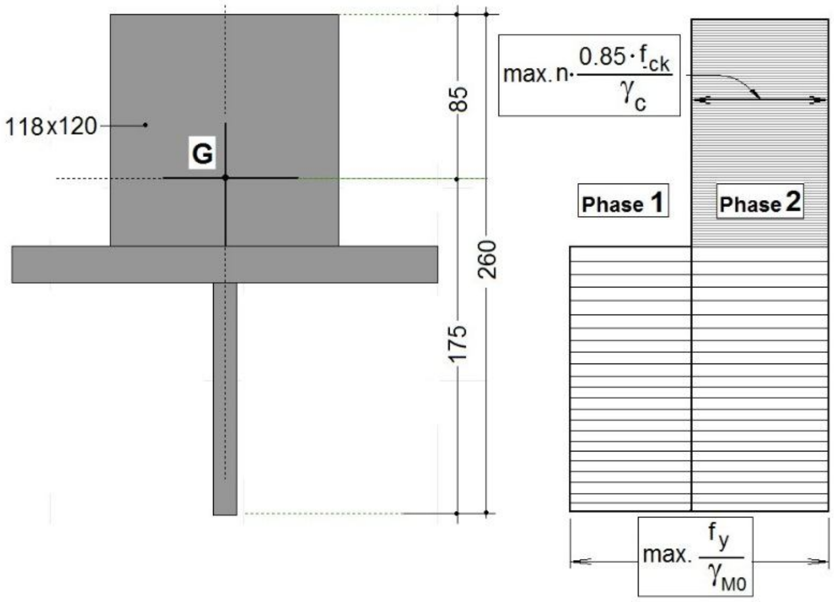


Fig. 8 – Total stresses in the top flange.

It yields:
a) in the steel:

$$\sigma_a = \frac{N_{0.Ed}^{F1}}{\chi_{LT} A_{TS}^{\text{Steel}}} + \frac{N_{0.Ed}^{F2}}{A_{TS}^{\text{Total}}} = 2{,}324\,\text{daN/cm}^2 < \frac{f_y}{\gamma_{M0}} = 2{,}750\,\text{daN/cm}^2;$$

b) in the steel equivalent concrete:

$$\sigma_c = \frac{N_{0.Ed}^{F2}}{A_{TS}^{\text{Total}}} = 513\,\text{daN/cm}^2 < n\frac{0.85 f_{ck}}{\gamma_c} = 2{,}162\,\text{daN/cm}^2.$$

If the reduction coefficient χ_{LT} is not taken into account, the unit stress in the metal flange reaches the value: σ_a = 1,890 daN/cm^2.

The girder cross section verification at a certain distance from the hole middle is made complying with the ideas expressed in paper (Moga, 2013, 2014), which are synthesised in Figs. 9 and 10.

The verification is performed in areas where high simultaneous values for the bending moment and shear force are present.

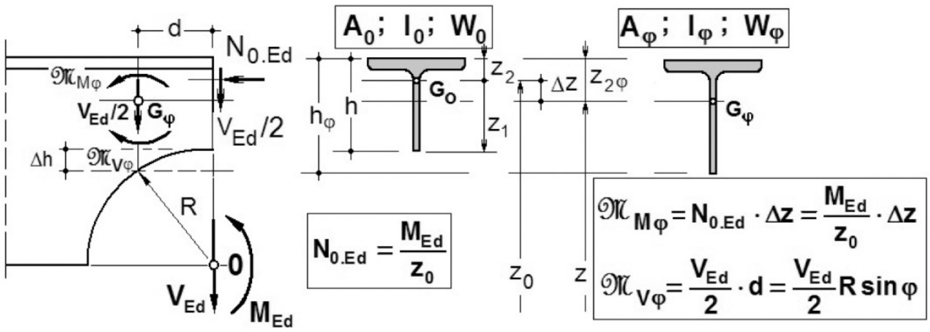


Fig. 9 – Model for the evaluation of the state of stresses.

In Table 1 a comparative analysis of the unit stresses found through the two methods and hypotheses is presented.

Table 1

Stress	Method 1 daN/cm^2	Method 2 daN/cm^2	$\frac{\sigma_{\text{method1}}}{\sigma_{\text{method 2}}}$
Top flange	1,537	1,420	1.08
Bottom flange	1,882	1,890	0.99*
Concrete slab	622/n	513/n	1.21

