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INVESTIGATION OF CONCRETE CRACKING EFFECT IN DECK SLAB OF CONTINUOUS BRIDGES

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Abstract. Analysis of the literature has shown that two main factors, which should be taking into consideration in designing of continuous bridges, are the influence of cracking on serviceability of the structures and inelastic behaviour of structural elements. Crack control is often the governing design criterion for the choice of the amount of longitudinal pre-stressing of reinforced concrete bridges as well as for the construction sequence of composite bridges. Present paper investigates cracking behaviour of reinforced concrete decks of continuous bridges over intermediate supports. It should be noted that this section behaves almost as pure tension ties. The paper presents results of complex investigation which has included experimental and numerical analyses of deformations and cracking resistance of reinforced concrete tensile members and a deck slab. Different techniques on crack controlling were analysed. Analysis was showed that the techniques worth predicting deformational behaviour of reinforced concrete members as well as assuring the cracking control in the bridge deck.

Keywords: tensile members, reinforced concrete (RC), bridge deck slab, cracking, and experimental investigation.

1. Introduction

Civil engineering structures, especially bridges, are omnipresent in every society, regardless of culture, religion, geographical location and economical development. The rapid pace of life in nowadays demands transportation infrastructure which is reliable, efficient, and safe. As traffic volumes continue to increase, the performance and durability of existing roads and bridges are challenged and scrutinised on a daily basis. When problems arise, it is important that the department of transportation have the ability to perform the necessary maintenance or construction in a timely and effective manner.

For last forty years, a number of studies have been performed to investigate deformational behaviour and cracking resistance of bridge structures (Gustafson, Wright 1968; Juozapaitis *et al.* 2006, 2010; Kudzys, Kliukas 2008a; 2008b; Mari, Montaner 2000; Moffatt, Lim 1976; Moffatt, Dowling 1979; Parsekian *et al.* 2009; Ryu *et al.* 2004; Shim *et al.* 2000; 2001; Shim, Chang 2003; Takács 2002; Yousif *et al.* 1995). Analysis has shown that two main factors, which should be taken into consideration in designing of continuous bridges, are the influence of cracking on serviceability of the structures and inelastic behaviour of structural elements. The authors (Gribniak *et al.* 2007; 2008) have shown that one of major factors affecting cracking of reinforced concrete (RC) bridge deck is the restrained shrinkage of concrete.

Present paper investigates cracking behaviour of RC decks of continuous bridges over intermediate supports. Crack control is often the governing design criterion for the choice of the amount of longitudinal pre-stressing of RC bridges as well as for the construction sequence of composite bridges. It should be noted that a deck slab over intermediate supports behaves almost as pure tension ties in the longitudinal direction. This fact allows numerical modelling of the deck based on test data of RC tensile members. The paper presents results of complex investigation, which has included experimental and numerical analyses of deformations and cracking resistance of RC tensile members and a deck slab.

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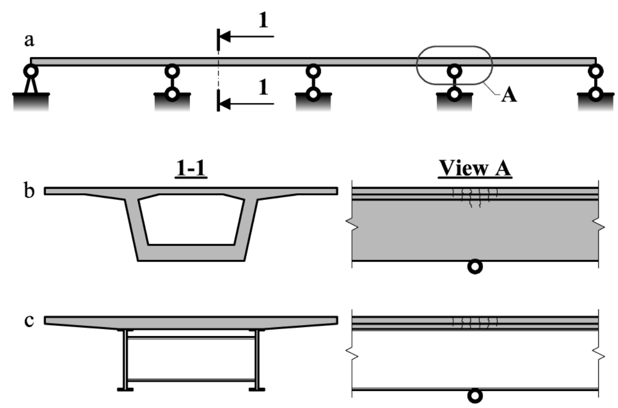

Fig. 1. Sketch of a continuous bridge (a); cracking pattern of concrete (b) and composite (c) sections

2. Modelling of behaviour of bridge deck

Fig. 1 presents the structural scheme and cracking behaviour of a typical continuous bridge. Cross-section of the deck slabs over intermediate supports can be assumed as a tie with a flange subjected to pure tension. Such idealisation is adequate under assumption that the strain is constant over the depth of the flange and neglecting the effect of local moments due to traffic loads.

