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THE ANALYSIS OF REINFORCED CONCRETE BOX GIRDER VIADUCT DEFECTS AND THEIR ESTIMATION

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Abstract. The article describes the main damages of box girder viaducts constructed in Lithuania and discusses the reasons for their occurrence. Several viaducts are examined in more detail with regard to their design data and experimental results. Using finite element software "Midas Civil" the distribution of the main stresses was determined in the box and reduced girders of the Pareizgupis viaduct caused by symmetrical and asymmetrical loads. The widths of flexural cracks in the Pareizgupis viaduct are analyzed evaluating the behaviour of prestressed reinforcement suffered from corrosion. The above mentioned program was used to calculate the deflection of viaduct when damage of reinforcement is estimated. The paper examines the possibility to evaluate viaduct durability using the probability methods based on the initial investigation data.

Keywords: cracking, wire bunch corrosion, finite element method (FEM), modelling, stresses, defects, deflection, creep, bridge.

1. Introduction

Prestressed reinforced concrete became firmly used in bridge construction in the 1950s, after the Second World War. In 1956, the first prestressed concrete box girder bridge was constructed in the USA (Hewson 2003) which is still one of the longest bridges in the world today. In 1969, the first prestressed concrete box girder bridge was constructed in Lithuania.

At present, there are around 4000 bridges and viaducts in Lithuania, with the overall length of 93 km. The majority of bridges are constructed in motorways and only around 14% – in railways. The highest number of defects and damages are found in reinforced concrete bridges (Kamaitis 1995). 95% of Lithuanian bridges are reinforced concrete bridges. The greatest concern has been caused by the prestressed concrete frame – box girder viaducts constructed over the main Lithuanian highways during the period of 1968–1983. There are 17 viaducts of this type reinforced by stressed wire bunches made from wires of high strength: sixteen viaducts have a span of (16 + 48 + 16) m and one viaduct has a span of (18 + 36.1 + 18) m. All these viaducts were erected by the cantilever method. Prefabricated segments are joint together using epoxy glue. The above mentioned viaducts became the object of concern from 1995 when, carrying out static experiments, it was found that some viaducts were in pre-emergency condition. Since then all viaducts are inspected regularly (viaduct decks are graded twice a year) and the cracks opening in box girders are observed.

The paper pays more attention to the defects and damages of the Pareizgupis viaduct (Fig. 1) conducting its thorough theoretical and analytical research, the results of which are described in the following chapters.

The static design scheme of the Pareizgupis viaduct is a three-span six times statically indeterminate frame with hinges at supports. A mobile hinge is erected in the first support and a fixed hinge in the other one. The bottom height of a frame beam varies according to the quadratic

Fig. 1. General view of the Pareizgupis viaduct

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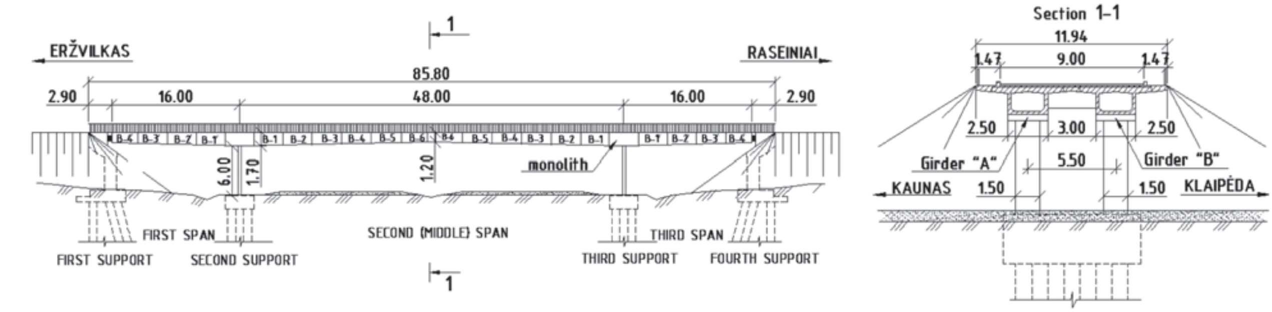

Fig. 2. The Pareizgupis viaduct plan, segment numeration and cross section, in meters

parabola and is equal to 170 cm at the support and 120 cm in the middle of the span. In viaduct cross-section, there are two box girders joined together by a monolithic reinforced concrete slab and transverse beams (diaphragms) at the supports and in the middle of the span. Frame beams are assembled from the segments of different length (16–19 t in weight).

The viaduct piers are the columns of oval cross-section with 38 cm in thickness and 150 cm in width. Thin supports were designed in order to get relatively low bending moments in them resulting from the temperature and plastic deformations of a span construction. The prestressed wire bunch consists of 24 high-strength wires. There are 24 prestressed high-strength wire bunches at the bottom of the span in one box girder and 40 at the top of the support. The Pareizgupis viaduct plan and segment numeration are presented in Fig. 2.

