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A. J. Notkus, Z. Kamaitis

1. Introduction

Concrete box-girder bridges are most widespread bridge type today. Many prestressed box-girder bridges are constructed by balanced cantilever method. Erection of a bridge starts at piers where segments are progressively placed on each side of the pier and tied together by post-tensioning of top tendons. The bridge construction is completed by stressing the bottom cables in the span. The method of construction influences the design and structural characteristics of the cross-sections. The structural system is changed many times from statically determinate cantilever during erection up to frame or continuous beam in the final state. It brings a number of uncertainties concerning the actual state of stress of the completed structure. The additional stresses are produced in the structure during its construction. Sometimes the state of stress cannot be characterized by simple calculations. More detailed structural analysis must be used.

In many precast segmental box-girder bridges built in Lithuania intensive shear cracking in the box-girder webs is observed (Fig 1). The first cracks in some bridges subjected only to dead load weight have been observed just after construction. Shear cracks are inclined at approximately 15-60 degree angle to the longitudinal axis of the girders. They usually appear in a zone near supports or close to midspan where minimum cross-section depth or tendon anchors are provided. Cracks develop when the principal tensile stress due to combination of service loads and restraint forces exceed the tensile strength of concrete. The possible causes of cracking in post-tensioned box girder bridges are discussed in [1, 2]. Cracking may occur for a number of reasons, one of them is inefficient attention of designers to the effects caused by prestressing forces.

The bridges in question built in Lithuania were designed to meet the SNIP [3] requirements. Shear

