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ДОСЛІДЖЕННЯ КРИТИЧНИХ СТИКОВИХ НАПРУГ В СТІНІ БАЛКИ МІЖ ОТВОРАМИ В ПЕРФОРОВАНИХ БАЛКАХ ВИКОРИСТОВУЮЧИ МЕТОД КІНЦЕВИХ ЕЛЕМЕНТІВ

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Анотація. У цій статті проаналізовані балки, довжина яких 12 м, товщина стінки - від 6 мм до 12 мм і висота від 500 до 1000 мм. Діаметр перфорації підібраний від половини висоти стінки до висоти стінки мінус 100 мм. Площа двох полиць була підібрана такою, що дорівнює двом товщинам, а товщина полиць дорівнювала двом товщинам стінки. Розглянуті балки були на двох опорах, навантажені рівномірно розподіленим навантаженням, їхні верхні полиці були обмежені площиною. Основною метою роботи було визначити критичні стикові напруги в стіні між отворами, при яких стінка балки втратила б місцеву стійкість. Розрахунки проведені при використанні аналітичних формул та методу кінцевих елементів. Результати розрахунків були порівняні між собою і подані у вигляді графіків.

Ключові слова: балка з перфорованою стінкою, перфорація, форма перфорації, метод кінцевих елементів, діаметр перфорації, раціональна висота, випуклість стінки балки між перфораціями, критичне навантаження, критичне стикове навантаження.

ИССЛЕДОВАНИЕ КРИТИЧЕСКИХ КАСАТЕЛЬНЫХ НАПРЯЖЕНИЙ В СТЕНКЕ-ПЕРЕМЫЧКЕ ПЕРФОРИРОВАННЫХ БАЛОК ПУТЕМ ПРИМЕНЕНИЯ МЕТОДА КОНЕЧНЫХ ЭЛЕМЕНТОВ

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Аннотация. В этой статье проанализированы балки, длина которых 12 м и толщина стенки от 6 мм до 12 мм при высоте стенки от 500 до 1000 мм. Диаметр перфорации подобран от половины высоты стенки до высоты стенки минус 100 мм. Площадь двух полок подобрана равной площади стенки, а толщина полок - равной двум толщинам стенки. Рассматриваемые балки были нагруженные равномерно распределенной нагрузкой, деформации верхней полки были ограничены из плоскости. Основная цель работы была установить критические касательные напряжения в стенке-перемычке, при которых она утратила бы местную устойчивость. Расчеты проведены с использованием аналитических формул и метода конечных элементов. Результаты расчетов были сопоставлены между собой и приведены в виде графиков.

Ключевые слова: перфорированная балка, перфорация, форма перфорации, метод конечных элементов, диаметр перфорации, выпучивание стенки-перемычки, критические касательные напряжения.

FINITE ELEMENT METHOD INVESTIGATION OF CRITICAL SHEAR STRESSES OF THE WEB OF STEEL BEAMS

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Abstract. The beams of 12 m long, web thickness is from 6 mm to 12 mm and web depth from 500 mm to 1000 mm, are analysed in this paper. The perforation diameter is selected from half of the web depth to the total web depth 100 mm minus. The cross-section area of two flanges was selected equal to the cross-sectional area of the web. The flanges thickness is equal two times the web thickness. Given beams have been loaded equally as for distributed load the upper flanges deformation are restrained out of plane. The general purpose of this work is to settle the critical shear stresses in the web wall and in this case it would lost the local stability. Calculations have been done using analytical formular and finite elements methods. Calculation results are represented compared with each other and are shown as charts.

Keywords: perforated steel beam, perforation, perforation form, finite element method, diameter of perforation, web-post buckling, critical shear stresses.

1. Introduction

The bigger area the flange is the more rational the beam is [1]. Thus, the flanges cross-section should be as big as possible to get a beam similar to the castellated beams (Fig 1).