Present study deals with cracking behaviour of RC decks of continuous bridges, aiming at control of cracks in the bridge sections over intermediate supports. Three analyses have been performed using: 1) finite element (FE) software ATENA; 2) layer section model and 3) semi-analytical method, proposed by Muttoni and Ruiz (2007). It has been assumed that the deck slab was subjected to pure tension.

2.1. Numerical modelling by ATENA

A plane (2D) FE model (FEM) was considered. Such approach simplifies behaviour of real structures on one hand, but enables a refinement with respect to the model based on the plane section hypothesis on the other hand.

Non-linear material model, based on the concept of smeared cracks and damage, was assumed for concrete. Un-cracked concrete is considered as isotropic. After cracking it is assumed being orthotropic. In this study, the fixed crack model was used: crack direction and material axes are defined by the principal stress direction at the onset of cracking when the principal stress exceeds the tensile strength. In further analysis, this direction is fixed and cannot be changed (Červenka *et al.* 1998).

2.2. Layer section model

Layer section model is a universal approach which has been extensively used by the authors solving various structural problems (Kaklauskas 2004; Juozapaitis *et al.* 2006; Kaklauskas *et al.* 2007; 2008; Gribniak *et al.* 2008). The calculation is based on formulae of strength of materials extended to application of Layer section model and material diagrams. The following assumptions have been adopted:

- average strain, also called as smeared crack, concept;
- linear strain distribution within the depth of the section;
- perfect bond between layers.

2.3. Analytical method

Muttoni and Ruiz (2007) have been proposed a method for cracking and deformational analysis of tensile RC members. In this method, three different stages (Fig. 2) may be considered for deformational behaviour of RC member: 1) an uncracked stage until concrete reaches its effective tensile strength; 2) the cracking stage when all cracks appear (in load-deformation diagram approximated by a horizontal line); 3) once the number of cracks is stabilized, the last phase controls the response of the tie in which the number of cracks remains constant but their openings increase with load. The third phase ends with the yielding of the reinforcement. This method applies a stress-bond slip law is based on following assumptions:

- discrete crack concept, i.e. each of cracks is traced individually;
- rigid-plastic bond law over the transfer length;
- for crack width analysis strains of tensile concrete are neglected. In deformation analysis (Section 4.3) they are taken into account.

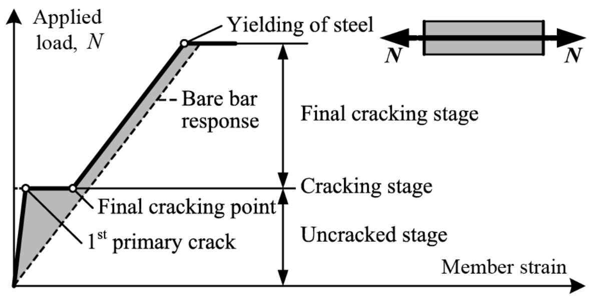

Fig. 2. Idealised loading-deformation response of RC member

3. Tensile tests

The experimental investigation aimed at cracking behaviour of tensile concrete and RC members has been conducting. The tests were performed in the *Laboratory of Dept of Reinforced Concrete and Masonry Structures of VGTU* in 1993.

3.1. Description of test specimens

The specimens subjected to direct tension were nominally 660 mm long. The central part 350 mm in length was a square 60×60 mm section, whereas the ends were widened. The experimental programme has comprised of two series of specimens. Each of the series was consisted of eight specimens of three types. The specimens of the first type were un-reinforced. The specimens of second type were reinforced by one ∅5 mm cold worked wire, whereas the members of the third type had four ∅5 mm wires. The respective specimens had reinforcement ratio 0.54 and 2.18%. Present study deals with the reinforced specimens.

3.2. Production of the specimens and material properties

Each series of tension specimens were cast from one batch into steel formwork. Test specimens such as 100 mm concrete cubes and 100×100×400 mm prisms were also produced. The experimental specimens were cured under the laboratory conditions at average relative humidity 65% and average temperature 20 °C. The age of the tensile specimens at testing was 180 days.

