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Original Research Article

INFLUENCE OF THE RIBS PARALLEL TO WEB ON WARPING AND LOAD-BEARING CAPACITY OF A STEEL I-BEAM

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Abstract

The article analyses the method of enhancing a steel beam by adding additional steel members like ribs. They are rigidly connected with both flanges in a plane parallel to the web. That plates reduces warping during in-plane bending of steel beam under lateral-torsional bucking. Different thicknesses of steel plates used as ribs and different cross-sections were taken into account. Calculations were conducted using FEM and ABAQUS CAE environment. The outcomes were compared with ones from previous studies which concerned an influence of endplates on load-bearing capacity of an I-beam.

Keywords: warping, lateral-torsional buckling, endplates, ribs, bending

1. INTRODUCTION

The load bearing capacity of a steel I-beam is often determined by the lateral-torsional buckling (figure 1). This phenomenon depends largely on the resistance to a lateral bending, warping, relation between second moment of area for two major axes, boundary conditions and others. The lateral-torsional buckling is connected with a critical moment of bending. It describes the largest value of bending moment the geometrically and materially ideal beam should bear

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before out plane instability (without taking under consideration the load bearing capacity of a cross section). The bigger value of a critical moment is the more load can be applied to the beam. The slenderer the element is the greater risk of out of plane instability will be.

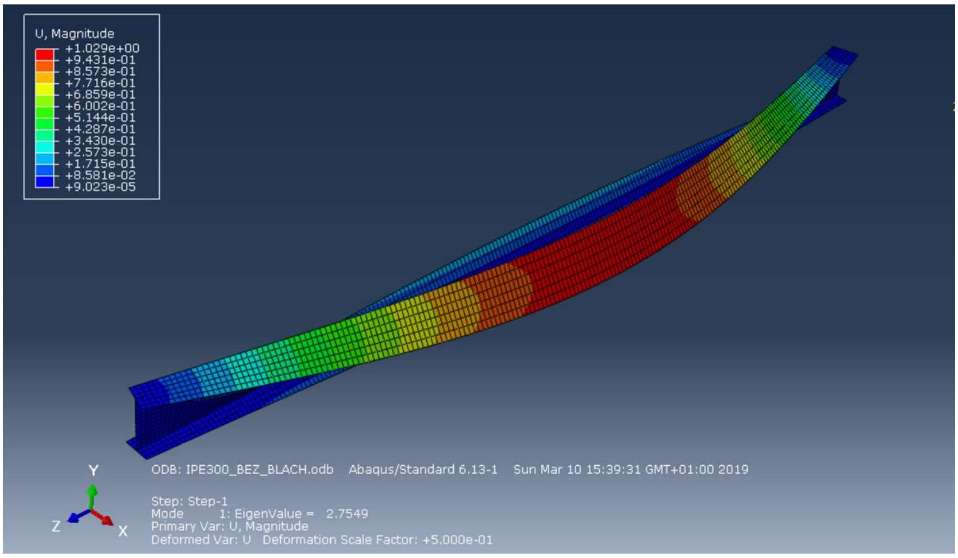

Fig. 1. Lateral-torsional buckling of IS beam in ABAQUS CAE – without any ribs

What is curious, in the European Standard 3 [1] there is no information how to calculate the critical moment of bending value (1.1). It can be found in the deco mmissioned version of that code [6] or other codes like European Standard for aluminium structures [2].

$$M_{cr} = C_1 \frac{\pi^2 \cdot E \cdot I_z}{(k_z \cdot L)^2} \left(\sqrt{\left(\frac{k_z}{k_w}\right)^2 \frac{I_\omega}{I_z} + \frac{(k_z \cdot L)^2 \cdot G \cdot I_T}{\pi^2 \cdot E \cdot I_z} + (C_2 \cdot z_g - C_3 \cdot z_j)^2} - C_2 \cdot z_g - C_3 \cdot z_j \right) \quad (1.1)$$

where:
M_{cr} – the critical moment of bending,
C_1, C_2, C_3 – factors depending on the loading and end restraint conditions,
k_z – the effective length factor of lateral bending,
k_w – the effective length factor of warping,
E – the Young's modulus,
G – the shear modulus,
L – the length of the beam,
I_ω – the warping constant,
I_T – the torsion constant,
I_z – the second moment of area about the weak axis,

z_g – the distance between the point of load application and the shear center about the strong axis,
z_j – the distance between the point of load application and the shear center about the weak axis.

