- density of optimum concrete mixture - 2385 kg/m^3.

The solution seems to be fully reliable. It confirms intuitive, subjective opinion abort examination of many kinds of usual and special concretes and applied into the practice of working recipes.

Aggregates were experimentally combined into an optimum aggregate composition – characterized by maximum tightness and minimum water absorbability. The low W/C ratio of 0,391 results from the superplasticizer application into the concretes mixtures.

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THE TENSION IN FLANGES OF FLEXURAL CONTINUOUS REINFORCED CONCRETE BEAM OF A T SECTION

The purpose of the paper is to analyze what is the tension stress distribution and how to arrange longitudinal reinforcemenet in web and flanges in flexural beam of the cross section in the shape of a T. In continuous bridge girders it occurs that tension stresses are in flanges. The effective flange width in the case of compression zone is well elaborated and obvious. In the situation when the flanges are in tension there is no any rule in codes and technical literature to arrange the reinforcement in flanges. The observed damages in carry deck of Baranów Bridge over the Wieprz River direct authors to indicate the problem.

Introduction. The bending theory of reinforced concrete satisfies the three fundamental principles of the mechanics of deformable bodies:

– first, the stresses in the concrete and reinforcement satisfy the equilibrium condition,
– second, the linear strain distribution satisfies Bernoulli’s hypothesis,
– third, the constitutive laws of concrete and reinforcing bar (as described below) are obeyed.

The stresses in the compression stress block are related to the strain through the stress-strain relationship of the concrete. The stress-strain relationship of concrete is assumed to be identical to the stress-strain curve obtained from the compression test of standard concrete cylinder. The small tensile stress of concrete adjacent to the neutral axis is neglected. All the tensile stresses in the rebar are assumed to be concentrated in the centroid of the reinforcing bars with a total area of A_s. For privilege types of reinforcing bars, the stress-strain curve is first linear, with a slope of E_s up to the yield stresses f_y, and then follow roughly a straight line with a much smaller slope. In design practice these principles are applied to three typical types of beams: singly reinforced rectangular beams, doubly reinforced rectangular beams and flanged beams. Flanged beams made with concrete have mostly cross section in the shape of T. The flanges may be purposely added or may be available as parts of a structure, such as a floor system or a bridge system. In a slab-and-beam system the slab serves as the flange and the beam serves as the web.

At present, in the methods of dimensioning of reinforced concrete flexural members, as far as T section is concerned, it is assumed that in such a section the flanges are used to enhance the compression forces of the internal couples.

Flanged beam. In a continuous beam the bending moment *M* creates compression stresses in the top part of the cross section and tensile stresses in the bottom part in the middle of the span. In the beam of T section the effective flange width in compression zone is taken into account during the dimensioning.

On the supports a bending moment *M* creates the opposite situation and compression stresses are in the bottom part and tensile stresses are in the top of the cross section. The width of the compression zone of course equals the width of the web. The width of the tensile zone is not into account during the dimensioning because the total area of reinforcement is calculated according to the centroid of the reinforcing bars. But the width of the tensile zone is important in order to arrange bars of longitudinal reinforcement properly and, adequately to the distribution of tensile stresses. The bars should not be only inside the web but also in flanges of a slab monolithically joined with the web.

According to codes, for example Eurocode 2 [1], The Polish codes [2,3], the American Code ACI [4], the effective width of flanges of a **T** beam shall not exceed limited value and the rules for estimating it differ a little one from the other.

For example, the effective flange width b_{eff} for a T beam, see Fig. 1., may be derived, according to Eurocode 2, as:

$$b_{eff} = \sum b_{eff,i} + b_w \leq b$$

where

$$b_{eff,i} = 0.2b_i + 0.1l_0 \leq b \text{ and } b_{eff,i} \leq b_i$$

The effective flange width is assigned for compression area as well as for tensile one, what is additionally showed on the Fig. 4. copied from [3].