Concrete mix proportion, presented in Table 1, was taken to be uniform for all experimental specimens at each series. The ordinary portland cement and crushed granite aggregate were used. Water/cement and aggregate/cement ratios by weight were taken as 0.47; 3.13 and 0.61; 5.27 for first and second series, respectively.

Table 1. Mix proportion of the experimental specimens, kg/m^3

Material	Series 1	Series 2
Sand 0/2.5 mm	640	650
Crushed granite aggregate 5/35.5 mm	1046	1042
Cement CEM I 42.5 N	320	540
Water	195	255

In order to determine physical and mechanical properties of concrete, listed in Table 2, twelve 100 mm cubes and fifteen 100×100×400 mm prisms were tested. Compressive strength and deformation tests were performed at test day and at 28 days after casting. Three cubes and three prisms were tested at each age. The latter specimens were also used for determining the Young's modulus and free shrinkage strain of concrete.

Table 2. Physical and mechanical properties of concrete

Parameter	Series 1	Series 2
Compressive strength at 28 day	23.90 MPa	30.72 MPa
Compressive strength at test	24.10 MPa	32.49 MPa
Young's modulus at test	24.72 GPa	30.12 GPa
Free shrinkage strain at test, ε_{cs}	-244×100^{-6}	-256×100^{-6}

Cold worked wire ∅5mm was used as reinforcement. Three samples were tested and several lengths were weighed to check the nominal size. The stresses and modulus of elasticity are based on nominal diameters. The 0.2% proof yield stress and modulus of elasticity of the reinforcement were 527 MPa and 170 GPa, respectively.

3.3. Experimental set-up of test specimens

The experimental set-up of a test specimen is shown in Fig. 3. The test specimens were loaded with a 50 kN electro-mechanic test machine, having a stiff frame. The tests were loading controlled, with a velocity of 0.003–0.07 mm/min. Concrete surface strains were measured on all sides of the test specimens by means of 51.5 mm strain gauges glued at eight different levels of the tensile zone of constant cross-section. All measurement equipments were connected to a personal computer to acquire data and record the failure tensile load and load–displacement diagrams.

Fig. 3. Experimental set-up of a test specimen

4. Verification of the models

This Section reports results of verification of the modelling techniques discussed in Section 2. In present analysis two RC members 1-B7 and 2-B8 from both batches, having moderate reinforced ratio p = 2.18%, were used. Agreement between the modelled load-deformation behaviour of tensile RC members and the experimental results has been analysed. Model parameters for each of the techniques are observed below.

4.1. Finite Element Model

Fig. 4 shows geometrical parameters of the test member and shows its FEM. Due to symmetry conditions, only quarter of the specimen was modelled. It should be noted that shrinkage deformations of concrete (Table 1) was included into model as separate factor.

As known, the crack pattern can be predicted most realistically by using finest FE mesh size, thus in the applied model it was assumed 2.5 mm. In the previous study, the authors were obtained that FE simulation results, using ATENA, are mesh-dependent (Gribniak *et al.* 2010). To reduce such dependence, a scaling technique was proposed. Furthermore, it was shown that FE size h = 50 mm can be used as a reference for adequate modelling of RC

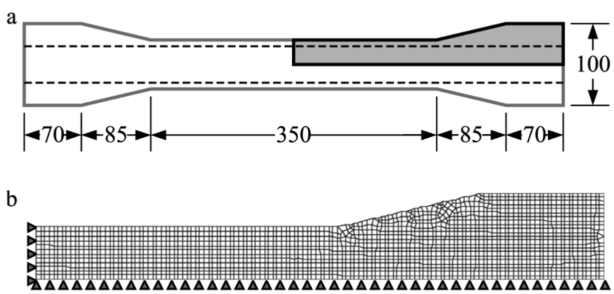

Fig. 4. Geometrical parameters of the test specimen (a) and FE mesh of the model (b)