What is more, there is a mathematical problem of including the warping effect in a general equation of critical moment [8]. It is a problem to adjust the warping in that equation because it is soluble only by trial and error method. That is why the warping effect is described by "k_w" factor which takes value between 0.5 and 1.0. It means respectively full fixation and pinned support for both ends of simply supported beam. It is possible to calculate the "k_w" factor using the Saint Venant's stiffness for instance for endplates (1.2 and 1.3) [6] and [9]. But there are solutions only for the most co mmon situations like endplates in the simply supported I-beam under evenly distributed load.

$$k_w = \mu_\omega = 0.5 + 0.14(\kappa_{\omega,1} + \kappa_{\omega,2}) + 0.055(\kappa_{\omega,1} + \kappa_{\omega,2})^2 \tag{1.2}$$

$$\kappa_\omega = \frac{1}{1 + \frac{s_{\omega,ini} L}{2EI_\omega}} \tag{1,3}$$

where:
$s_{\omega,ini}$ – the stiffness of an endplate,
t_f – the thickness of flanges,
h – the height of cross section of a beam,
b– the width of cross section of a beam,
t_p – the thickness of an endplate,
κ_ω – the coefficient of the stiffness of fixations on both ends,
$\kappa_{\omega,1}$, $\kappa_{\omega,2}$ – the coefficients of the stiffness of fixation - respectively in the left and right support.

According to previous research, to improve the resistance for out of plane instability using elements like endplates sometimes it is needed to connect the I-beam with unco mmonly thick ones like 40 mm or more. This is quite unusual for typical steel constructions where the cross sections of hot rolled steel members are up to 500 mm height and the endplates of these beams are about 20 – 30 mm thick.
The warping effect is caused by buckling of a compression part of an I-beam which rotates the whole cross section. To protect against mutual rotation of both flanges longitudinal ribs (parallel to the web) can be used. In fact, they are co mmonly used to make possible to connect I-beams with perpendicular elements like bracings or angle ties. In spite of their bigger stiffness against warping, they

are not taken under consideration during calculation the load bearing capacity of an I-beam. Therefore, in this paper the ribs, which co mmonly are not taken into consideration as reinforcement of beams, were analysed as an enhancement for load-bearing capacity of an construction element. Because, in fact, they are a part of a whole beam and makes it stronger.

2. MATERIALS AND METHODS

2.1. Geometry of the beam

A welded, steel I-beam IS-300/150/10.77.1 (welded equivalent for hot-rolled IPE300 without radiuses) were taken into consideration (figure 2). It is 5.0 m long and simply, fork supported on both ends. The load is evenly distributed on the centre of top flange with magnitude of 10kN/m. It is made of S355JR steel, but analyses were made only in an elastic state (stresses up to 355 MPa), so material was modelled with linear relationship between stress and strain (without plasticity).

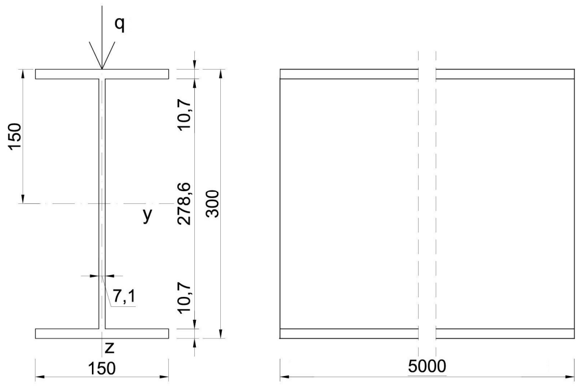

Fig. 2. Geometry of the beam without ribs

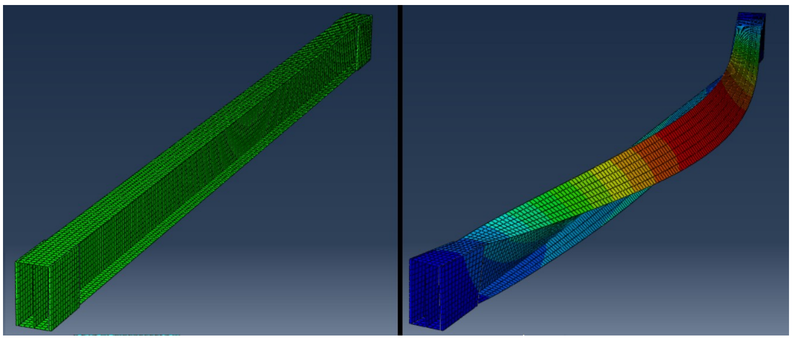

Fig. 3. Longitudinal ribs before and after buckle analysis in ABAQUS CAE

To enhance the load bearing capacity of bending few longitudinal ribs were taken into account. They were rigidly connected at the ends of the beam to both flanges at each side (figure 3). Their length is 150, 300, 450, 900 and 1350 mm and thick is from 5 mm to 60 mm.