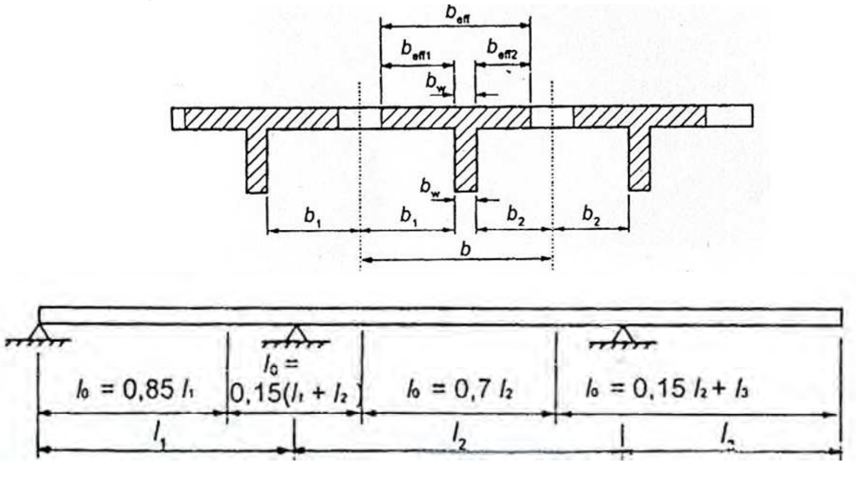


Fig. 1. Effective flange width parameters

Mostly in technical literature tensile bars are located only in the web, see for example Fig. 2. and Fig. 3.

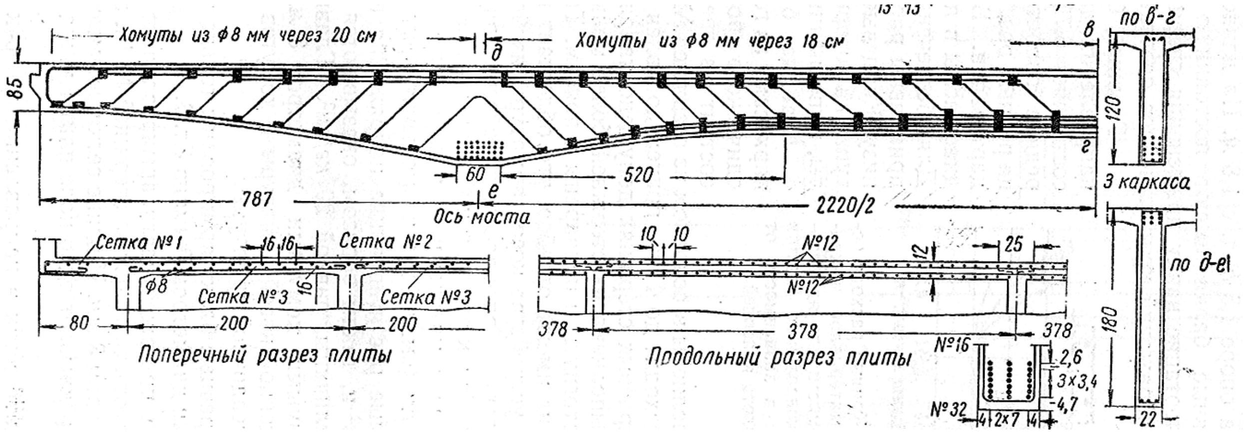


Fig. 2. The longitudinal- and cross-section of reinforced concrete bridge; [5], p. 127