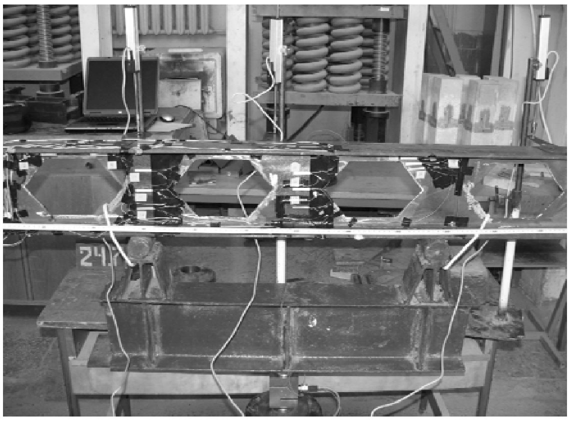


Fig 1. Castellated steel beam.

The perforated beams have a wide range of applications ranging from commercial and industrial buildings to parking garages. They have a scene of beauty as well. These beams are well acceptable for big spans. Perforated beams have a structural advantage, because it is possible to sink different kind of communications through the web openings of the beam [2], which allows to save the effective height of the room, which is very important in multi-storey buildings. Any savings in steel weight because of the use of these beams are also a positive factor as it relates to the overall supporting structural and foundation systems. In addition to the economic and functional advantages aforementioned, these structures have an aesthetic and attractive look too.

Advanced analysis for castellated beams generally has a verification character, i.e. a beam with certain dimensions can be calculated whether it carry a load or not. In addition , there are tables used to select different types of castellated beams according to the load applied and the span. As a result of the accurate analysis using finite element method, design of castellated beams has become easier. An existing problem raises for castellated beams determining the rational dimensions like perforation diameter, distance between perforations, web thickness, effective depth and etc. Therefore, it is very important to develop a method for the selection the rational parameters of castellated beams.

The beams used nowadays are built up using steel plates not only of rolled sections [3,4]. They are called steel plate girders with web openings.

The steel plate girders with multiple web openings also may be made from normal I-section. The web of such beam is split lengthwise in a rack-shaped pattern (Fig 2.). The halves so obtained are shifted a half-pitch in relation to one another and then welded together at the tops of the teeth. The result is a beam with a row of hexagonal holes in the body. The beam is much deeper than the original profile it is made from, while its weight is of course (almost) the same.

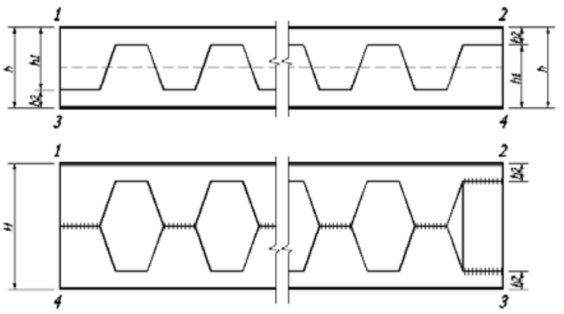


Fig 2. Producing of the castellated steel beam.

Using such beams allow us to dispose of all cross-sectional dimensions and find rational ones.

As mentioned above, as much as possible cross-section area of the beam should be concentrated in the flanges and as less as possible in the wall. In this case it is possible to dispose of all cross-sectional dimensions, so the beams may be designed with very thin walls. Because of this, local stability of the web-post becomes very topical.

Now, according to the experimental data, are known eight failure modes of castellated beams [5, 6, 7]:

1. Flexural mechanism;
2. Lateral torsional buckling;
3. Distortional buckling;
4. **Web post buckling due to shear force;**
5. Web post buckling due to compression force;
6. Vierendeel or shear mechanism;
7. Rupture of welded joints;
8. Ultimate deflection.

The main aim of the calculations is to estimate the main differences between calculations' results obtained using finite element method (numerical experiment) and theoretical equations

2. Scope and aim of the investigation

Nowadays many researchers have investigated quite well the calculation methods for load carrying capacity of castellated beams. There are known not a single theory (which we can trust and use) for calculation the castellated beams. Some of them are based on calculation of the stresses in characteristic points such as in corners of the openings and in flanges, local stability of the web posts and local stability of tee sections over openings etc. [8, 9, and 10].

Most difficult part is to find rational depth of the beams, which depends on many things. To succeed this is very complicated for the mathematical difficulty in dealing with many unknowns and formulas. Using finite element method analysis enables to avoid such problems as mentioned above.