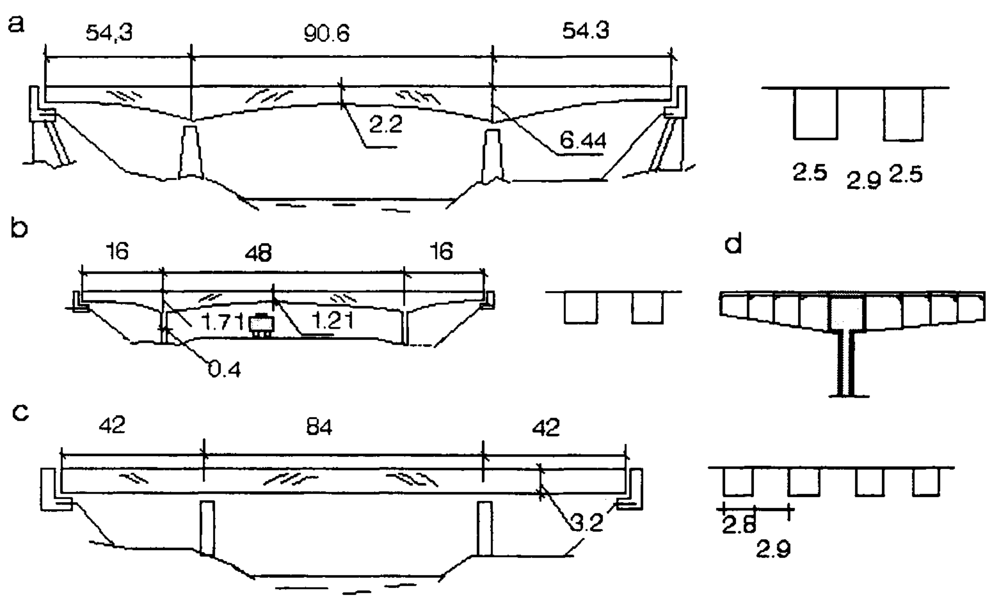

Fig 1. Cracking of prestressed concrete segmental box-girder bridges

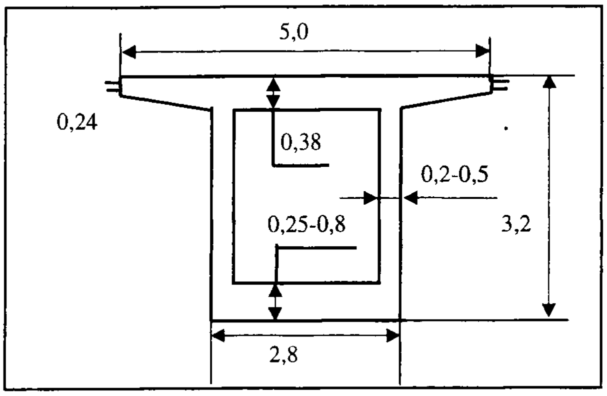

Fig 2. Cross-section of box-girder (Šilo St. bridge – Fig 1, c)

cracks are frequently found in the webs of box-girders despite the fact that calculations according to SNIP show sufficient safety margin. It was found [1] that the code provisions did not pay attention to many effects in structures. On the other hand, the analysis shows [4] that designers do not take in account the additional shearing stresses which occur in the structures during construction and prestressing of tendons. This, in summary, results in damage of bridges serious enough to be repaired.

In this article the shear cracking of box-girder segmental prestressed concrete bridges due to post-tensioning forces is analyzed.

2. Analysis of stresses in the webs using finite element method

Recently constructed Šilo St. bridge over the river Neris was chosen for investigation of stress-strain state (Fig 1,c). We used a finite-elements method to gain a clear picture of the shear stress distribution in the webs of box-girders. The cross-section of the box-girder used in analysis is shown in Fig 2. Spatial computer model for continuous girder was prepared (Fig 3, a, b) in which actual geometry of girders, width of webs and slabs as well as variation of structural scheme from cantilever during erection up to continuous girder (after casting of slab joints and supporting girder ends on abutments) are taken in consideration. The cantilever girder was loaded by weight of segments and prestressing in upper cables while the continuous girder – by permanent load (deck, cast-in-situ joints, heating pipes, etc), prestressing in lower cables and traffic loads. Other actions are also modelled: creep of concrete, temperature variation (dilatation and gradient). Action effects and stresses due to each of these actions and their combinations are calculated. The graphical representation of shear stresses in the webs is presented in Fig 3 c. Maximum values of obtained shear stresses in cracking zone τ_{zy} and $\tau_{max} = (\sigma_1 - \sigma_3)/2$ due to each of mentioned actions and their combinations are presented in Table 1 and Fig 3 d.

To avoid shear cracking in the webs, according to SNIP [3] the following two conditions shall be satisfied:

$$\tau_b = \tau_q + \tau_m \le m_{b6} \cdot R_{b,sh}, \quad (1)$$

$$\sigma_1 \sigma_{1bt,ser} \le R_{bt,ser}. \quad (2)$$

where τ_q is shear stress due to external load and prestress; τ_m is shear stress due to torsion; $R_{b,sh}$, $R_{bt,ser}$ are flexural shear and tensile strength of concrete; σ_1 is principal tensile stress in concrete respectively.

Table 1. Maximum shear stresses in cracked zone

No	Loads, actions	Composition of combination	Tangential stresses τ_{xy}/τ_{max}
Cantilevering system			*MPa*
1	Permanent 1. Weight of segments	1	2.5/4.0
2	Prestressing of upper cables	2	1.5/9.0
	Combination 1	1+2	3.0/6.5
System of continuous girders			
3	Permanent 2. Deck, cast-in-situ, etc.	3	1.2/1.5
4	Prestressing of lower cables	4	1.5/2.5
	Combination 2	1+2+3++4	3.8/6.5
	Only prestressing in cables *Combination 3*	2+4	1.5/9.5
5	Temperature gradient of 10^0 C	5	0.5/1
6	Highway traffic load AK11	6	0.5/0.75
	Combination 4	1+2+3+4+5	4.0/6.5
	Combination 5	1+2+3+4+6	4.0/7.0

