TECHNICAL TRANSACTIONS | **CZASOPISMO TECHNICZNE**
CIVIL ENGINEERING | BUDOWNICTWO

4-B/2014

KAZIMIERZ FURTAK*

FATIGUE BEARING STRENGTH OF CONNECTION WITH FLEXIBLE CONNECTORS

NOŚNOŚĆ ZMĘCZENIOWA ZESPOLENIA Z ŁĄCZNIKAMI WIOTKIMI

Abstract

The paper presents an analysis of the effect of load variations on the bearing strength of connection in steel–concrete elements with flexible connectors in the form of bolts. Data relating to the residual stresses from concrete slab shrinkage have been included in the analysis. The results of the author's experiments on the effect of variable loads on the bearing strength of the connection have been discussed following the author's experimental studies. Following the analysis of the results, it can be stated that the bearing strength of a connection with flexible connectors is significantly affected by the rigidity of the steel girder and slab (coefficient δ) together with the parameters of load cycles (κ, *R*) and the value of concrete shrinkage strain.

Keywords: *fatigue bearing strength*, *flexible connectors*, *variable loads*, *load cycles*, *shrinkage strain*

Streszczenie

W artykule poddano analizie wpływ zmienności obciążenia na nośność zespolenia w elementach typu stal–beton z łącznikami wiotkimi w formie sworzni. W analizie uwzględniono naprężenia własne od skurczu betonu płyty. Wpływ obciążeń zmiennych na nośność zespolenia uwzględniono na podstawie wyników własnych badań doświadczalnych. Na podstawie wyników przeprowadzonych analiz można stwierdzić, że istotny wpływ na nośność zespolenia z łącznikami wiotkimi ma stosunek sztywności dźwigara stalowego i płyty (współczynnik δ) oraz parametry cyklu obciążenia (κ, *R*), a także wartość odkształceń skurczowych betonu.

Słowa kluczowe: *nośność zmęczeniowa*, *łączniki wiotkie*, *obciążenia zmienne*, *liczba cykli*, *odkształcenia skurczowe*

* Prof. Ph.D. Eng. Kazimierz Furtak, Institute of Building Materials and Structures, Faculty of Civil Engineering, Cracow University of Technology.

1. Introduction

In calculating the connection bearing strength, the disintegrating forces from external loads operating in the reinforced concrete slab–steel girder contact plane are usually taken into account. These forces are compared with the bearing strength of the connection, identified with the bearing strength of the connectors. In such calculations, the forces operating in the connection plane are assumed to be constant and equal to the maximum ones. In actual reality, forces occur in the connection plane from residual stresses originating from concrete shrinkage and from the temperature difference between the concrete slab and the steel girder as well as from cement hydration heat [2, 3, 6]. Moreover, external loads can be time variable, which can lead to fatigue.

Residual stresses from concrete hydration occur only at the initial phase of concrete curing when its modulus of elasticity is still insignificant, thus, they do not have any major practical effect on fatigue strength. This is why they are usually disregarded in calculations. The stresses originating from the temperature difference between the concrete slab and the steel girder have also been neglected because in instances of seasonal temperature change, this difference arises very slowly in comparison to the usual load change rate. Besides, in the case of twenty-four hour changes, the effect can be either favourable or unfavourable.

The paper presents an analysis of the effect of load variations on the bearing strength of the connection in steel–concrete elements with flexible connectors in the form of bolts Residual stresses from concrete slab shrinkage are included in the analysis. The results of the author's experiments on the effect of variable loads on bearing strength of the connection are discussed following the author's experimental studies.

2. Effect of variable loads on connection bearing strength

The effect of variable loads on the bearing strength of the connection with flexible connectors has been determined on the basis of the results of the author's experiments carried out within a broader project on steel–concrete composite elements. The connectors used were 75 mm in length, d = 10 mm and 16 mm in diameter. Apart from the connectors' diameter, the other variables used in the tests included [3]: concrete compressive strength $\overline{f}_c$ and shear strength $\overline{f}_{ct}$ (concrete I: $\overline{f}_c = 34.80$ MPa, s_c = 1.71 MPa, $\overline{f}_{ct} = 2.36$ MPa, s_t = 0.12 MPa, concrete II: $\overline{f}_c = 45.96$ MPa, s_c = 2.16 MPa, $\overline{f}_{ct} = 3.08$ MPa, s_t = 0.15 MPa, concrete III: $\overline{f}_c = 61.10$ MPa, s_c = 2.63 MPa, $\overline{f}_{ct} = 4.10$ MPa, s_t = 0,19 MPa), maximum load $P_{\max}$ to ultimate strength P_n ($\kappa = P_{\max}/P_n$; κ = 0.60, 0.70, 0.80) ratio, and stress ratio $R = P_{\min}/P_{\max}$; R = 0.10, 0.20, 0.30), where $P_{\min}$ is the cycle maximum load. The measured value was the limit number of load cycles N resulting in fatigue failure. The tests were carried out on 72 elements.