2. Defects and damages of bridges and viaducts

The defects occurring during service life of bridges have a significant effect on the durability of reinforced concrete structures. The major and most important defects occurring in prestressed concrete bridges are their deck cracking (Gribniak *et al.* 2007; Kaklauskas *et al.* 2008; Muttoni, Ruiz 2007, 2010) and the opened shear and flexural cracks. The majority of reinforced concrete bridges built in Europe and the USA have defects. According to the American scientist Polodny (1985) cracking in prestressed reinforced concrete bridges can be produced: a) during design, b) by excess permanent loads, c) by secondary stresses and overloads, d) associated with bridge operation. Dynamic loads and overloads cause secondary stresses and large shear stresses later producing cracking in oblique sections (Wang 2005). In segmental prestressed concrete bridges with the glued joints of girder segments, cracks generally occur near segment joints and the places where wire bunches are corroded (Darmawan 2009; Darmawan, Stewart 2007; Kamaitis 2008; Liang, Wu 2001; Moon *et al.* 2005; Polodny 1985).

The most frequent defects of prestressed concrete bridges in Lithuania are thoroughly examined and presented in the books of Jokūbaitis and Kamaitis (2005). The causes of shear cracks forming are analyzed by Kamaitis (1996, 2000, 2002).

The main defects observed in prestressed concrete viaducts are the following: leakage in expansion joints over mobile and fixed hinges, a rough, rolling and deteriorated pavement of a carriageway, leakage in expansion joints of footpaths, the lack of or inadequate viaduct drainage system, inadequate (leaky) waterproofing on the top of deck beams, inadequate erection of segment joints resulting in leakage (Fig. 3), the bottom wires of girders affected by corrosion and broken in several viaducts, the webs of girder segments with shear cracks on the outside and from the inside (Fig. 4), the segments of a central span with vertical cracks on the outside (red cracks) and shear cracks in the inside (green cracks) (Fig. 5). The forming of cracks in the Pareizgupis viaduct "B" girder is presented in Fig. 5.

The Pareizgupis viaduct has been constructed over the A1 highway carrying the heaviest traffic flow. Since the viaduct has been constructed, the traffic flows increased significantly. Also, the weights of heavy vehicles have changed, environmental factors such as CO_2, Cl and SO_2 affect the concrete intensively (Rombach, Specker 2000) and cause the corrosion of wires. When carrying out the

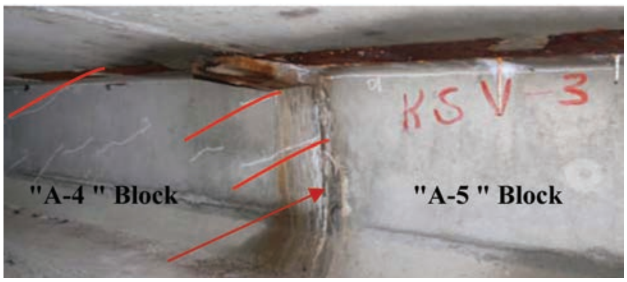

Fig. 3. Leakage in the segment joint between blocks 4 and 5

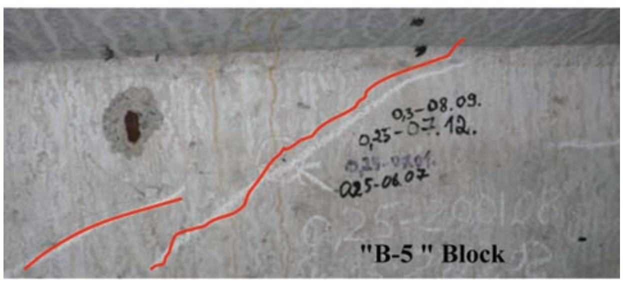

Fig. 4. Shear crack in the internal web of a box girder

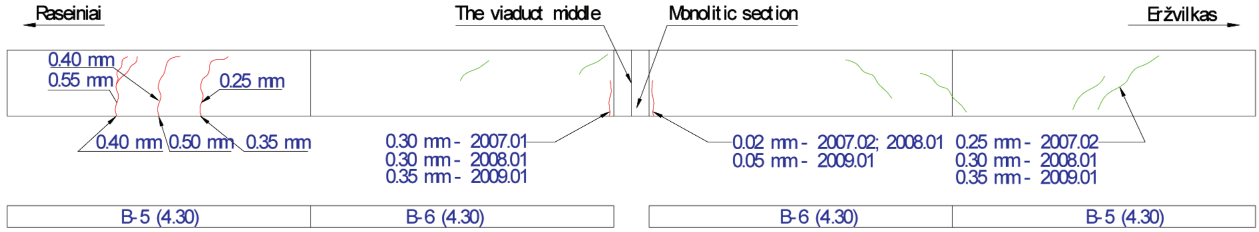

Fig. 5. Girder "B"of the Pareizgupis viaduct in Klaipėda direction. The opened vertical and shear cracks