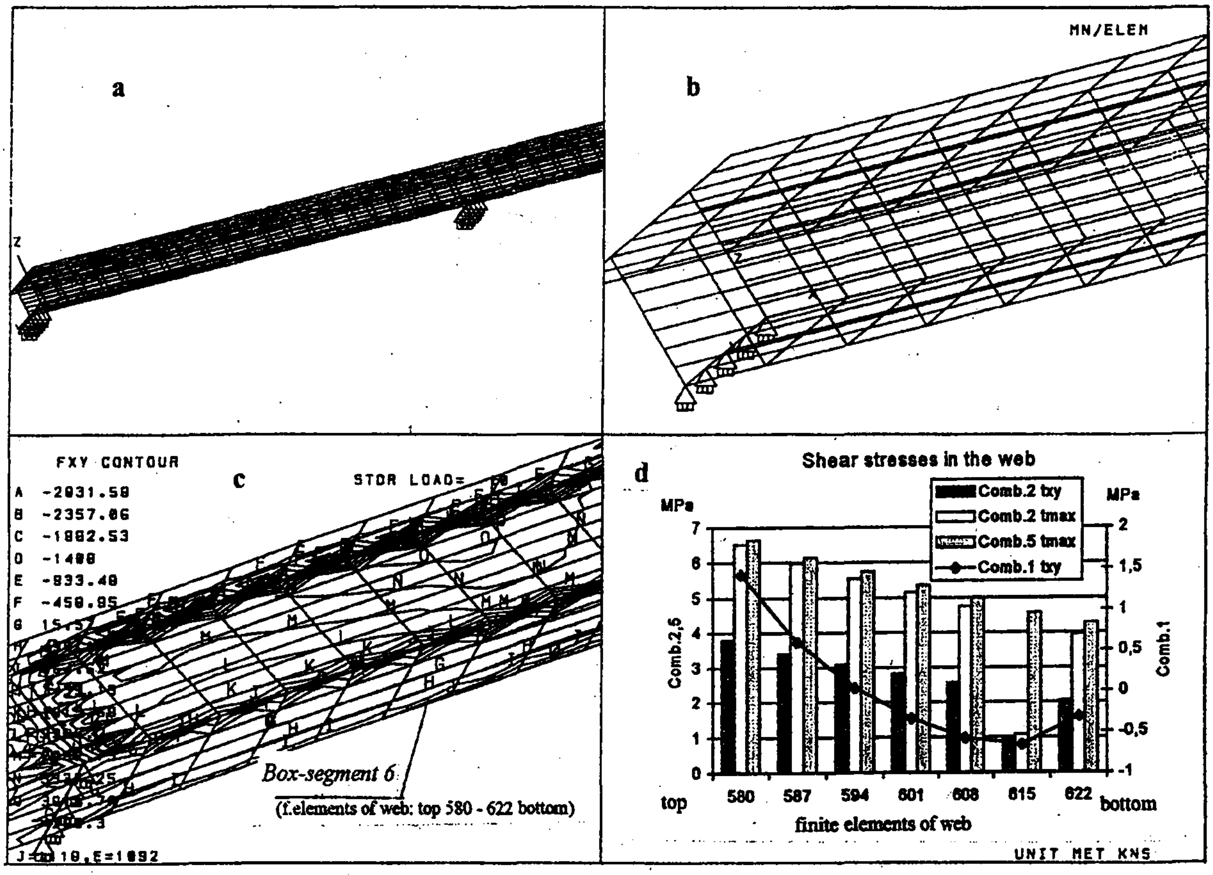

Fig 3. Design structural system of the bridge girder (a, b), distribution of shear stress due to combination of actions No 5 (c), and diagram of shear stress through the web cross-section in the box-segment 6 due to combination of actions (d)

The equation for principal stresses of a point subjected to combined shear, τ_{xy}, and normal, σ_x and σ_y stress:

$$\sigma_{1,2} = \left(\sigma_x + \sigma_y\right)/2 \pm \sqrt{\left(\sigma_x + \sigma_y\right)/4 + \tau_{xy}^2}. \qquad (3)$$

Examination of the results predicted by finite element model and presented in Table 1 shows that the shear stresses τ_{max} up to two times exceed the allowable limit according to condition (1) which for the bridge in question is equal to 3.5 MPa.