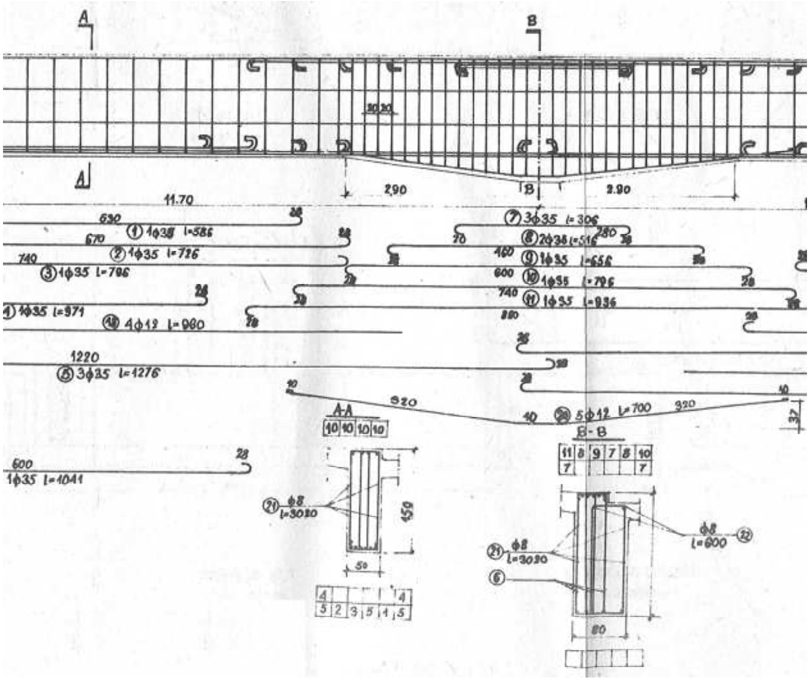


Fig. 3. The reinforcing of continuous beam;
A-A – in span, B-B – over support; [6], p. 306

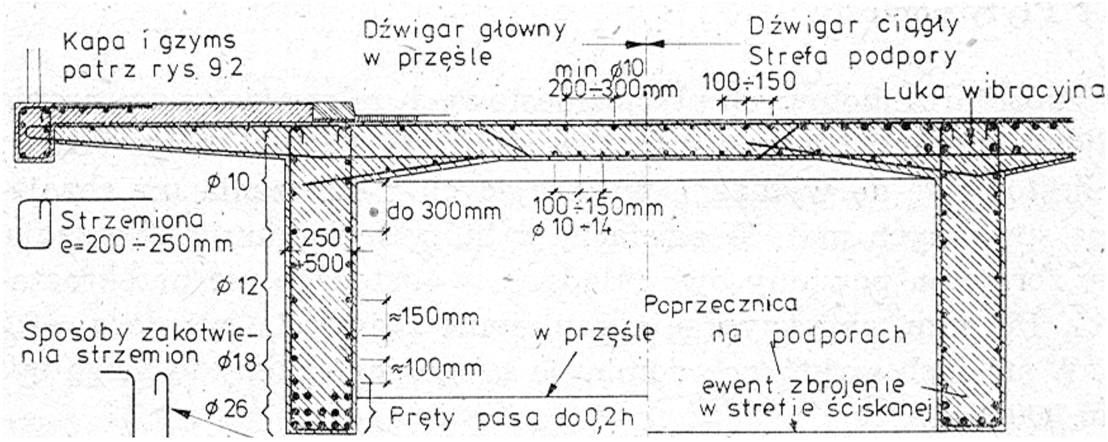


Fig. 4. The reinforcing of continuous beam bridge;
left – in span, right – over support, photo copy

Finally, after a large scale literature review, one example of reinforcement arrangement in flanges has been found, in Fritz Leonhard's book [7] on a page 226 (Polish issue) which is presented on Fig. 4. Unfortunately, there are only two sentences of author's laconic comment. On this basis it is not possible to form any general rule.

Probably the recalled example from [7] is based on the experiences coming from composite girders methods. Recalling steel-concrete composite bridges design it is impossible to input the proper amount of steel bars into small adequate for concrete web upper area. In composite bridges the idea of effective flange width is in use for years. As it is shown at a Fig. 4., the only possible way is to locate the bars in the plate. In such a case it is also necessary to analyze the shearing between a steel girder and integrated concrete slab. Because of the shear between the web and the flanges of T- section additional amount of reinforcement should be provided. But this reinforcement is the transverse one, which is placed in flanges perpendicular to the longitudinal reinforcement in the beam web.