detailed inspection of the Pareizgupis viaduct, it was found that dangerous flexural cracks opened in the 5th segment of girder "B" of the mid-span (Fig. 5, B-5 block). Having measured the crack width in different places, it was determined that the crack varied from 0.25 mm to 0.50 mm in height. The max crack width (0.50 mm) was recorded at the bottom of the fifth segment flange. This crack occurs along the entire bottom of the box. Having inspected the fifth segment wires of girder "B" of the mid-span, 48 fully broken wires (2 wire bunches) were found, another two wire bunches were greatly subject to corrosion. The cross-section of wires decreased from 5 mm to 3 mm in the corrosive parts (Fig. 6). Additionally, around 20 wires were released but were not broken. After the visual evaluation and summing up of all the deteriorated wires, it was determined that from 60 to 80 wires were detrimental to the performance of the viaduct, which constituted about 3–4 wire bunches.

As a result of corrosion, the cross section area of prestressed wires and the strength of concrete decrease and the losses of prestress increase.

3. Design data of the viaduct

The Pareizgupis viaduct design was produced in 1977, according to the *Russian design code CH 200-62 "Techniceskie usloviya projektirovaniya zeleznodopoznych avtodopoznych i gorodskich mostov i trub"* (1962). The design loads of H–30 and HK–80 were chosen. The H–30 design load consisted of two automobile queues (equivalent load of one automobile queue $\lambda = 1.76$ t/m) located in two traffic lanes and the load of a crowd of people on the viaduct footpaths ($\lambda = 0.4$ t/m). Reliability factor of the load $\gamma_f = 1.4$.

Dynamic factor is calculated by formula (*CH 200-62*):

$$\mu_{din} = 1 + \frac{(45 - \lambda)}{135}, \tag{1}$$

where λ – span length, m.

Dynamic factor for the H–30 load in the mid-span is $\mu_{din} = 1.0$. The HK–80 load is a wheel four-axle vehicular load. The weight of each axle is 20 t. Reliability factor of the wheel load – $\gamma_f = 1.1$ and dynamic factor – $\mu_{din} = 1.0$.

The viaduct deck segments have been designed from M400 mark concrete which, according to *LST EN 206-1:2002 "Concrete – Part 1: Specification, Performance, Production and Conformity"*, conformed to C30/37 concrete class. According to the *Russian design code CH 365-67 "Ukazaniya po projektirovaniju zeleznobetonych i betonych konstrukciji mostov i trub"* (1967) theoretical concrete compressive strength is 14 MPa and concrete tensile strength is 2.0 MPa.

Fig. 6. The view of wires in the places of crack opening

For the calculation of the viaduct deck effects, the computer program "Midas Civil" was used to simulate the viaduct girder model. Having evaluated the self weight, the permanent load and design loads, bending moments were calculated. Characteristic bending moment in the span middle resulting from the HK–80 load together with the self weight and permanent load is 8.48 MNm and characteristic bending moment from the H–30 load together with the self weight and permanent load is 8.59 MNm.

In the central part of the viaduct, there are 576 wires stressed by the initial stresses of 1100 MPa. Having evaluated prestress losses, the cracking moment of 8.69 MNm calculated according to the requirements of the *Design Code CH 365-67* was found. Cracking moment is higher than the moment resulting from design loads, thus there have to be no cracks at normal and oblique sections of the mid-span.

4. Analysis of the main concrete tensile and compressive stresses

As can be seen in Fig. 5, shear cracks are also opened in the central part of the viaduct (blocks 6), therefore the aim was set to determine the main concrete stresses in the mid-span. The same problem was also analyzed in the article of

Plos and Gylltoft (2006). Moreover, in order to evaluate the potential effect of cross-sectional reduction on the results, the main concrete stresses of the viaduct mid-section deck (resulting from design loads) were calculated according to two methods: the requirements of the *Design Code CH 365-67* and the FEM.

Using the FEM, the Pareizgupis viaduct was simulated with regard to its real dimensions. Also, the top and bottom wire bunches were located precisely as was indicated in the viaduct design. The viaduct box girder cross-sectional model is presented in Fig. 7.