bridge elements. Therefore, tension-stiffening relationship, which was applied in the present study, has been scaled avoiding extra stiffness of FEM. In this analysis a linear tension-stiffening model based on fracture mechanics approach, shown in Fig. 5a, was used. The fracture energy was defined as

$$G_F = 2.5\times10^{-5} \times f_{ctm}, \qquad (1)$$

where f_{ctm} – the tensile strength of concrete, calculated by *EN 1992-1-1:2004 Eurocode 2: Design of Concrete Structures – Part 1: General Rules and Rules for Buildings,* using compressive strength of concrete (Table 1). The above equation is unit-dependent, the fracture energy G_F is in MN/m and the tensile strength is in MPa. The scaling was performed multiplying G_F by factor δ (Gribniak *et al.* 2010):

$$\delta_{(1)\to(2)} = \sqrt{\frac{h(2)}{h(1)}} = \sqrt{\frac{2.5}{50}} \approx 0.224, \qquad (2)$$

where $h(1)$ and $h(2)$ – the reference and the actual sizes of the FE mesh, respectively.

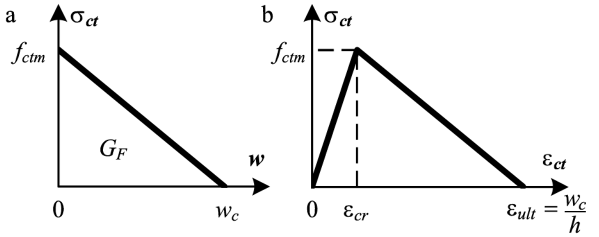

Fig. 5. Tension-stiffening models based on stress-crack width (a) and average stress-strain (b) relationships

4.2. Layer section model

Layer section analysis of shrunk RC tensile members was discussed in (Kaklauskas *et al.* 2009) and is not presented here in details. Only two points of the analysis are mentioned. First, the average stress-strain model (analogue to that was applied in ATENA), shown in Fig. 5b, was applied in the Layer model. The ultimate strain was calculated using following equation (Gribniak *et al.* 2010):

$$\varepsilon_{ult} = \frac{2G_F}{f_{ctm}\,h}\delta_{(1)\to(2)} = \frac{5\times10^{-5}}{0.0025}\sqrt{\frac{2.5}{50}} \approx 4.47\times10^{-3}. \qquad (3)$$

Second, shrinkage effect was taken into account by introducing fictitious axial force

$$N_{cs} = \varepsilon_{cs} E_c A_c, \qquad (4)$$

where E_c and A_c – the effective modulus of elasticity of concrete and the area of concrete net section, respectively.

4.3. Analytical method

Present analysis based on the method, proposed by Muttoni and Ruiz (2007) and slightly modified by the authors. Deformation of the tensile RC member is calculated as following. The average strain at the first stage (Section 2.3), elastic behaviour can be expressed as follows:

$$\varepsilon_{sm} = \frac{N}{(A_c E_c + A_s E_s)}, \qquad (5)$$

where N – the applied load; E_c and E_s – the modulus of elasticity of concrete and steel, respectively.

In the third stage (after formation of final crack), average strain of the member is calculated as follows:

$$\varepsilon_{sm} = 0.5(\varepsilon_{s,cr} + \varepsilon_{s,ts}),$$
$$\varepsilon_{s,cr} = \frac{N}{E_s A_s}, \qquad (6)$$

where $\varepsilon_{s,cr}$ and $\varepsilon_{s,ts}$ – the strain in the crack and in the mid-section between the cracks, respectively; E_s and A_s – the modulus of elasticity and the area of the reinforcement.

Deformation analysis is based on average strains and average distance between cracks $s_{r,m}$. Under the assumption that the distance $s_{r,m} = 1.5 l_{ba}$, strain $\varepsilon_{s,ts}$ can be defined from the following equation:

$$\varepsilon_{s,ts} = \varepsilon_{s,cr} - \frac{3 l_{ba}\tau_a}{d_b E_s},$$
$$l_{ba} = \frac{d_b f_{ctm}}{4\rho\tau_a},$$
$$\tau_a = 2 f_{ctm}, \qquad (7)$$

where l_{ba} – the transfer length; ρ – the reinforcement ratio; f_{ctm} – the tensile strength of concrete; d_b – the bar diameter.