On the basis of experimental results, the relation between the connection ultimate bearing strength R_{m} and fatigue strength R_{mN} can be adopted as:

$$R_{mN} = \kappa_s R_m \tag{1}$$

where κ_s is a coefficient dependent on the number and parameters of load cycles.

The value of coefficient κ_s can be calculated from the formula:

$$\kappa_s = 1.05 - 0.095(1 - R)\log N \tag{2}$$

where N is the number of load cycles leading to a loss of bearing strength due to changing loads.

Transformation of formula (2) leads to:

$$\log N = (1.05 - \kappa_s)/[0.095(1 - R)] \tag{3}$$

With formula (3) it is possible to compare the results of the author's experiments with the proposal of taking into account the effect of load variations in determining the bearing strength of a connection with flexible connectors. The following has been obtained [3]:

- for $\kappa = 0.60$ and $R = 0.10$; $N_t = 183.30 \cdot 10^3$ at $N_e = 191.57 \cdot 10^3$ (concrete II),
- for $\kappa = 0.70$ and $R = 0.20$; $N_t = 40.30 \cdot 10^3$ at $N_e = 47.69 \cdot 10^3$ and $N_e = 42.57 \cdot 10^3$ (concrete I), $N_e = 47.69 \cdot 10^3$ (concrete II) and $N_e = 51.37 \cdot 10^3$ (concrete III),
- for $\kappa = 0.80$ and $R = 0.30$; $N_t = 5.75 \cdot 10^3$ at $N_e = 5.99 \cdot 10^3$ (concrete II),

where N_t is a theoretical number of load cycles calculated from (3), while N_e is the limit number of cycles obtained in the experiment. The test results confirm the correctness of formula (2).

The results of experiments carried out by the author are convergent with the results obtained by other authors, including those in Fig. 1 [4] and Fig. 2. In these Figures, the difference of maximum and minimum shear stresses ($\Delta\tau = \tau_{max} - \tau_{min}$) depends on the number of load cycles N. A similar approach was adopted in EC4 [1] and publications [8–10].

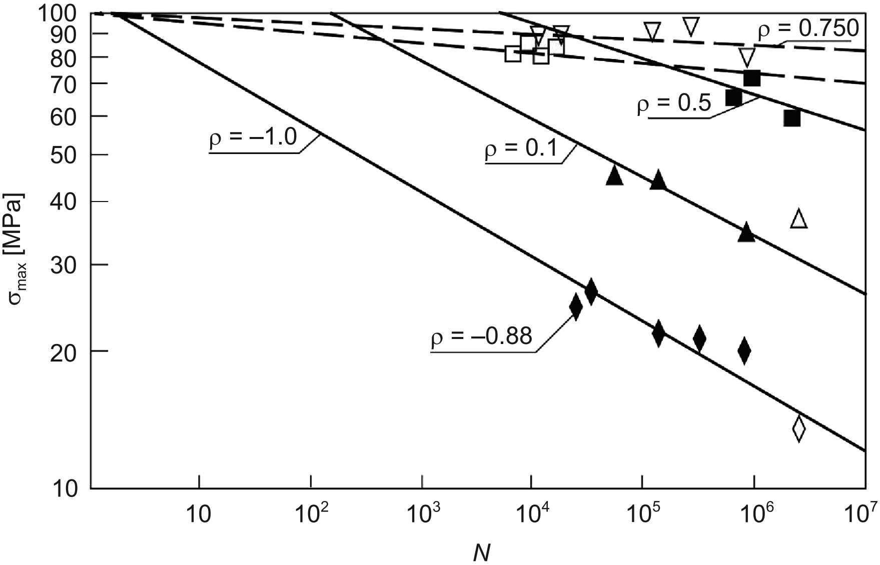

Fig. 1. Results of fatigue tests on flexible connectors [4]