Fig. 7. The Pareizgupis viaduct simulated according to the FEM

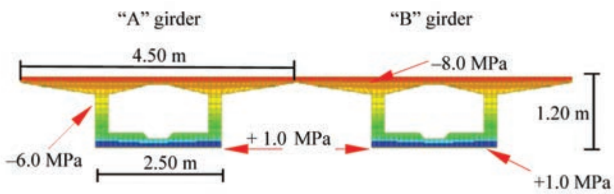

Fig. 8. Distribution of the main stresses in the middle section of the mid-span under the symmetrical design H–30 load and the total permanent loads

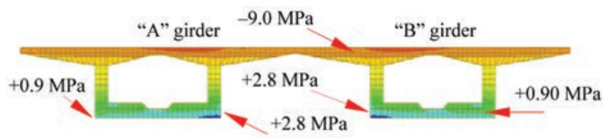

Fig. 9. Distribution of the main stresses in the middle section of the mid-span under the symmetrical design HK–80 load and the total permanent loads

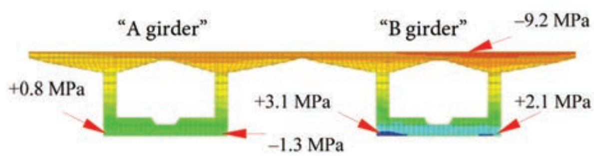

Fig. 10. Distribution of main stresses in the middle section of the mid-span under the asymmetric design HK–80 load and the total permanent loads

According to the *Design Code CH 365-67*, the main concrete stresses may be calculated as follows:

$$\sigma = \frac{1}{2}(\sigma_x + \sigma_y) \pm \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2}\begin{Bmatrix} \le +2.0 \text{ MPa} \\ \ge -14 \text{ MPa} \end{Bmatrix}, \quad (2)$$

where σ_x– stresses in the direction of x axis; σ_y– stresses in the direction of y axis; τ– tangential stresses.

Having calculated, according to the above mentioned code, the main stresses of reduced cross-section in the middle of the bottom flange, the middle of the top flange and the web middle of the central part, it was determined that the main compression stresses did not exceed 14 MPa and the main tensile stresses did not exceed 2 MPa. The main concrete tensile and compressive stresses resulting from the design H–30 load and the total permanent loads, calculated according to the FEM, are presented in Fig. 8, and the stresses resulting from the design HK–80 applied on the middle part of cross section and the total permanent loads are presented in Fig. 9.

In the case of asymmetric loading, the main highest tensile and compressive stresses are formed in the girder on which the design HK–80 load is applied. The distribution of the main stresses in this type of loading is presented in Fig. 10.

Under asymmetric HK–80 load, compressive stresses are formed in the bottom flange, the top flange and the web of girder "A", when load is applied on girder "B". Tensile stresses are formed in the bottom flange and the junction of the bottom flange and the web of girder "B"; compressive stresses are formed in the top flange and the junction of the top flange and the web. The main tensile stresses of the bottom flange of girder "B" are equal to 1.9 MPa in the flange middle and the tensile stresses of 2.1 MPa and 3.1 MPa are formed near the edges of the flange. Having performed the analysis of the main stresses under symmetrical and asymmetric loads, the conclusion can be made that under asymmetric loads higher tensile stresses are formed, the occurrence of which may cause the cracking of concrete, especially if the wire corrosion is estimated.

In order to evaluate differences of the main stresses of the real box section and the reduced I-section and considering that it is not possible to calculate the main stresses of asymmetrically loaded elements according to the

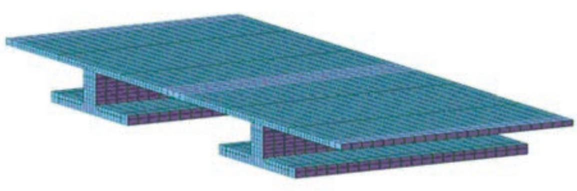

Fig. 11. The segment of girders "A" and "B" of the reduced cross-section of simulated Pareizgupis viaduct

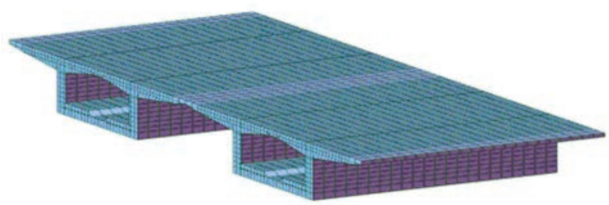

Fig. 12. The segment of girders "A" and "B" of the simulated Pareizgupis viaduct

codes, the reduced I-section (Fig. 11) and the box section (Fig. 12) of the Pareizgupis viaduct were simulated.

Both cross-sections were calculated from the load of the same combination: the self girder weight, decking weight, design H–30 load and precompresion load.

The distribution curves of both simulated cross-sections main stresses according to distance from viaduct middle were completed. The main stresses of these models in the junction of the web and the top flange are presented in Fig. 13, in the middle of web – in Fig. 14, in the junction of the web and the bottom flange – in Fig. 15, and in the bottom of the bottom flange – in Fig. 16.

It can be seen in the presented diagrams that the distribution of the main stresses is different at the same point in the box section and I-section under the same load. The largest difference in compressive stresses is noticeable in the top flange. The major disagreement of the main stresses occurs in the junction of the web and bottom flange and the bottom of the bottom flange. The main compressive stresses dominate across all the lenght in I-section of the web middle and in the box section the main stresses change from compressive stresses to tensile stresses.