The calculated principle stress σ_1 reaches 2 MPa and exceeds allowable value according to Eq. (2) ($R_{bt,ser}$ = 1.5 MPa). Its direction makes with horizontal an angle 15-25 degrees and approximately it corresponds to observed direction of diagonal cracks in the webs. Shear and principal stresses unavoidable propagate shearing cracks.

The large discrepancy between the predicted by finite element model shear and principal stresses and those calculated by Eq (1) and (2) are attributed to the Code provisions and imperfect design methods which do not pay attention to many effects in structures and first of all to the effect pre-tensioning in bridge girders.

In traditional direct calculations of shear stress τ_q through shear force $\left(\tau = Q \cdot S / b \cdot J\right)$, the influence of prestressing in lower cables on shear stress practically is not taken into account since shear force due to prestressing of these cables does not occur (Fig 4). Shear force Q due to horizontal forces does not develop because direction of shear stress changes within cross-section depth and their sum is equal to zero (Fig 3 d; 6 c). If the moment due to prestressing of tendons M = const, Q = dM / dx = 0.

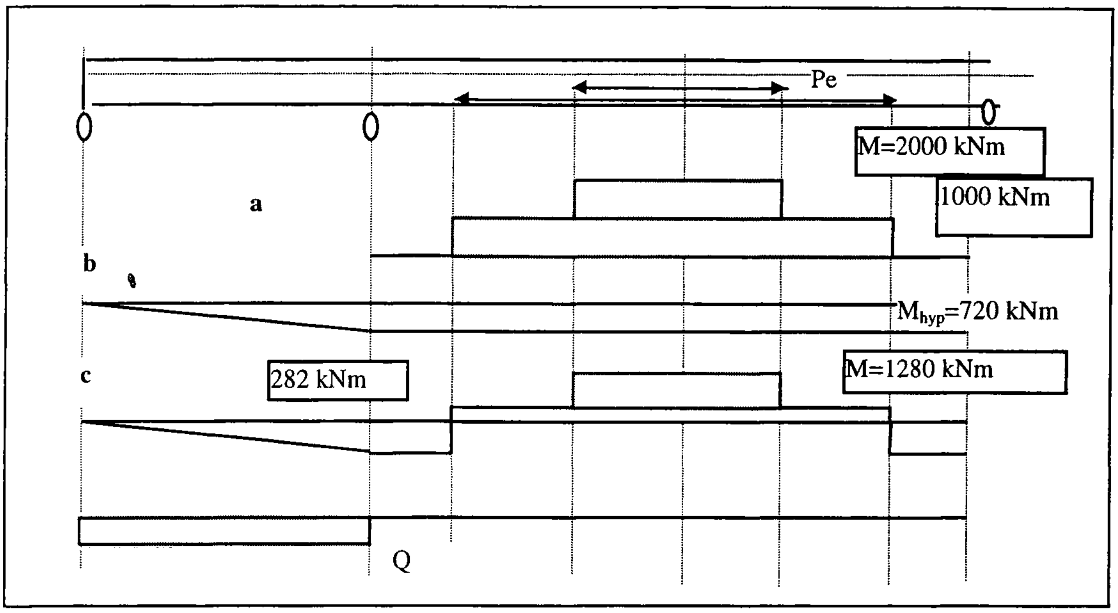

Fig 4. Residual action effects in simply supported (a) and continuous three span beam (c) due to prestressing of two cables (b – hyperstatic moments)