4.4. Results of the analysis

Predicted deformations of the test members are shown in Fig. 6. It can be stated that both numerical methods have demonstrated good accuracy. The semi-analytical method has also modelled behaviour of the test members with acceptable accuracy. It may be concluded that all the methods can be applied for the solving the problems of cracking control in the bridges.

5. Control of crack widths in the bridge deck

This Section presents an example cracking control in the bridge deck. The control is performed by checking the cracking response of the pre-stressed bridge in a section

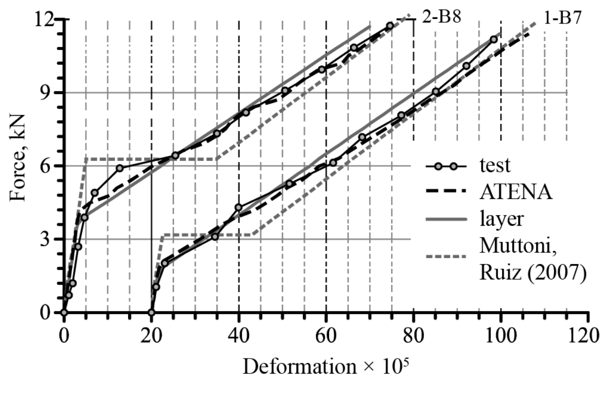

Fig. 6. Predicted deformations of the test members

over intermediate supports. Fig. 7a shows the main dimensions of the cross-section for a typical European continuous box girder bridge with a span between piers of approximately 55 m (Muttoni, Ruiz 2007). This Fig also presents main geometrical characteristics of the section. Other parameters are following: modulus of elasticity of the steel E_s = 205 GPa; tensile strength of concrete f_{ct} = 3.5 MPa (approx corresponding to the cylinder compressive strength f_{cc} = 50 MPa); free shrinkage strain of concrete $\varepsilon_{cs} = -300 \times 10^{-6}$.

This section is subjected to a bending moment M = 56 MNm due to the frequent loads combination. Considering the exposure conditions and the presence of pre-stressing, the crack width limit is adopted as $w_{per,\infty} = 0.2$ mm.

FE software ATENA. FEM has been idealised as console element, assuming geometrical parameters of the section, shown in Fig. 7b. The smeared reinforcement (p = 1%) was used to assure shear strength of the model. Fig. 8a presents FE meshing of the model. The mesh size was 50 mm, thus the scaling (Section 4.1) was not needed.

It should be noted that shrinkage strain of concrete was introduced into the model. The element was subjected to uniformly distributed load q in step-by-step manner increased up to bending moment = 56 MNm. The crack pattern obtained at the maximal load is shown in Fig. 8b. As illustrated in the figure, the analysis has resulted in the maximal crack width $w_{c,\max} = 0.263$ mm which has exceeded the permissible one. Therefore, each loading step was analysed separately obtaining the target load, at which the crack width reached the limit value. The min pre-stress force after losses was calculated from the condition of strain difference in the reinforcement at the max load and the target load, respectively. The calculated pre-stress force is given in Table 3. As shown in Fig. 7c, after pre-stressing crack width at maximal load was very close to the limit value.

Layer section model. The section model of the bridge is presented in Fig. 7b. The technique to determine the pre-stressing force using the Layer model is almost the same as presented above, excepting one peculiarity: the Layer model deals with average crack approach and average distance between the cracks. For crack width analysis, maximal distance between cracks $s_{r,\max}$ should be taken. In present study, this distance was calculated using a simplified formula (CEN 2004):

$$s_{r,\max} = 1.3(d - y_{cr}) \approx 3.0 \text{ m}, \tag{8}$$

where y_{cr} – the centroid of fully cracked section.

It should be noted that tension-stiffening and average strains in reinforcement slightly differ for the cases of average and maximal crack distances. The analysis based on average distances leads to slight overestimation of tension-stiffening and underestimation of average steel strains and crack width:

$$w = \int_0^{s_{r,\max}} \left(\varepsilon_{s,m} - \varepsilon_{c,m} \right) dx, \tag{9}$$

where $\varepsilon_{s,m}$ and $\varepsilon_{c,m}$ – the average strains of steel and concrete, respectively.