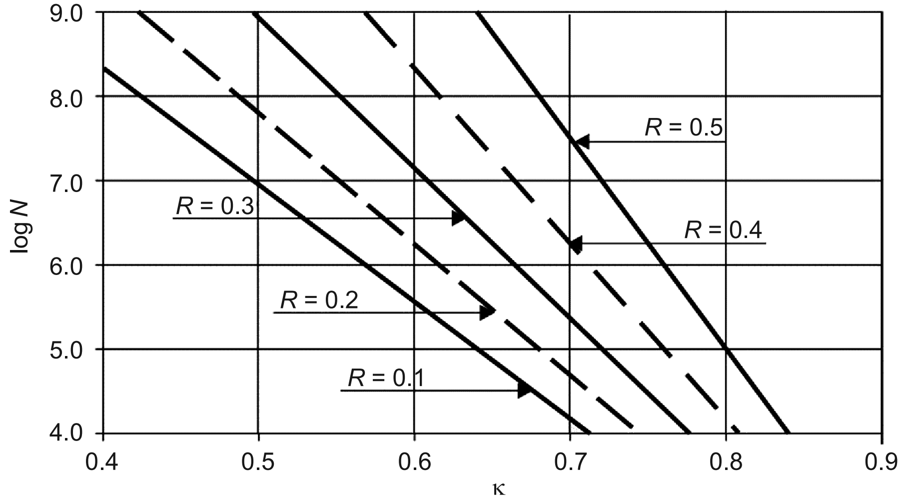

Fig. 2. Effect of parameters of load cycles (κ, *R*) on their limit number *N* (log*N*)

In the author's proposal formulae (2) and (3), fatigue bearing strength of the connection was made dependent on two parameters: the number of load cycles *N*; stress ratio *R*. The same parameters were decisive with regard to the difference between the maximum and minimum stresses in [4] and in Fig. 1. W EC4 [1] and in [8], inter alia, the difference in shear stress Δτ was made dependent only on the number of load cycles *N*.

Fatigue bearing strength dependence on a single parameter is profitable in structural design, which was demonstrated in EC4 [1]. In the analysis of the experimental results and theoretical solutions, it is more advantageous to introduce an additional parameter. This is why this approach was adopted in the present paper, which is justified by the test results quoted in [2, 4, 8–10].

The relationship between parameters κ and *R* and Δ*P* (Δτ) is given by the formula:

$$\Delta P = P_{\max} - P_{\min} = P_{\max}(1-R) = \kappa(1-R)R_m \qquad (4)$$

3. Effect of concrete shrinkage on stress state in connection plane

Following [2], maximum shear stresses from concrete shrinkage in the connection plane can be calculated from the formula:

$$\tau_{\max,s} = \frac{0.85\varepsilon_s E_c}{m(\delta+1)(1+\alpha\rho)}\,\mathrm{tg}h(ms_r) \qquad (5)$$

where:

- $\varepsilon_s(t)$ – free shrinkage strain,
- $E_c(t)$ – concrete modulus of elasticity,
- s_r – hypothetical crack spacing (calculated on the assumption that there is no external load acting on the composite element):

$$s_r = \frac{1}{m} \operatorname{arccos} h \frac{\varepsilon_{cs}(t)}{\varepsilon_{cs}(t) - \varepsilon_{ct,\lim}} \tag{6}$$

m – coefficient including slab reinforcement, approximately:

$$m = \sqrt{\frac{3.4\gamma_z}{A_c}} \tag{7}$$

γ_z – coefficient including reinforced concrete slab reinforcement, approximately:

$$\gamma_z = \frac{1 + 0.75\alpha\rho}{1 + \alpha\rho} \tag{8}$$

α – modulus of elasticity of reinforcement E_a to modulus of elasticity of slab concrete ratio E_c ($\alpha = E_a/E_c$),
ρ – reinforced concrete slab reinforcement ratio,
A_c – reinforced concrete slab cross-section,
x – distance of the cross-section in question from the end of beam or cross-section with a crack

$$\varepsilon_{cs}(t) = \frac{\delta_c}{\delta_a + \delta_c} \varepsilon_s(t) \tag{9}$$