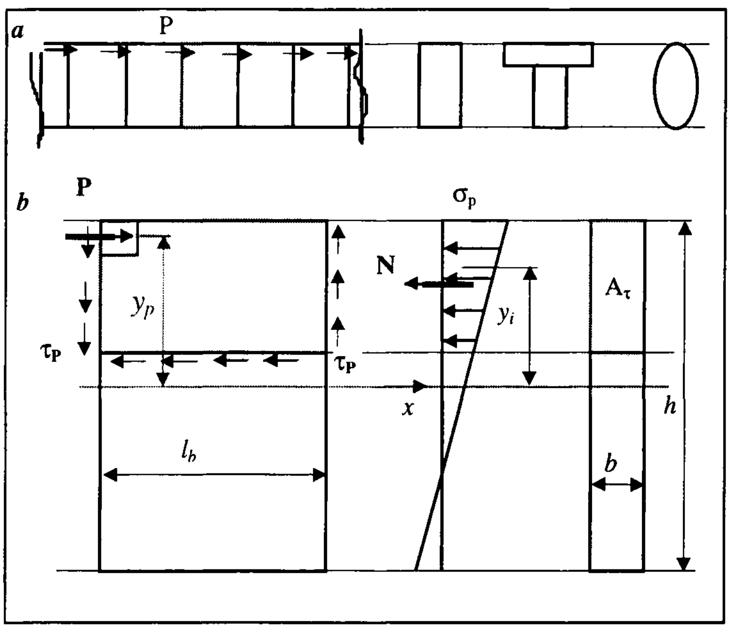

Fig 5. Prestressing forces in bridge beam (a) and stress distribution in the bridge segment due to prestressing force (b)

The analysis of temperature effects has shown [1] that additional loads can be induced by temperature gradients in bridge girders as is also shown in Table 1 (see combination No 5). While a discussion of temperature effects is outside the scope of this paper, it is important that a careful analysis be done for box-girder bridges.

3. Shear stress due to post-tensioning forces

If the post-tensioned force P is applied to the bridge segment, normal σ_P, and shear τ_P stresses are set up as shown in Fig 5. The role which each of these two stresses plays in the behaviour of the web has not been fully understood. It is assumed that normal forces and shears are uniformly distributed over the thickness of the web and bending stresses to be linear distributed over the depth of segment. Normal stress σ_P appears due to prestressing force P acting only in this segment. Normal forces from adjacent segments are applied to the both sides of the segment in opposite directions and they are not shown in Fig 5 b. Only the effects of straight tendons are considered, the stress concentration due to post-tensioning in the zone of anchors are not taken in consideration.

For the given stress distribution in concrete at any distance y_i from the neutral axis, τ_P may be obtained from the following equilibrium equation:

$$\sum x = 0 . \quad P - N - \tau_P \cdot b \cdot l_b = 0 . \tag{4}$$

The horizontal force, N, acting in segment is

$$N = ? \quad N = \int \sigma_P \cdot dA ,$$

The value of σ_P can be determined from:

$$\sigma_P = \frac{P}{A} \pm \frac{M \cdot y_i}{J} = \frac{P}{A} \pm \frac{(P \cdot y_P) \cdot y_i}{J} . \tag{5}$$

$$N = \frac{P \int dA}{A} \pm \frac{P \cdot y_P \cdot \int y_i \cdot dA}{J} = \frac{P \cdot A\tau}{A} \pm \frac{P \cdot y_P \cdot S\tau}{J} . \tag{6}$$

Substituting Eq (6) by Eq (4), we obtain:

$$\tau_P = \frac{1}{l_b \cdot b} \cdot (P - N) = \frac{1}{l_b \cdot b} \cdot \left(P - \frac{P \cdot A\tau}{A} - \frac{P \cdot y_P \cdot S\tau}{J}\right) =$$

$$= \frac{P}{l_b \cdot b} \cdot \left(1 - \frac{A_\tau}{A} - \frac{y_P \cdot S_\tau}{J}\right) .$$

Equation for determination the prestressing-induced shear stresses:

$$\tau_P = \frac{P}{l_b \cdot b} \cdot \left(1 - \frac{A_\tau}{A} - \frac{y_P \cdot S_\tau}{J}\right) . \tag{7}$$

This equation indicates a variation of shear stress through the depth of the cross-section of the web.