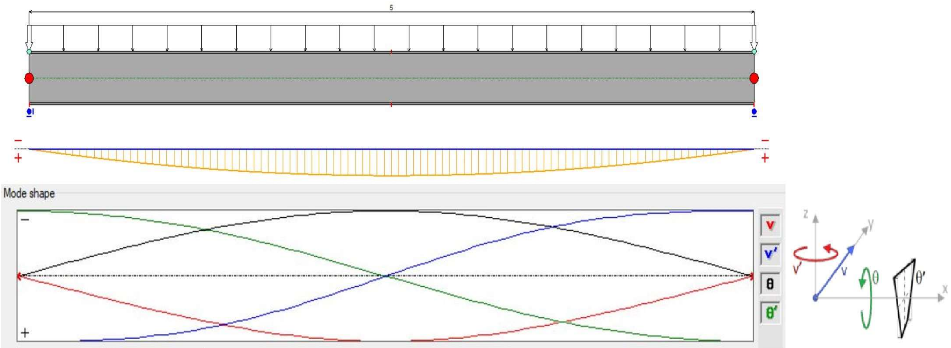

Fig. 4. Diagram of out of plane rotations and displacement for simply supported beam

It can be seen that the ribs are situated in the place of maximum value of mutual rotation between flanges (figure 4 – green line) – at both ends of the beam. The closer the middle point of the beam they are the minor influence of them for load-bearing capacity will be.

2.2. Methods

Calculations were conducted in ABAQUS CAE environment. The beam was created using 25 mm S4R shell elements. The longitudinal ribs (plates parallel to the web) were discretised using 25 mm S3D20R solid elements with reduced integration. The buckle analyses were carried out.

For comparison the LTBeamN application was used. Computations were carried on for a beam without any ribs and for those with rigid blockade of warping (θ') on the length and placement of longitudinal ribs at flanges – the same as in ABAQUS CAE model. There is no possibility to input those ribs directly to LTBeamN so it is necessarily to input it by fixing θ' degree of freedom at length 150, 300, 450, 900 and 1350 mm from each end of the beam.

3. RESULTS AND DISCUSSIONS

According to results (figure 5) there is a big influence of ribs which are parallel to web on critical moment of bending – even for the shortest and the thinnest steel plates like 5 mm. The longer the ribs are the bigger is their influence to simple bending too – that is why M_{cr} function of ribs thickness for 1350x300 mm is a way more ascending that in the previous ones with the shorter ribs. For the smallest length of steel plates there in an almost linear growth of critical moment per unit of thickness.

Table 1. Critical moments values for figure 5 [kNm]

	ribs 150x300 mm	ribs 300x300 mm	ribs 450x300 mm	ribs 900x300 mm	ribs 1350x300 mm
without ribs			86.1		
5 mm	105.6	131.5	160.6	257.9	403.0
10 mm	109.4	136.9	167.3	272.4	436.3
15 mm	114.8	143.5	174.1	283.2	461.0
20 mm	121.7	151.4	181.7	293.3	483.2
25 mm	129.1	159.5	189.5	303.0	504.3
30 mm	136.1	166.9	196.6	312.1	524.4
35 mm	142.0	173.1	202.6	320.3	543.2
40 mm	146.0	177.5	207.1	326.9	559.3
50 mm	152.7	184.2	213.8	337.7	587.8
60 mm	156.8	188.2	217.9	344.9	608.8