$$\delta = \frac{\delta a}{\delta c} \tag{10}$$

δ_a – generalised modulus of rigidity of steel girder

$$\delta_a = \frac{1}{E_a A_a} + \frac{z^2}{E_a I_a} \tag{11}$$

z – distance of centres of gravity of steel girder and reinforced concrete slab,
E_a – modulus of elasticity of steel,
A_a – cross-section of steel girder,
I_a – moment of inertia of steel girder relative to its axis of inertia,

$$\delta_c = \frac{1}{E_c A_c} \tag{12}$$

$\varepsilon_{ct,\lim}$ – limit concrete extension that can be calculated from the formula

$$E_{ct,\lim} = \frac{f_{ctm}}{E_c} \eta_\phi \tag{13}$$

where:

$$\eta_\phi = 1 + \frac{0.08\rho_r}{\phi^{1.5}} \tag{14}$$

f_{ctm} – mean concrete tensile strength [MPa],
E_{ct} – concrete modulus of tensile elasticity [MPa] adopted as equal to concrete modulus of compressive elasticity E_c,

ρ_r – reinforcement ratio [–] referring to effective area of cross-section of concrete in tension,

ϕ – diameter of reinforcement rods [m]; only in formula (14).

When $\varepsilon_{cs}(t) \leq \varepsilon_{ct,\lim}$ distance s_r should be adopted as equal to the distances between the cracks from external loads; when these do not occur calculation-wise, s_r is adopted as equal to the element length.

4. Analysis of connection bearing strength

The results of analysis as to the limit number of load cycles N ($\log N$) have been shown in Figures 1 and 2. The independent variables adopted were the operating maximum disintegrating force $P_{\max}$ to connection bearing strength R_{m} ($\kappa = P_{\max}/R_{\mathrm{m}}$) ratio and the stress ratio R ($R = P_{\min}/P_{\max}$; $P_{\min}$ – cycle minimum load, $P_{\max}$ – cycle maximum load).

The real degree of connection bearing strength being exhausted at maximum load is $\kappa \approx 0.6$, while the stress ratio is $R \approx 0.2$. This indicates that reaching the connection fatigue strength is real. This is valid if there are no factors causing a decrease of coefficient κ and an increase of coefficient R.

Figure 3 illustrates the dependence of maximum shrinkage stresses τ in the connection plane on shrinkage strains ε_{s} (free shrinkage) and on the value of parameter δ expressing the interrelationship between the steel girder rigidity and the reinforced concrete slab bonded with it. The values of strains τ at average values of free shrinkage ($\varepsilon_s \approx 2.0 \cdot 10^{-4}$)

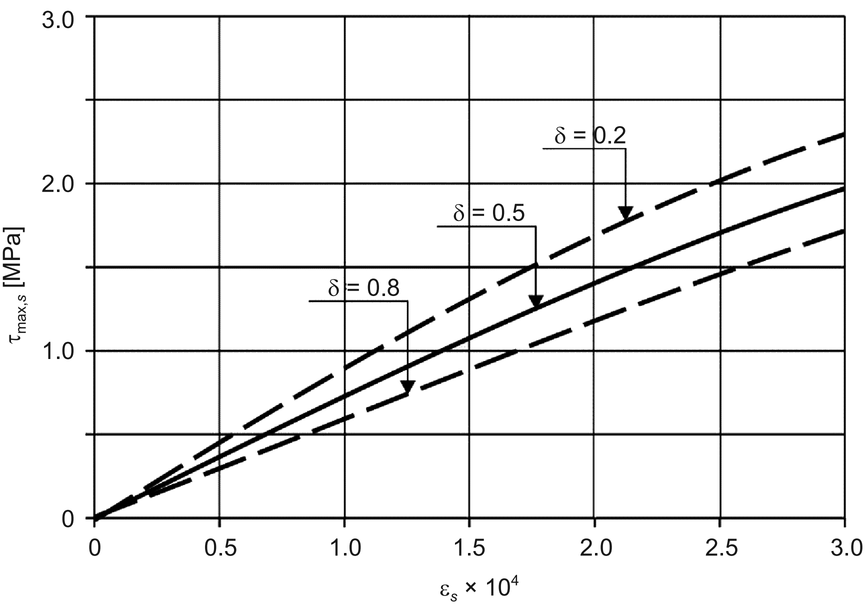

Fig. 3. Effect of concrete free shrinkage ε_s and rigidity of steel girder and connected reinforced concrete slab (δ) on shrinkage stresses τ in connection plane

are comparable with concrete adhesion to the steel girder ($\tau_p \approx 0.03 f_c$ [2]; f_c – concrete compressive strength). This indicates that concrete adhesion to the steel girder flange should not be taken into account when calculating the connection fatigue strength.