The objective of this paper is to study failure of the castellated beams with particular emphasis on web-post buckling. The goal is to investigate experimentally (numerical experiment using finite element method) and to analyse theoretically the critical shear stresses of the web-post of the castellated beams.

The main aim of this paper is:

1. To create charts $\tau_{cr,exp}$=f(d) and $\tau_{cr,teor}$ =f(d), where d – diameter of the perforation; $\tau_{cr,exp}$ [MPa] – critical shear stresses (when web-post buckles) obtained using numerical experiment (FEM); $\tau_{cr,teor}$ [MPa] – critical shear stresses (when web-post buckles) obtained according to theoretical formula;
2. To compare the charts mentioned above;
3. To analyse the charts mentioned above.

3. Description of the problem

Simply supported perforated beams with hexagonal form of perforations were analysed (Fig 3.).

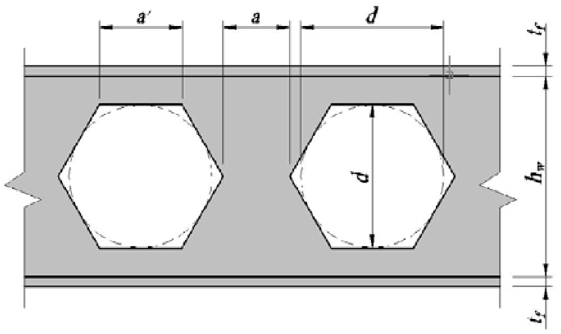


Fig 3. Geometry of beam's web with hexagonal form of perforation.

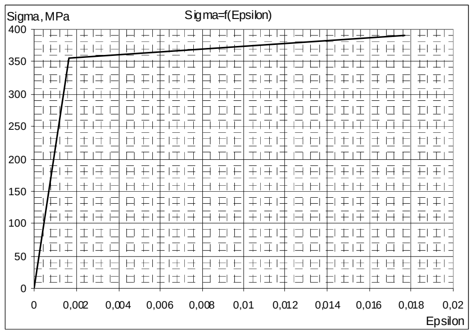


Fig 4. Bi-linear stress-strain curve of material.

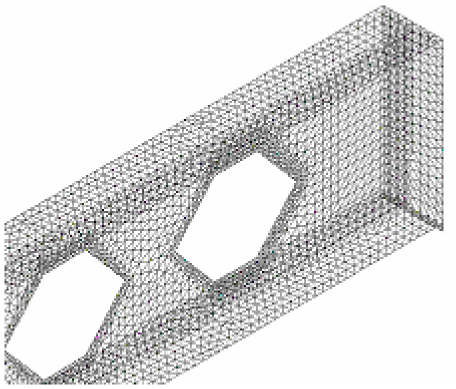


Fig 5. Finite element model with hexagonal form of perforation.

The main beam parameters taken for analysis are as follow:

1. Length L = 12 m;
2. Web depth h_w = 500 ÷1000 mm, every 10 mm;
3. Web thickness t_w = 6 ÷ 12 mm, every 1 mm;
4. Flange thickness t_f equal to doubled thickness of the web;
5. Thickness of the end stiffeners t_s = 10 mm;
6. Cross-sectional area of two flanges equal to the cross-sectional area of the web;
7. Diameter of perforations $h_w/2 \div h_w-100$ mm, every 10 mm;
8. Distance between the edges of perforations a = 15 cm;
9. Given an integer number of perforations, the distance from the end of the beam to the edge of first perforation is minimal, but not less than 250 mm;
10. The uniformly distributed load per unit of the span length;
11. The upper flange restrained out of plane;
12. Analysis – geometrically and physically nonlinear;
13. Steel grade S355;
14. Form of perforation is hexagon circumscribed about circle.

All values of these parameters were selected because they are most common in practical use. Of course, some of them may be changed, especially those mentioned in clauses 1, 4, 6, and 8.

3.1. Finite element method analysis

Calculations are performed using finite element programme COSMOSM. As type of finite elements SHELL3T has been used. In order to simulate the structural behaviour of castellated steel beams with hexagonal web openings, a finite element model is established as follows:

- With material non-linearity incorporated into the finite element model. A bi-linear stress-strain curve is adopted in the material modelling of steel as is shown in Fig 4.
- Moreover, with geometric non-linearity incorporated into the finite element model, large deformation in the model may be accurately predicted, allowing load redistribution in the web across the opening after initial yielding.