Since some prestressed reinforcement was found broken in the inspected viaduct, the main concrete tensile stresses were calculated according to the FEM having accepted the assumption that two wire bunches were broken:

- Under the symmetrical H–30 and HK–80 load, tensile stresses of 2.0 MPa are formed in the bottom flange of girder „A“ and tensile stresses of 2.4 MPa are formed in the bottom flange of girder „B“;
- Under the asymmetrical HK–80 load, tensile stresses of 1.5 MPa are formed in the bottom flange of girder „A“ and tensile stresses of 2.9 MPa are formed in the bottom flange of girder „B“.

5. Calculation of reinforcement stress according to crack width

In order to find out the reinforcement stress increment because of broken wires in the Pareizgupis viaduct (where flexural cracks are the largest and reach 0.35 mm in the middle of span), it was tried to use a flexural crack width. The crack width was calculated according to the *Lithuanian Construction Technical Regulation STR 2.05.05:2005 “Design of Concrete and Reinforced Concrete Structures”* valid in Lithuania and the requirements of *ENV 1992-2:1996 Eurocode 2: Design of Concrete Structures – Part 2: Concrete Bridges.* According to the code valid in Lithuania, the crack width is calculated as follows:

$$w_k = \delta\varphi_l\eta\frac{\sigma_s}{E_s}20(3.5-100\rho_1)\sqrt[3]{\phi_s}, \qquad (3)$$

where δ, φ_l, η – coefficients; σ_s – stresses in wire bunches, MPa; E_s – the modulus of elasticity of wires, MPa; ρ_1 – reinforcement ratio of element cross-section; ϕ_s – wire bunch diameter, m.

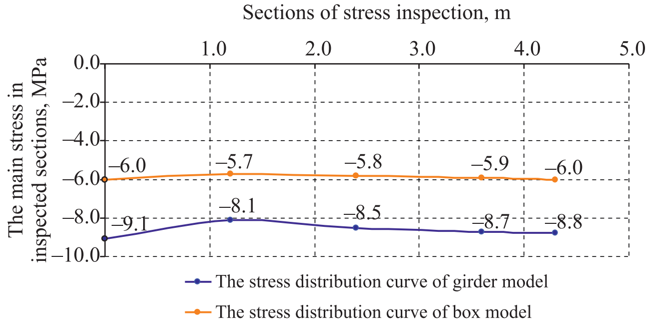

Fig. 13. Distribution curves of the main stresses of the girder and box models in the junction of the web and the top flange

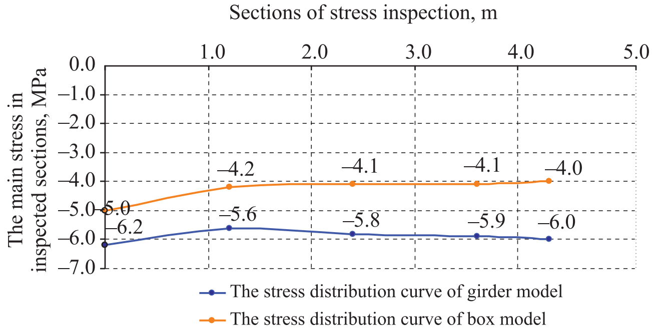

Fig. 14. Distribution curves of the main stresses of girder and box models in the web middle

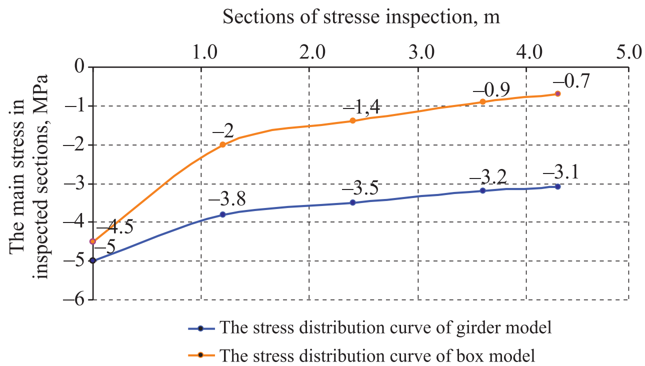

Fig. 15. Distribution curves of the main stresses of girder and box models in the junction of the web and the bottom flange

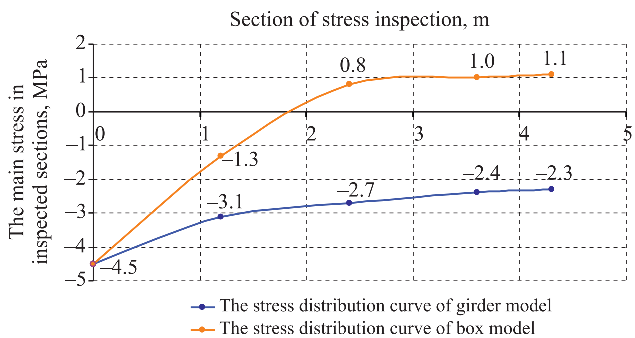

Fig. 16. Distribution curves of the main stresses of girder and box models in the bottom of the bottom flange