Shrinkage stresses are, in cross-sections of utmost effort, of opposite direction relative to disintegrating stresses from external loads. This indicates that they reduce the real value of coefficient κ, and in this way, increase the number of load cycles leading to fatigue strength being exceeded. A change of the value of κ can be described by an additional parameter λ equal:

$$\lambda = \kappa_{ws} / \kappa_w \tag{15}$$

where:

- κ_{ws} – coefficient κ calculated with shrinkage stresses taken into account:

$$\kappa_{ws} = P_{\max} / R_{ms} \tag{16}$$

$$N_{ms} = R_m + R_s \tag{17}$$

- κ_w – coefficient κ calculated disregarding shrinkage stresses:

$$\kappa_w = P_{\max} / R_m \tag{18}$$

- $P_{\max}$ – cycle maximum load,
- R_{ms} – connection bearing strength with concrete shrinkage included,
- R_m – connection bearing strength disregarding concrete shrinkage,
- R_s – disintegrating force in connection plane, from concrete shrinkage:

$$R_s = \tau_s A_p \tag{19}$$

- A_p – connection surface investigated.

After transformations we obtain:

$$\lambda = 1 / (1 + R_s / R_m) \tag{20}$$

The plots for λ are shown in Fig. 4. They depend on the interrelationships between the steel girder rigidity and reinforced concrete slab represented by δ and on the value of free shrinkage ε_s. The higher ε_s, the lower the value of λ. The lower the value of δ, the higher λ. This indicates that the value of λ increases with increases of steel girder rigidity relative to the reinforced concrete slab cooperating with it.

A change in the value of κ results in a change in the limit number of cycles *N* which is followed by the fatigue failure of the connection (the connection fatigue strength becomes fully exhausted). The effect of concrete shrinkage on the limit number of cycles *N* (and log*N* to be more precise) can be expressed by introducing factor μ, defined by formula:

$$\mu = \log N_{ms} / \log N_m \tag{21}$$

where:

- N_{ms} – limit number of load cycles with concrete shrinkage included,
- N_m – limit number of load cycles disregarded concrete shrinkage.

After transformations we receive:

$$\mu = (1 - \lambda \kappa_w) / (1 - \kappa_w) \tag{22}$$

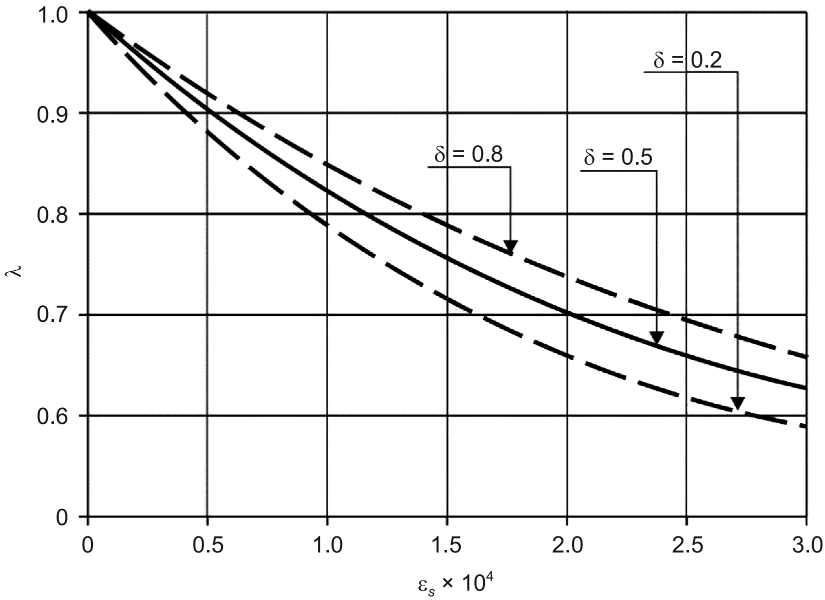

Fig. 4. Effect of concrete free shrinkage ε_s and rigidity of steel girder and connected reinforced concrete slab (δ) on coefficient λ

The value of μ depends on the value of free shrinkage ε_s and coefficient δ, as well as κ (cf. Figs. 5, 6 and 7). The higher the values of κ_s and κ, the higher the value of μ, and, consequently, the higher the fatigue durability. Additionally, the value of μ increases with the increase of the steel girder rigidity relative to the reinforced concrete slab rigidity.