Fig 5 illustrates the finite element model where the flanges and the web of a steel beam are discretised with three-noded shell elements.

A hexagonal opening is formed in the web with refined mesh configuration. After sensitivity studies on the density of the finite element mesh, it was found that a size of finite element may be about 5 cm. A size of finite element around the opening is chosen about 1.5 cm. The calculations were made with iterations, as the analysis was geometrically and physically nonlinear. Arc-length algorithm has been chosen for calculations. This algorithm was chosen not accidentally. Other calculation methods were not suitable, because when beam buckles, the increase out of plane displacements when load is leaving almost the same, is observing. The calculations stop then the maximum stress or strain values were reached or node's displacement exceeds L/250, or it starts to increase very rapidly without sufficient load changing. Since the geometrical and physical analysis was proceeded, the values of stresses, nodal displacements and buckling load may be received from the results of single calculation.

The critical shear stresses $\tau_{cr,exp}$ were obtained in this way:

1. For all foreseen beams a buckling load factor (BLF) using linear buckling analysis was calculated;
2. For all foreseen beams the shear stresses in the nearest to the end of the beam web-post using nonlinear analysis were calculated;
3. For all foreseen beams critical shear stresses cr,exp according to formula (1) were calculated.

$$\tau_{cr,exp} = BLF^*\tau \qquad (1)$$

3.2. Theoretical method analysis

Theoretical equation (2) for calculation of the critical shear stresses is developed by V. M. Dobrachev and V. G. Sebeshev [8].

$$\tau_{cr} = 5\kappa E/[6(1-\nu^2)\lambda^2_{wp}] \qquad (2)$$

Where $\kappa = (1-\nu) + 5\alpha^2$; $\nu= 0.3$ – Poisson ratio; $\alpha= a'/d$; $\lambda_{wp} = d/t_w$ – slenderness of the web-post.

4. Results of the calculations

From experimental investigation (numerical experiment using finite element method) and theoretical analysis the critical shear stresses of the web-post of the castellated beams are obtained. The charts $\tau_{cr,exp}$=f(d) and $\tau_{cr,teor}$=f(d) are drawn.

The calculations are performed for wide range of beams, but because of big amount of results are presented only some of them.

4.1. Results of analysis using finite element method

Because of big amount of a data bellow are given the charts $\tau_{cr,exp}$=f(d) only for web thicknesses 6, 9, 12 mm, and with web depth 500÷980 mm, every 40 mm (Fig 6).