According to the requirements of *ENV 1992-2:1996* the crack width is calculated as follows:

$$w_k = \frac{\phi_s(\sigma_s - 0.4\dfrac{f_{ctm}}{\rho_{p,eff}}(1+\alpha_e\rho_{p,eff})}{3.6\rho_{p,eff}E_s}, \qquad (4)$$

where ϕ_s – wire bunch diameter, m; $\rho_{p,eff}$ – reinforcement ratio; f_{ctm} – the average concrete tensile strength, MPa.

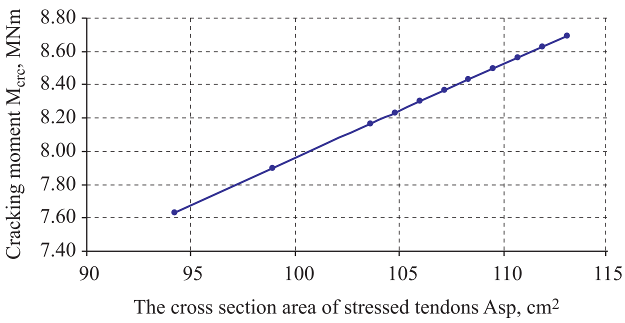

Fig. 17. The dependence of cracking moment on the cross-sectional area of prestressed wires after the evaluation of their breaks (according to *STR 2.05.05:2005*)

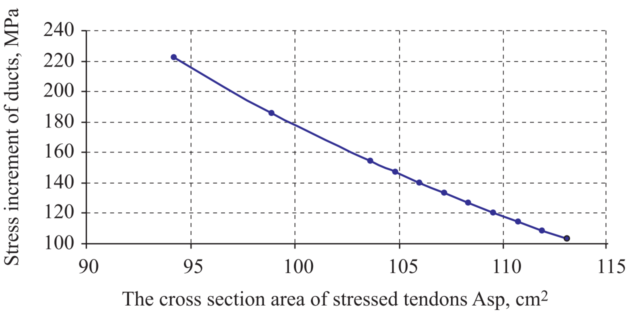

Fig. 18. The dependence curve of stresses in wire bunches on the cross-sectional area of wires after the evaluation of their breaks (according to *STR 2.05.05:2005*)

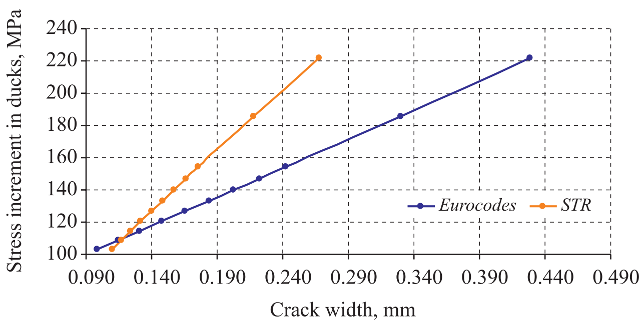

Fig. 19. The curves of dependence of the stresses in wire bunches on the crack width calculated according to the methods of *STR 2.05.05:2005* and *ENV 1992-2:1996* after the evaluation of wires breaks

The stresses in wire bunches were calculated by *STR 2.05.05:2005* requirements because *ENV 1992-2:1996* does not present such calculations.

Firstly, the crack width, cracking moment and stresses in wire bunches were calculated for all wires, then up to four wire bunches were broken and it was determined how the calculated parameters changed. The dependence of parameters on the cross-sectional area of stressed wires is presented in Figs 17 and 18.

It can be seen in Fig. 17 that flexural cracks are likely to open when nearly 8 wires were broken, whereas during the visual inspection it was determined that 48 wires (2 wire bunches) were already fully broken and others were damaged by corrosion or released (Fig. 6).

The variation in crack width, calculated according to the methods of the *STR 2.05.05:2005* and *ENV 1992-2:1996* with regard to the increment of stresses in wire bunches is presented in Fig. 19.

According to *STR 2.05.05:2005* the real crack (0.35 mm) will open when reinforcement stress increment will be ~280 MPa. From the stress increment equation *(STR 2.05.05:2005)*, such value (external bending moment is equal to 8.59 MNm, chapter 3) is obtained if cross sectional area of prestressed reinforcement decreases by ~5.2 wire bunches. According to *ENV 1992-2:1996,* at the real crack width the reinforcement stress increment should be ~195 MPa. From the same equation *(STR 2.05.05:2005)*, the obtained decrease of prestressed reinforcement is ~3.2 wire bunches.