5. Conclusions

It was proved in the paper that concrete shrinkage does have an effect on the connection static (ultimate) grammatical errors, meaning unclear and fatigue bearing strength. For ordinary conditions, the extreme shrinkage stresses are higher than the concrete adhesion to the steel girder flange. Therefore, this adhesion should not be taken into account in calculating the connection bearing strength.

The shrinkage stresses in the connection plane reduce the stresses from external loads. This indicates the possibility of allowing a higher external load. The reduction of stresses from external loads reduces the value of coefficient κ, which is equal to the cycle maximum load to bearing strength (of the connection) ratio.

With a decrease of the value of κ, the limit number of cycles at which the connection bearing strength becomes exhausted increases. Following the results analysis presented in Figures 5–7, it can be stated that when concrete shrinkage is disregarded, a fatigue hazard appears in calculations. When concrete shrinkage is taken into account, there is no such hazard.

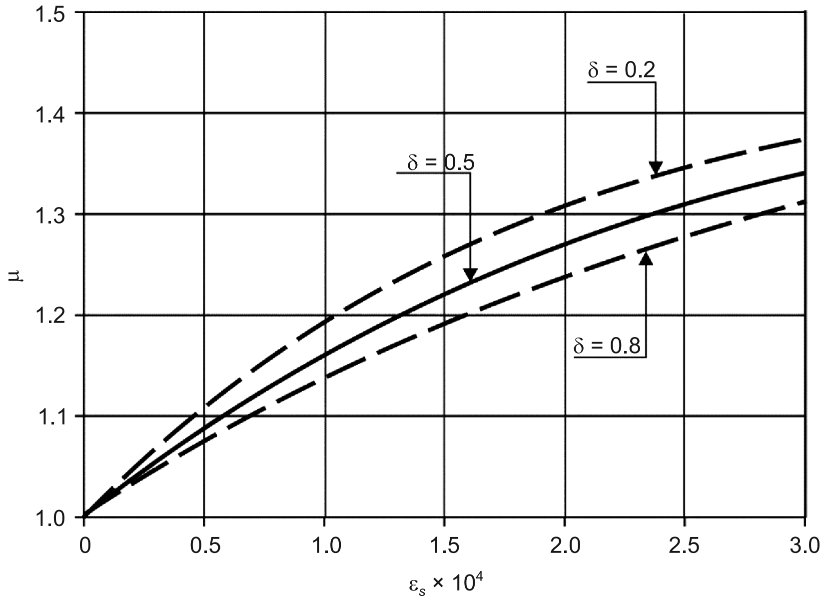

Fig. 5. Effect of concrete free shrinkage ε_s and rigidity of steel girder and connected reinforced concrete slab (δ) on coefficient μ at κ = 0.5

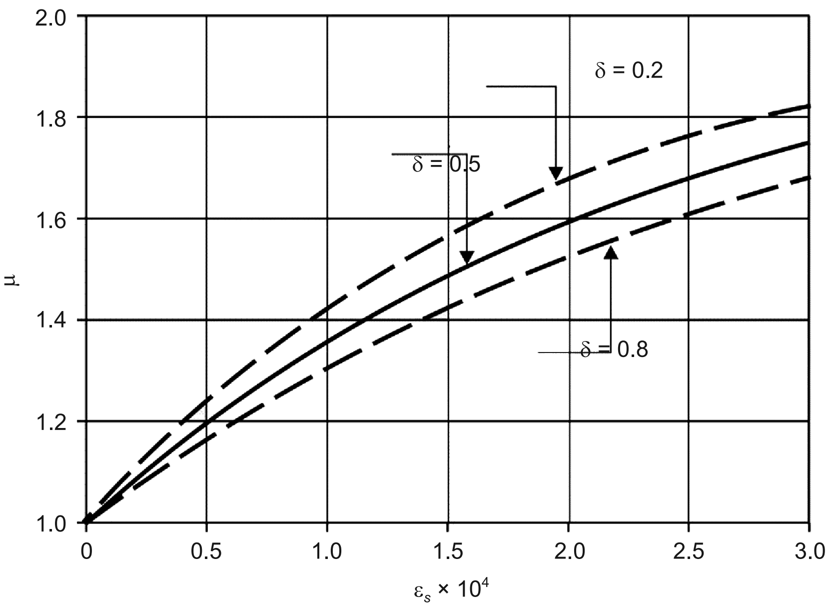

Fig. 6. Effect of concrete free shrinkage ε_s and rigidity of steel girder and connected reinforced concrete slab (δ) on coefficient μ at κ = 0.5μ and at κ = 0.7