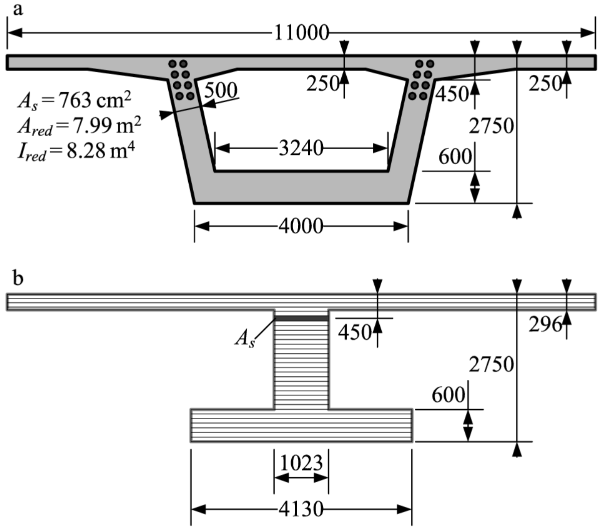

Fig. 7. Box girder cross-section (a) and its Layer model (b)

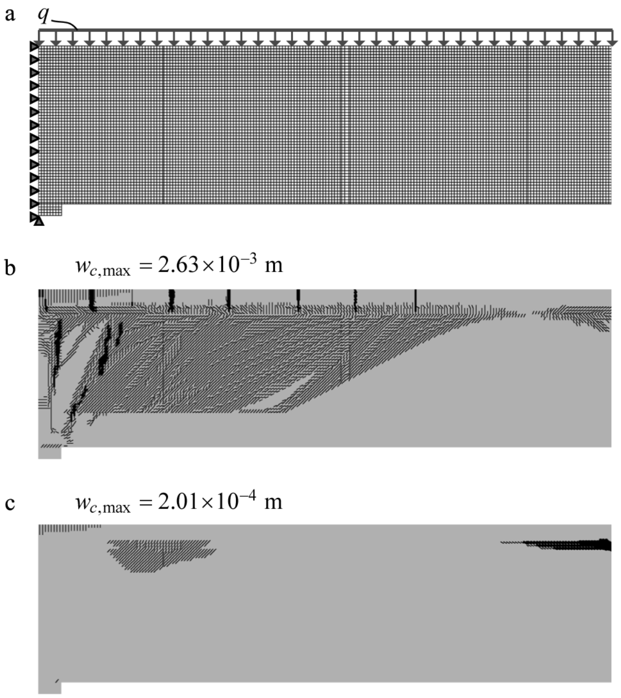

Fig. 8. FEM of the bridge segment (a) and the crack pattern predicted without (b) and with (b) pre-stressing

As in this analysis the scaling was not performed, the ultimate strain of tensile concrete calculated by Eq (3) was equal to 0.01. The obtained pre-stressing force is given in Table 3.

Table 3. Predicted pre-stressing force after losses

Calculation method	Force, MN
FE software ATENA	17.6
Layer section model	17.1
Semi-analytical technique (Muttoni, Ruiz 2007)	19.0

The analytical technique. The cracking control problem for the given cross-section was solved in (Muttoni, Ruiz 2007) under assumption that $s_{r,\max} = 2l_{ba}$. Therefore, the solution is not presented here and the required prestressing force is given in Table 3.

The differences between the forces predicted by the techniques were below 10%. The obtained differences are mostly due to simplified assumptions adopted in the Layer section model and the analytical approach.

6. Concluding remarks

Present paper investigates cracking behaviour of RC decks of continuous bridges over intermediate supports. The paper presents results of the investigation including experimental and numerical analyses of deformations and cracking of RC tensile members and a deck slab. Different numerical and semi-analytical techniques on crack controlling were analysed. The analysis has shown that all the calculation techniques, can be used both for deformational and crack width analysis.

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