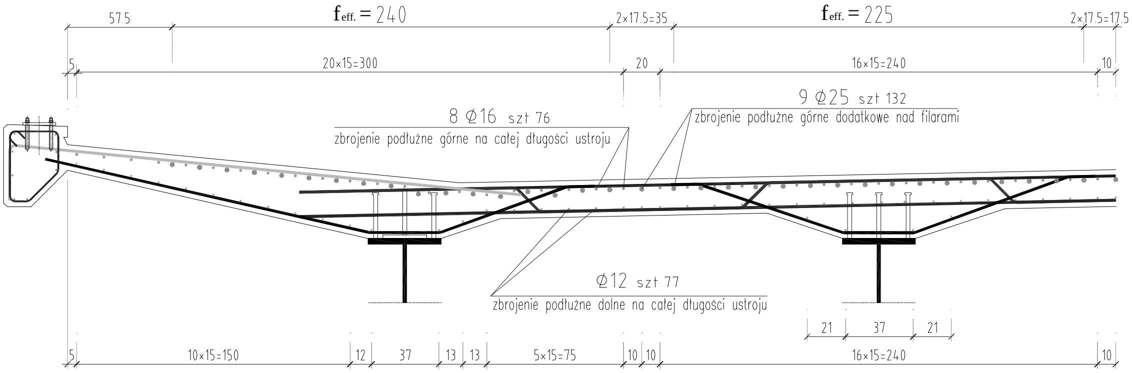


Fig. 5. The reinforcement of composite bridge in Neple over the Krzna River,
the cross-section in over pillar axe; designer - eng. G. Zieliński

The looking for the reasons of Baranów Bridge demage. During the inspection of Baranów Bridge in Drążgów (Eastern Poland) in 2005 [8], Fig. 6., there were observed, beyond typical cracks and corrosion effects, characteristic for 50 years old reinforced concrete continuous beam structure, the unexpected gaps of deck plate were observed. The gaps were perpendicular to main girders and they were located ca 1÷3 m from pillars' support axes. The width of them varies between 3 to 10 mm; Fig 7.

Fig. 6. General view on Baranów Bridge

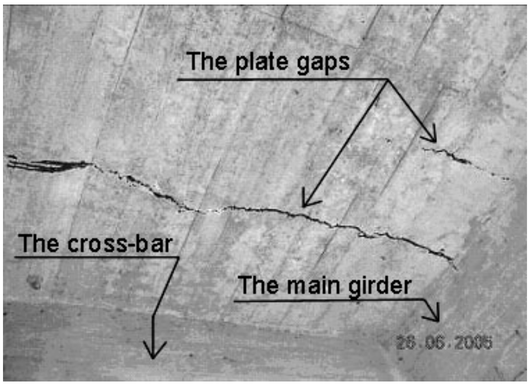
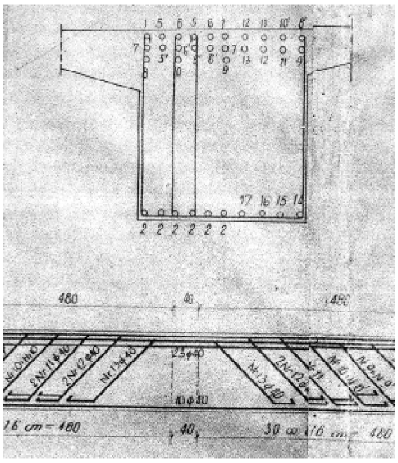


Fig. 7. The gaps

On the basis of technical draft [9] the reinforcement was located in the upper part of the beam only in the web of T section as shown on Fig. 8. There was not any reinforcement provided in flanges area, so the flanges were neglected in cooperation during design. The designers in [9] obeyed strictly the rules of reinforced concrete methods.