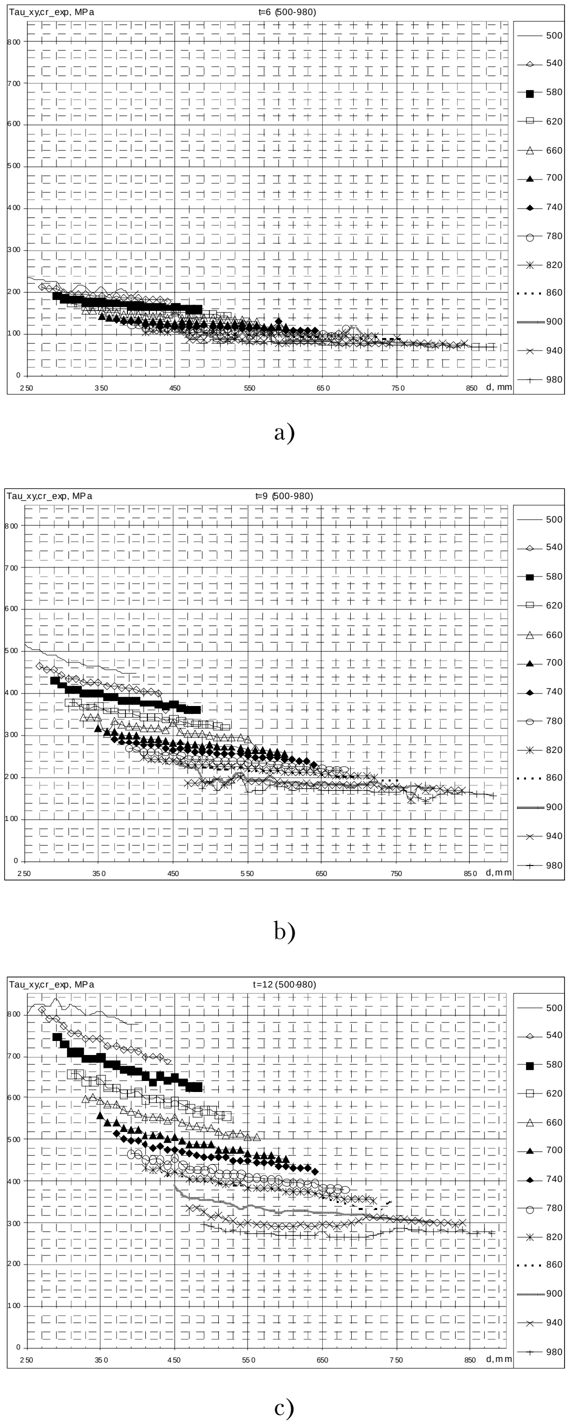
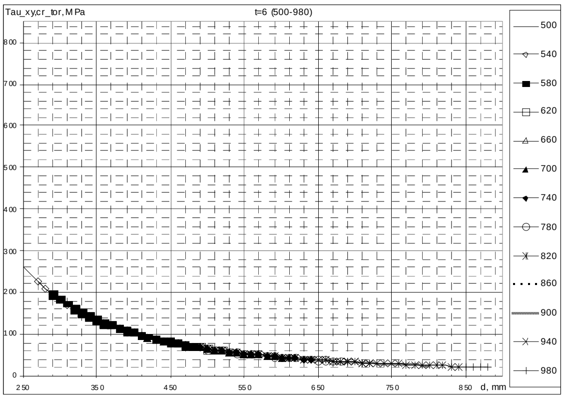


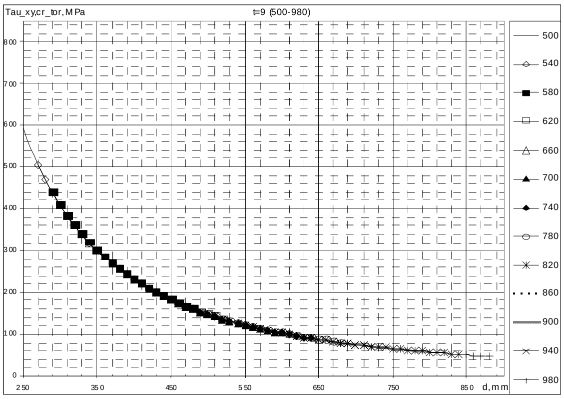
Fig 6. Dependence of cr,exp versus d: a – for t_w = 6 mm; b – for t_w = 9 mm; c – for t_w = 12 mm.

To see the critical shear stresses changes in different charts, the scale of ordinate has been taken the same.

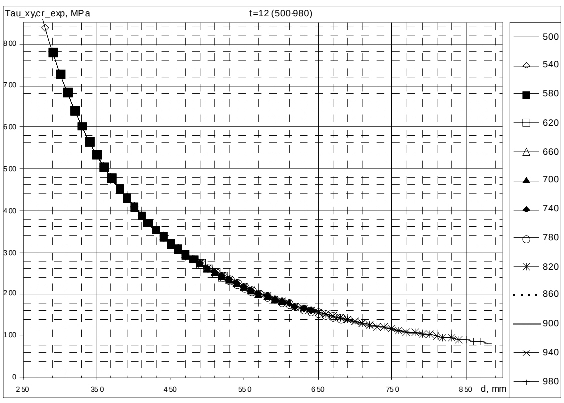
From the charts above it is seen, that the critical shear stresses $\tau_{cr,exp}$ depends on web slenderness.



a)

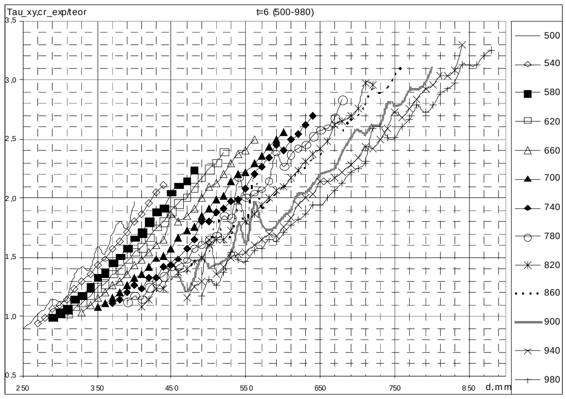


b)