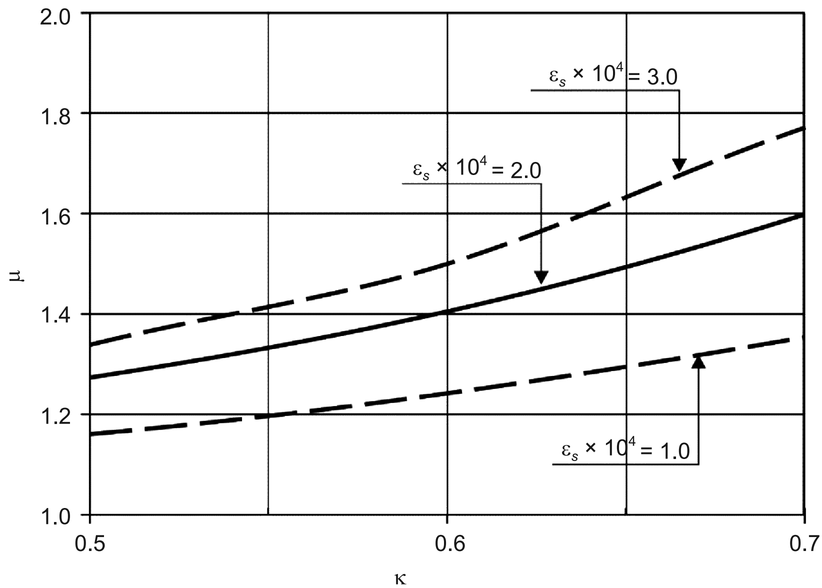

Fig. 7. Effect of concrete free shrinkage ε_s and coefficient κ on coefficient μ at δ = 0.5

The proposal for calculating fatigue strength has been supported by the results of the experiments carried out by the author. Following the results analysis, it can be stated that the bearing strength of a connection made with strip connectors is significantly affected by the rigidity of the steel girder and slab (coefficient δ) together with the parameters of the load cycles (κ, *R*) and the value of the concrete shrinkage strain.

Final report of research project No. 7 T07E 025 14. Author K. Furtak.

References

[1] PN-EN1994-2: 2010: Eurocode 4 – Design of composite steel and concrete structures – Part 2: General rules and rules for Bridges.
[2] Furtak K., *Mosty zespolone* (*Composite bridges*), PWN, Warszawa–Kraków 1999.
[3] Furtak K., *Wpływ skurczu betonu i obciążeń zmiennych na nośność zespolenia w elementach typu stal – beton* (*Effect of concrete shrinkage and changing loads on connection bearing strength in steel–concrete elements*), Inżynieria i Budownictwo, 1–2/2009.
[4] Knowles P.R., *Composite Steel and Concrete Construction*, Butterworth and Co., Ltd. London 1973.
[5] Kuczma M., Kuczma B., *Steel-concrete composite beams with elasto-plastic connection* [In:] *Mathematical methods in continuum mechanics*, Technical University of Lodz Press. Lodz 2011.
[6] Łącki J., *Nośność zespolenia żelbetowej płyty z dźwigarem stalowym* (*Load capacity of connection reinforced concrete slabs with steel girder. PhD thesis*), praca doktorska, Kraków 2004.

[7] Madaj A., *Wytrzymałość zmęczeniowa łączników w stalowo-betonowych belkach zespolonych* (*Fatigue bearing strength of connectors in steel-concrete composite beams*), Prace IBDiM, nr 1–2/1997, Warszawa.
[8] Pechar J., Bureš J., Schindler A., *Kovovė mosty*, SNTL, ALFA, Praha 1990.
[9] Roik K., Hannswille G., *Zur Zur Dauerfestigkeit von Kopfbolzendübeln bei Verbundträgern*, Der Stahlbau, 10/1983.
[10] Šimalová M., Bujňák J., *On determination of shear connection behavior by push test*, Transcom Conference University of Žilina, Žilina 1997.