Having compared the widths of flexural cracks calculated by the above mentioned methods, it was noticed that w_k *(STR 2.05.05:2005)* and w_k *(ENV 1992-2:1996)* calculating values differ up to 1.4 times, whereas the calculated stresses σ_s *(STR 2.05.05:2005)* and σ_s *(ENV 1992-2:1996)* are not different. Jokūbaitis and Juknevičius (2009), when analyzing the calculation of flexural crack widths of prestressed reinforced concrete beams according to the methods of *STR 2.05.05:2005* and *ENV 1992-2:1996,* obtained similar results as in our calculations.

6. Forecasting of bridge deflections

Due to of various defects, irregular cracks are formed and developed in bridges and the deflection of bridge increases. Based on these parameters (which depend on defects), it is convenient to evaluate the present viaduct status and at the same time the reliability of a viaduct. It is not easy to accurately determine the widths of cracks opened in a bridge and their development because the crack width varies across its height.

Therefore, in order to evaluate bridge reliability, it is more convenient to apply the parameter of bridge deflection which is easier to determine and the determined values are more reliable (when new cracks are produced, the development of the first cracks is getting slow, whereas bridge deflection is getting higher in this case).

According to the viaduct deflection data of 13 inspection years, the Pareizgupis viaduct has deflected most intensively from all the unreinforced viaducts. The deflection variation curves of the four viaducts during the inspection period and the flows of heavy traffic on each viaduct per day were presented for the analysis (Fig. 20). It can be seen in Fig. 20 that viaduct deflections depend on its transport flow.

Therefore, it is important to evaluate this fact in the analysis of bridge behaviour. In order to determine the reliability of reinforced concrete box girder bridges, three viaducts were chosen in which the flow of moving heavy transport varied from 285 to 332 vpd (Fig. 20). For the evaluation of reliability, structural sustainability equations were chosen (Kudzys *et al.* 1992). The probability that ultimate deflection will be reached is calculated according to the following expression:

$$Q = 1 - F\left[\frac{EY - Ey}{\sigma^2 Y + \sigma^2 y}\right], \quad (5)$$

where EY, Ey, $\sigma^2 Y$, $\sigma^2 y$, – the average values of parameters Y and y and their dispersion.

Probability index of structural sustainability:

$$P = \prod_{i=1}^{k=1} \exp(-nQ), \quad (6)$$

where k – the number of element types (k = 1); n – the number of analyzed deflections (n = 3).

Having evaluated the average values of results (Fig. 20), it was obtained that the increment of deflection $v_{f,m}$=1.015 mm/year, its dispersion $\sigma^2 v_f$ = 0.204 mm^2/(year)^2. The average critical deflection assumed f_{crit} = 25 mm and distribution dispersion $\sigma^2 f_{crit}$ = 1.4 mm^2. Having calculated the dispersion of deflection, it was obtained $\sigma^2 f$ = 1.259 mm^2.

Taking the initial deflection equal to its average value at the end of inspection (2008) f = 11.67 mm, the probability and reliability values, calculated according to expressions (5) and (6), are presented in Table 1.

Table 1. Probability and sustainability indexes of the analyzed viaducts

Operation forecasting time from the chosen moment, year	Probability that bridge deflection will reach critical value Q	Sustainability index, *P*
2	0	1
4	6.00E-06	0.99998
6	0.00318	0.99905
8	0.0118	0.96522
9	0.0478	0.86641
10	0.1416	0.6539
12	0.617	0.15708
13	0.9522	0.05746

Taking that viaduct sustainability has to be not lower than 95%, it was obtained that the critical viaduct deflection will be reached in less than 9 years. In this case the critical deflection is chosen conditionally because it is difficult to determine deflection before the inspection (observations started about 20 years after the beginning of operation) and the initial bridge camber. The aim of this calculation was to show that it is not difficult to forecast the residual bridge operating time using the probability method. However, using this method it is more difficult to evaluate concrete creep which has a direct effect on bridge deflection and is the most evident during the first several years.

In order to evaluate the potential effect of creep, which is damped in the course of time, on the deflections of analyzed viaducts, creep coefficient and modulus of deformation of M400 mark concrete (design based concrete of the viaduct deck) with regard to its composition were calculated according to the method proposed by Bažant and Baweja (1995). The concrete composition is the following: cement – 425 kg/m^3, sand – 820 kg/m^3, chrushed granit – 975 kg/m^3, water and cement ratio – 0.40. The obtained variation curves of creep coefficient and deformation modulus for the period of 30 years are presented in Figs 21 and 22.