Eq (7) can be used to predict the stress τ_P in the webs of all types of sections (rectangular, I and T-shaped, circular, etc).

4. Numerical example

To illustrate the use of Eq (7), the case will be considered of I and T-shape bridge girders (Fig 6 a).

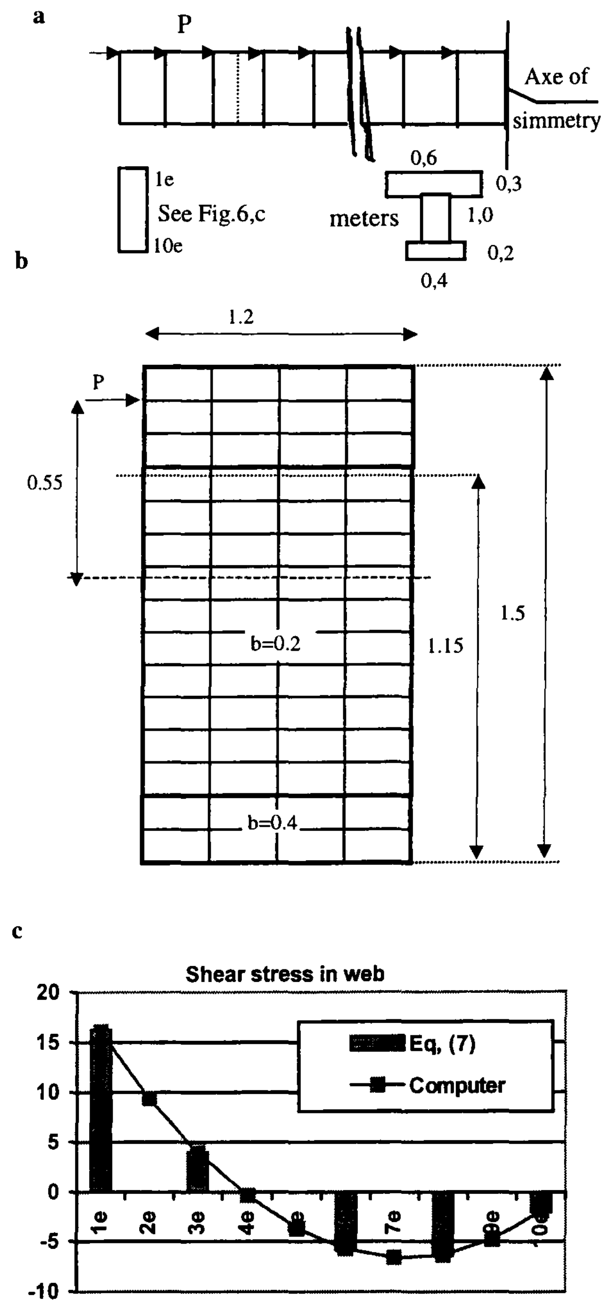

Fig 6. Bridge girder (a), FE mesh of bridge segment for T-shape (b), and shear stresses diagram for I-shape (c)

Available information for T-shape girder

Geometrical characteristics of cross-sections

H=1,5 m, b=0,2 m, l_b = 4·0,3=1,2 m, A=0,46 m^2, I= 0,11278 m^4, z_{sc}=0,85 m , y_p= 0.55 m, $A\tau$ = 0,19 m^2, $S\tau$=0,09003 m^3.

It is required to determine the shear stress in the section y = 1.5 – 0.35 = 1.15 m due to prestressing force P = 400 kN.