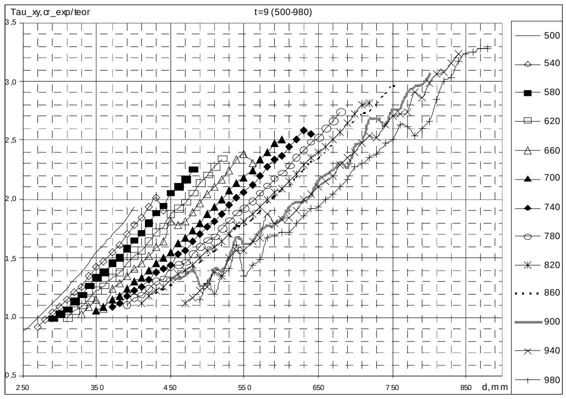


c)

Fig 7. Dependence of $\tau_{cr,teor}$ versus d: a – for t_w = 6 mm; b – for t_w = 9 mm; c – for t_w = 12 mm.



a)



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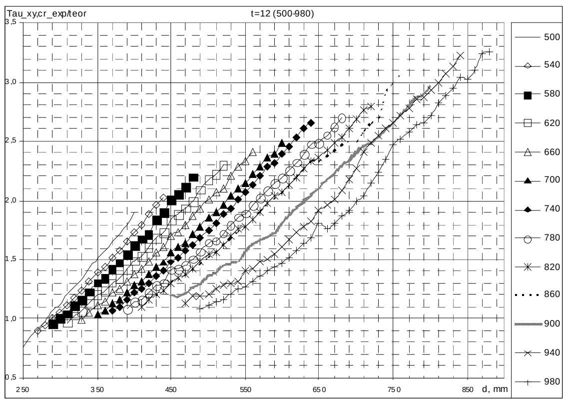

c)

Fig 8. Dependence of $\tau_{cr,exp}$ / $\tau_{cr,teor}$ ratios versus d: a – for t_w = 6 mm; b – for t_w = 9 mm; c – for t_w = 12 mm.

Obviously, the increasing of the web-post slenderness causes the decreasing of critical shear stresses $\tau_{cr,exp}$. Also it is observed, that increasing of the web opening causes decreasing of the critical shear stresses $\tau_{cr,exp}$, but not so marked as in case of $\tau_{cr,teor}$. For the webs with bigger slenderness the size of the web openings has smaller influence on the critical shear stresses $\tau_{cr,exp}$. From this, the conclusion may be drawn, that as bigger is the web slenderness as bigger opening may be presented in the web with very small influence on $\tau_{cr,exp}$. When the web thickness is increasing the critical shear stresses cr,exp is also increasing, but nonlinear. As thicker is the web as increase of the critical shear stresses $\tau_{cr,exp}$ is bigger.

4.2. Results of analysis using theoretical equations

Because of big amount of a data bellow are given the charts $\tau_{cr,teor}$=f(d) only for web thicknesses 6, 9, 12 mm, and with web depth 500÷980 mm, every 40 mm (Fig 7).

To see the critical shear stresses changes in different charts, the scale of ordinate has been taken the same.

From the charts above it is seen, that the critical shear stresses $\tau_{cr,teor}$ depends on web slenderness. Obviously, the increasing of the web-post slenderness causes the decreasing of critical shear stresses $\tau_{cr,teor}$. Also it is observed, that increasing of the web opening causes decreasing of the critical shear stresses $\tau_{cr,teor}$. For the webs with bigger slenderness the size of the web openings has smaller influence on the critical shear stresses $\tau_{cr,teor}$. From this, the conclusion may be drawn, that as bigger is the web slenderness as bigger opening may be presented in the web with smaller influence on $\tau_{cr,teor}$. When the web thickness is increasing the critical shear stresses $\tau_{cr,teor}$ is also increasing, but nonlinear. As thicker is the web as increase of the critical shear stresses $\tau_{cr,teor}$ is bigger. The charts $\tau_{cr,teor}$ =f(d) as distinct from $\tau_{cr,exp}$=f(d) are more smooth. The main differences between cr,exp and $\tau_{cr,teor}$ are described in the following chapter.