* without χ_{LT}

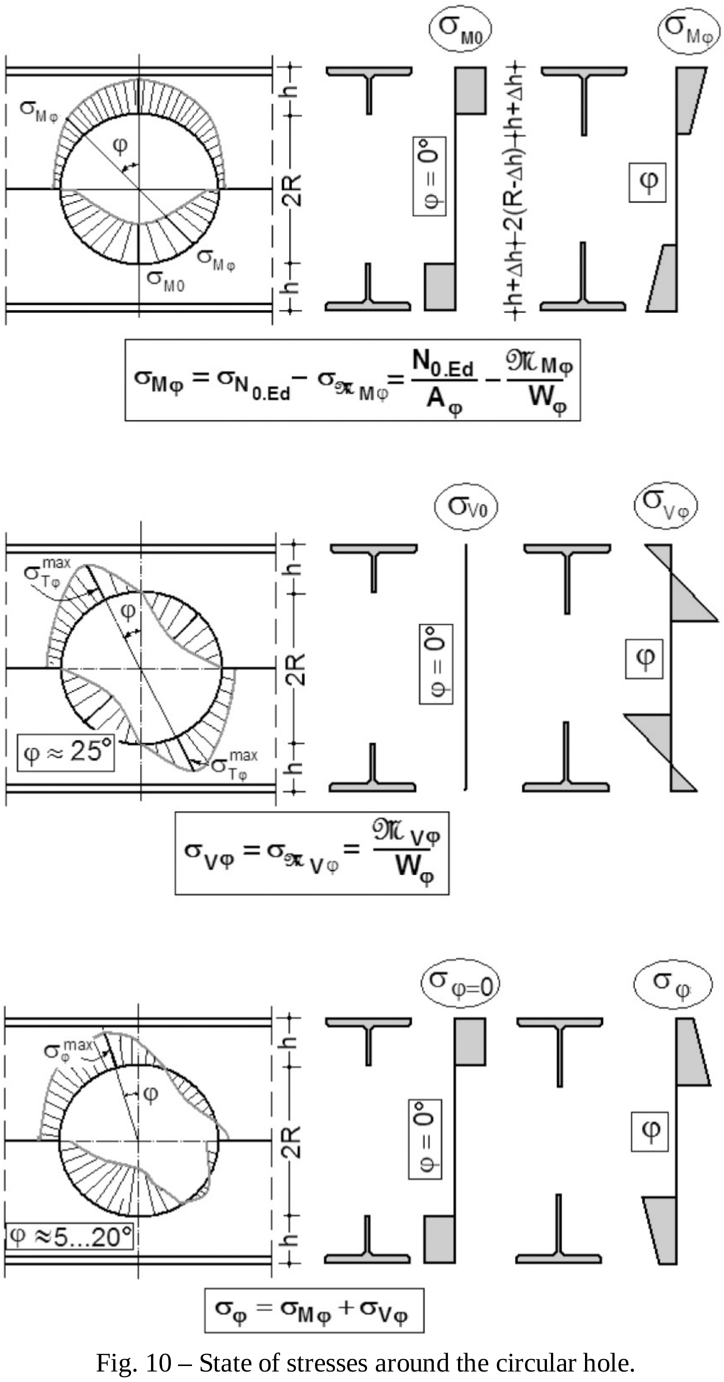


Fig. 10 – State of stresses around the circular hole.

2.3. Natural Structure Frequency

The upper natural frequency of the structure for vibration Mode 1 is verified with the relationship:

$$f_n = \frac{n^2\pi}{2L^2}\sqrt{\frac{EI}{\rho S}} \;\Rightarrow\; f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI}{\rho S}}$$

where: L = 24 m; $E = 210 \times 10^9$ N/m^2.

The inertia moment is calculated for short term loads.

The average inertia moment is taken into consideration, between the hole area moment and the full section area.

$$n_0^{s.t.} = \frac{E_a}{E_{cm}} = \frac{2.1\times10^6}{3.3\times10^5} = 6.36 \text{ (s.t. = short term).}$$

It yields:

In the hole area: $I_1 = 2\cdot I_{y.el.gol}^{compus} \approx 2\cdot 887\,613\,cm^4 = 1.775\cdot10^{-2}\ m^4$.

In the full section area:

$$I_2 = 2I_{y.el.\text{plin}}^{\text{compus}} = 2\times952{,}090\,\text{cm}^4 = 1.9\times10^{-2}\,\text{m}^4 = 1.07\times I_{y.el.\text{gol}}^{\text{compus}}.$$

The average inertia moment:

$$I = \frac{I_1 + I_2}{2} = 1.84\times10^6\,\text{cm}^4 = 1.84\times10^{-2}\,\text{m}^4.$$

Unloaded footbridge linear density: ρS = 1,440 + 370 = 1,810 kg/m.

It gives: f_1 = 3.98 Hz – *low resonance risk.*

3. Conclusions

In the field of small and average sized openings, the most used constructive solutions for the footbridge superstructure concern steel girders, made of rolled profile, full section welded section girders, hollow girders and lattice girders.

Metal girders can be made in structural work with a reinforced concrete slab, component of the flooring, forming a composite steel-concrete structure. In the case of pedestrian footbridges, besides the verifications for ultimate limit states and service limit states, it is necessary to check for the traffic comfort, which is in direct correlation with the structure frequency (risk of resonance) and acceleration.

The girder under investigation having a composite steel-concrete cross section with circular holes satisfies the ULS and SLS verifications and also the conditions for resonance risks and traffic comfort.

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PASARELE PE GRINZI METALICE COMPOZITE CU GOLURI CIRCULARE

(Rezumat)

Sunt prezentate câteva aspecte privind calculul şi alcătuirea pasarelelor pietonale realizate pe grinzi compuse oţel-beton, având grinzile realizate din profile metalice laminate cu goluri circulare. În cazul pasarelelor pietonale, pe lângă verificările corespunzătoare stărilor limită ultime şi ale stărilor limită de serviciu, este necesar să fie verificat confortul de circulaţie al pietonilor, aflat în corelare directă cu frecvenţa de vibraţie a structurii (riscul de rezonanţă) şi cu acceleraţia acesteia.