Let us now investigate numerically the stress state in the deck plate. Using 2D elements the FEM model has been prepared according to concrete material volume. Additionally the 1D elements for the steel reinforcement; Fig. 9. The model is an admissible simplification of the 8 span real structure. The basic loads are: the own weight and 300 kN truck which is located in the middle of the second span. All the assumed values are characteristic as in SLS.

a



b

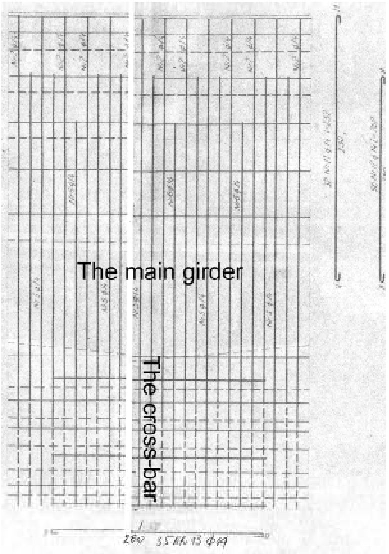


Fig. 8. The photo-copies of technical draft - over pillar area a) girder cross section; b) plate reinforcement

Fig. 9. FEM model

Normal stresses along with the bridge axe has been has been chosen from many obtained results and they are presented in graphical forms. On Fig. 10. the distribution of stresses for the whole bridge deck model is shown. In details, the normal stresses of deck plate are shown by sequence of perpendicular cross-sections in a distance period of 1m, starting from the support axe of 2nd pillar. The graphs given on Fig. 11. show the distribution of tensile stresses for two load cases: own weigh and when the truck weight is added.

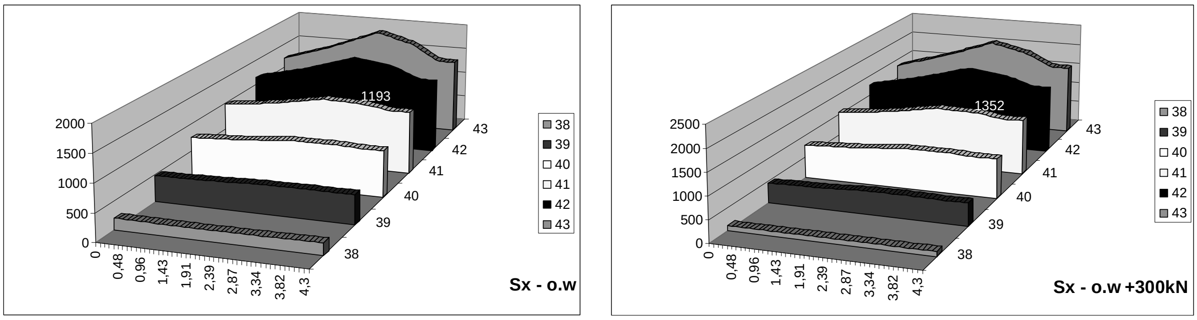


Fig. 11. Normal stress distribution of deck plate

The steel used for reinforcement can be characterized as B250. Reading the output, where for the element in the mid place of a plate, the stress value is equal 1279 kPa, we can calculate the resultant normal force as –

$$N = 1{,}352 * 0{,}2 * 1{,}0 = 0{,}2704\ MN\ ,$$

next, knowing that for this area we have 7 secondary steel bars 14 mm in diameter, the steel area is

$$A_s = 0{,}00108\ m^2\ .$$

Subsequently this yield the average stress value equals –

$$\sigma_s = N / A_s = 259{,}4\,\mathrm{MPa} \cong 250 = f_y \ \text{ and}$$

the appropriate strain is

$$\varepsilon_{sult} = \sigma_s / E_s = 0{,}0012\ .$$

Concluding this chapter, we can say that the stress state could have involved not only the typical cracks for a reinforced concrete plate (i.e. the admissible cracks with the width 0,2 mm) but also the observed gaps. The reason of this is the insufficient amount of reinforcement which is due to the neglecting of flanges cooperation in tensile zone of T girder.

Analizing building standards concerning designing and producing of reinforced concrete structures which were valid in Soviet Union and are valid now in Ukraine in chapter concerning designing reinforced concrete beams of a T section we can make the same conclusion – the width of the deck is not taken into account in zone under the tension in T-beam. The following standards were examined for this analysis: SNiP II-B.1-62*, SNiP II-21-75 and SNiP 2.03.01-84.