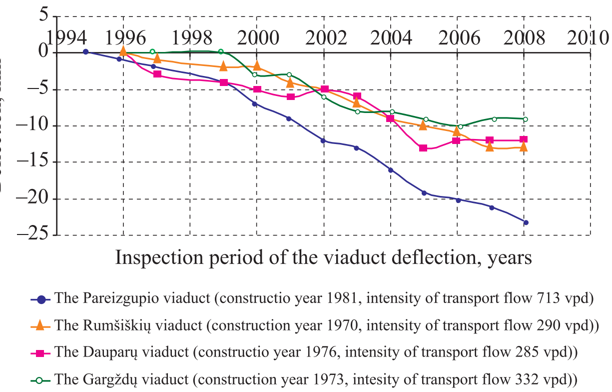

Fig. 20. Deflection variation curves of the viaducts during the inspection period

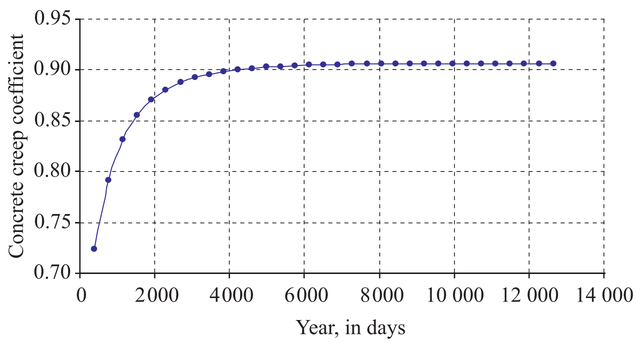

Fig. 21. The variation curve of creep coefficient

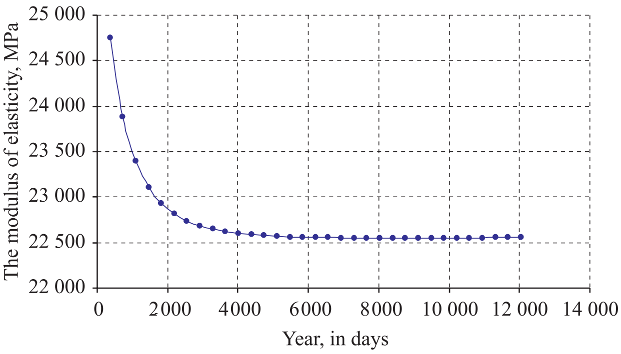

Fig. 22. The variation curve of deformation modulus

Having calculated, according to the method proposed by Bažant and Baweja (1995), deformation modulus for the M400 mark concrete composition, it was obtained that deformation modulus decreased by 30% in 21 years due to concrete creep, when design based elasticity modulus was equal to 32 000 MPa. The most intensive variations in deformation modulus occurred in the first three years, and then variation stabilized in 14 years and changed only marginally. Since the analyzed viaducts were started to be inspected after 20 years, the effect of creep on deflection was minimal.

According to the FEM, the deflection of the Pareizgupis viaduct from the design (HK-80, total permanent and precompression) loads was theoretically calculated with the software "Midas Civil", evaluating the concrete deformation modulus but not taking into account the cracking. The design based viaduct deflection from the short time load was equal to 14.98 mm and, after the evaluation of concrete creep, deflection increased up to 21.84 mm. 34 years have passed since the beginning of the Pareizgupis viaduct construction. The viaduct deflection has been inspected since 1995. During 13 years of observation the viaduct deflected by 24 mm. Having evaluated, according to the FEM, the concrete creep at this period and the factor that about 4 wire bunches out of 24 of girder "B" were detrimental to the performance, the average viaduct deflection value of 25.03 mm was obtained.

7. Conclusions

Having performed the analysis of the main stresses of the Pareizgupis viaduct with regard to the effect of symmetrical and asymmetrical loads, it was found that under asymmetrical loads much higher tensile stresses (~30%) and compressive stresses (~13%) are formed at the sides of box girder than under symmetrical loads.

The main stresses of the viaduct box section and reduced I-section calculated according to the FEM were obtained quite different. The main compressive stresses under asymmetrical loads in the box section top flange are ~32% higher and tensile stresses in the bottom flange are ~52% higher than in the reduced I-section.

Having estimated, after the visual inspection, that ~4 wire bunches are broken (~17% of prestress reinforcement) the increase of 9.6% in reinforcement stress (including a prestress of 1100 MPa) was calculated according to the FEM.

Due to relatively small losses of reinforcement area (up to 20% in the discussed case), the relation of losses with the crack moment values, the increase of reinforcement stress and the crack width is close to a line. In such case, it is not difficult to estimate the possible losses of reinforcement area according to the crack width; however, the width varies not uniformly in time and for this reason it does not reflect the increase of defects directly.

It is proposed to estimate the reliability of the examined viaduct by its deflection with regard to variation of parameters in time. When evaluating reliability parameters in the discussed case (such as the velocity of deflection increase, its dispersion, etc.) it is necessary to take the volume of traffic flows and the possible pavement roughness into consideration.

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