4.3. Comparison of the results obtained using theoretical equations and finite element method

Because of big amount of a data bellow are given the charts $\tau_{cr,exp}/\tau_{cr,teor}$=f(d) only for web thicknesses 6, 9, 12 mm, and with web depth 500÷980 mm, every 40 mm (Fig 8).

To see the $\tau_{cr,exp}/\tau_{cr,teor}$ ratios changes in different charts, the scale of ordinate have been taken the same.

From the charts above it is seen, that the increase of the web slenderness does not cause bigger discrepancy between theoretical and obtained using finite element method results.

From the charts it is seen if the web depth is almost the same the changes of the web thickness practically does not affect on the $\tau_{cr,exp}/\tau_{cr,teor}$ ratio. The results between theoretical and obtained using finite element method coincide quite well ($\tau_{cr,exp}/\tau_{cr,teor}$ ratio is equal near to unite) when the opening diameter d is equal half the web depth. When d value is increasing the $\tau_{cr,exp}/\tau_{cr,teor}$ ratio is increasing to, what shows a discrepancy in theoretical and finite element method results. When d is equal h_w-100 mm the critical shear stresses obtained using finite element method is about from 1.9 to 3.3 times bigger than obtained using analytical equations.

5. Conclusions

An analysis of castellated beams with hexagonal form of perforations using finite element method and analytical equations is accomplished in this paper. Some conclusions may be drawn:

1. When the diameter of the perforation is increasing, the reduction of the theoretical and obtained using finite element method critical shear stresses may be observed.
2. Critical shear stresses obtained using numerical experiment decreasing more than obtained using finite element method, when perforation diameter is increasing.
3. $\tau_{cr,exp}$ very similar to $\tau_{cr,teor}$ when diameter of the perforation is small and equal half the web depth.
4. When the diameter of the perforation is increasing, bigger discrepancy between the theoretical and finite element method results may be observed.
5. Critical shear stresses obtained using finite element method is up to 3.3 times bigger than obtained using analytical equations. Thus, webs of the beams calculated using analytical equations will be thicker and bigger amount of steel will be used for its production.
6. The coefficient, which may equalize results of theoretical and finite element method calculations, may be problem for further works.

References

1. Čižas A. Mechanics of materials. Mechanics of structures' elements. Vilnius: Technika, 1993. 408 p. (in Lithuanian).
2. Liu, C. H.; Chung, K. F. Steel beams with large openings of various shapes and sizes: finite element investigation. Journal of Constructional Steel Research, 59, 2003, p. 1159-1176.
3. Shanmugam, N.E.; Lian, V.T.; Thevendran, V. Finite element modelling of plate girders with web openings. Thin-Walled Structures, 40(5), May 2002, p. 443-464(22).
4. Hagen, N. Chr. On the shear capacity of steel plate girders with large web openings. Doctoral theses from Norwegian University of Science and Technology. 2004. 251 p.
5. Mohebkhah, A. The moment-gradient factor in lateral-torsional buckling on inelastic castellated beams. Journal of Constructional Steel Research, 60, 2004, p. 1481-1494.
6. Megharief, J. D. Behaviour of composite castellated beams. Thesis for the degree of master from McGill University, Montreal, Canada. May 1997. 162 p.
7. Zirakian T.; Showkati H. Distortional buckling of castellated beams. Journal of Constructional Steel Research, 62, 2006, p. 863 -871.
8. ENV 1993-1-1:1992/A2:1998. Eurocode 3: Design of steel structures – Part 1-1: General - General rules and rules for buildings.
9. Biruliov, V. V.; Koshin, I. I.; Krylov, I. I.; Silvestrov, A. V. Design of steel structures (Проектирование металлических конструкций). Leningrad: Strojizdat, 1990. 96 p. (in Russian).
10. SNiP II-23-81* Steel Structures. Design Code (Стальные конструкции. Нормы проектирования). Moscow: CITP Gosstroja SSSR, 1990. 96 p. (in Russian).

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