Conclusions. Most w Baranowie został zaprojektowany zgodnie z obowiązującymi w latach 50-tych XX w. normami i wiedzą techniczną. Nie uwzględniano wówczas szerokości współpracującej płyty w strefie rozciągania belek T-owych. Z tego powodu zbrojenie przenoszące rozciąganie ułożono w obszarze górnym środnika belki. Niezależnie od projektowania belki prowadzono wymiarowanie płyty wprowadzając jej pręty zbrojenia. Nieuwzględnienie współpracy pomiędzy belką i płytą było powodem powstania pęknięć w płycie w sąsiedztwie filarów.

Rozwój technologii belek zespolonych typu stal-beton wniósł nowe spojrzenie na zagadnienie współpracy pomiędzy dźwigarem stalowym a płytą betonową. W zakresie ściskania strefa współpracy płyty z belką jest w pełni opisana tak w żelbecie jak i w konstrukcjach zespolonych. W przypadku rozciągania w dźwigarach zespolonych zbrojenie w całości jest rozmieszczone w płycie współpracującej. Podobnie należy postępować w belkach żelbetowych.

Z przeprowadzonego przeglądu literatury i norm wynika, że brak jest wyraźnych wskazań co do postępowania przy rozmieszczeniu zbrojenia w strefach rozciągania nad podporami pośrednimi belek ciągłych. W większości podręczników z zakresu żelbetu oraz żelbetowych konstrukcji mostowych brak jest rozdziałów poświęconych sposobom zbrojenia belek teowych w przypadkach rozciągania strefy górnej, znaleziono jedynie krótkie opisy, z reguły odpowiadające schematowi występującemu w przypadku mostu w Baranowie. W dalszym ciągu, podobnie jak pół wieku temu, możliwe jest zatem projektowanie zbrojenia w zakresie jedynie prostokątnej części belki T-owej gdyż będące w powszechnym użyciu programy komputerowe w sposób automatyczny generują taki układ.

Zdaniem autorów należy wprowadzić do norm sposoby zbrojenia charakterystyczne dla belek zespolonych ciągłych również w zakresie żelbetu.

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IMPACT RESISTANCE OF SFRM MODIFIED BY VARIED SUPERPLASTICIZERS

The paper presents results of examinations of steel fiber reinforced mortar (SFRM) modified by superplasticizers based on different chemical substances. The described SFRM were made on the basis of fine aggregate cement matrix modified by the addition of hooked steel fibers. The examination was a drop-weight test. Results allowed to specify influence of the specific superplasticizer on SFRM dynamic features.

Introduction. Development of modern civil engineering includes an urgent need to develop higher performance engineering materials possessing high strength, toughness, energy absorption, durability, etc [3]. One of such steel developing high performance engineering materials is steel fiber reinforced mortar (SFRM). This paper carried out an experimental investigation on three series of SFRM. Steel fiber-reinforced mortar is more difficult to mix and place than plain concrete. Adding any type of steel fiber to a mortar reduces fluidity of the mixture because of the needlelike shape and high specific surface of the fibers. The geometry and water requirements of SFRM are an obstacle to its workability [1, 2, 12]. These factors lead to a reduction in consistency and the necessity to use superplasticizer. This study examines the effect of different new generation superplasticizers on the dynamic properties of SFRM.

It is much more difficult to quantitatively investigate dynamic properties of material than to qualitatively study static or quasi-static properties [3, 13]. Static and quasi-static properties of SFRM are already well-known and described [6, 7, 8, 9], but there is still lack of research programs concerning dynamic properties of such composites.

Materials and test method. Materials consisted of ordinary Portland cement with 28-day compressive strength of 32.5 MPa (CEM I 32.5), natural fine aggregate of maximum size 2mm, tap water for mixing and curing and three superplasticizing admixtures. The superplasticizers are codified as PC3, PE and CRSP. The PC3 is a superplasticizer based on polycarboxylate, the PE superplasticizer is based on