From Eq (7):

$$\tau_P = \frac{P}{l_b \cdot b} \cdot (1 - \frac{A_\tau}{A} - \frac{y_P \cdot S_\tau}{J}) =$$

$$= \frac{400}{1{,}2 \cdot 0{,}2}(1 - \frac{0{,}19}{0{,}46} - \frac{0{,}55 \cdot 0{,}09003}{0{,}11278}) = 246{,}6 \text{ kN/m}^2.$$

Computer result is $\tau_P = 245$ *kN/m*^{2}

A comparison of shear stress calculated from Eq (7) shows good agreement with the result obtained from computer analysis (Fig 6 c).

5. Conclusions

In box-girder statically indeterminate concrete bridges constructed by balanced cantilever method, the shear cracks in the webs are frequently found despite the fact that calculations according to SNIP show sufficient safety margin. The SNIP building code does not reflect many effects on structures. Shear cracks can occur in sections where very thin web sections are used. These cracks inevitably affect the service performance of the bridge girders.

Imperfect design methods should be considered to be the cause of cracking webs of prestressed concrete bridges and, first of all, not sufficient evaluation of effect of prestressing in statically indeterminate structures.

The finite element analysis allowed to obtain the real picture of shear stress distribution in the webs of continuous beam. Formula for predicting shear stress due to post-tensioning forces is proposed.

This investigation provides a starting point for the practical engineer to check and evaluate the post-tensioning forces on the thin web design of the box-girder bridges.

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TANGENTINIŲ ĮTEMPIŲ ĮTEMPTOJO GELŽBETONIO TILTŲ SIJŲ SIENELĖSE ĮVERTINIMAS

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Santrauka

Daugelyje Lietuvoje ir užsienyje gembiniu-pusiausvyriniu būdu pastatytų dėžinio skerspjūvio įtemptojo gelžbetonio tiltų pastebėtas sijų sienelių pleišėjimas įstrižuose pjūviuose. Šis pleišėjimas negali būti paaiškintas taikant įprastus tangentinių ir svarbiausiųjų tempimo įtempių skaičiavimo metodus, nes šių įtempių reikšmės neviršija projektavimo normomis [3] ribojamų dydžių. Buvo atlikta vieno iš neseniai pastatytų tiltų, kuriame atsirado įstriži plyšiai dar prieš eksploataciją (1 c pav.), detali sijų įtemptojo būvio kompiuterinė analizė [2], kurios metu buvo modeliuota reali sijų geometrija, nuolatinės ir laikinosios apkrovos, viršutinių ir apatinių lynų įtempimo jėgos, temperatūros poveikiai ir jų deriniai (1 lent.). Gauta, kad koordinatiniai tangentiniai įtempiai τ_{zy} (3 pav., 1 lentelė) ir ypač svarbiausieji $\tau_{max} = (\sigma_1 - \sigma_3)/2$, pleišėjimo zonose žymiai viršija leistiną reikšmę 3,5 MPa ir reikšmes, apskaičiuotas pagal Žuravskio formulę ($\tau = Q \cdot S / b \cdot J$). Svarbiausieji tempimo įtempiai σ_1 taip pat, nors ir mažiau, viršija leistinas reikšmes. Buvo parodyta, kad horizontalių lynų įtempimo jėgų sukeltas $M_P = const$ nesukelia skersinės jėgos $Q=dM/dx=0$ (4 pav.). Tangentinių įtempių τ_{Pxy} diagrama per sienelės aukštį yra dviženklė, o įtempių atstojamųjų suma lygi nuliui (3, 6 pav.). Taigi šie tangentiniai išankstinio apspaudimo įtempiai negali būti apskaičiuoti per skersinę jėgą Q. Šiuos įtempius siūloma skaičiuoti pagal kompiuterinius modelius, o paprastesniais atvejais pasinaudoti 5 pav. ir išraiškose (4-6) gauta formule (7), skirta tangentiniams įtempiams nuo lynų įtempimų jėgų apskaičiuoti. Formulės tinkamumas buvo patikrintas sulyginant analitines reikšmes su kompiuterinių skaičiavimų rezultatais (6 pav.). Pateikiamas tangentinių įtempių skaičiavimo pavyzdys.

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