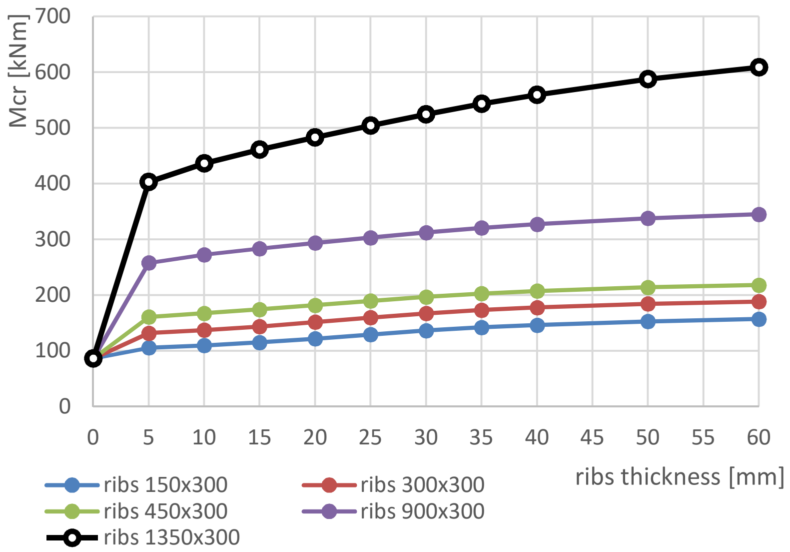

Fig. 5. Critical moment for different ribs connected to the beam

What is more, for the same dimension of steel plate there is a bigger growth of a critical moment value when plates are used as longitudinal ribs than endplates at both ends of a beam. An influence of endplates to critical moment was investigated in [3] and [4] or described in [5]. Only the endplates above 15 mm of thickness had the sufficient stiffness to make influence of the critical moment.

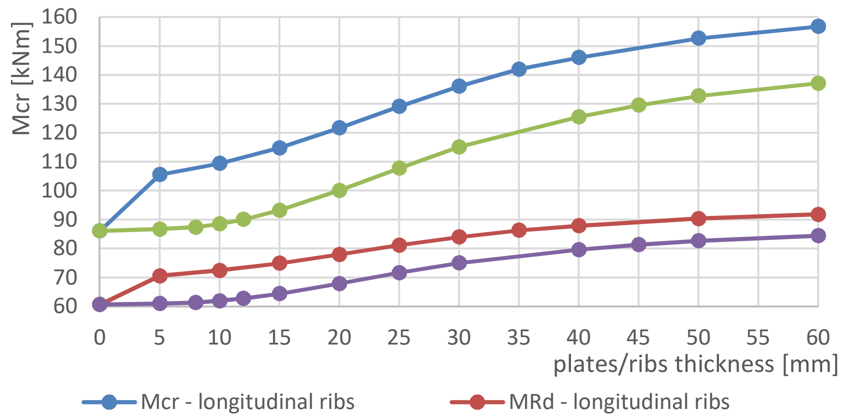

Fig. 6. Difference between the value of critical moment between ribs parallel to web and endplates

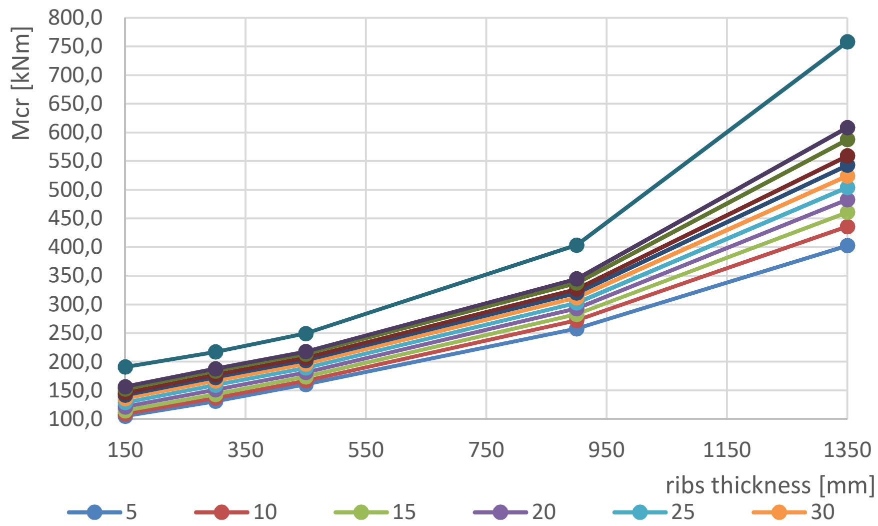

Fig. 7. Value of the critical moment for different thickness of longitudinal ribs

Analysed ribs, which are parallel to web, have bigger second moment of area for bending caused by the mutual rotation between planes than endplates. Even for ribs made of steel plates of 5 mm thick there is a significant growth of critical moment value (it is similar to case with 25 mm thick endplates). The endplates made of 5 mm thick members give a negligible change to the lateral-torsional buckling and – in the end – load bearing capacity for bending.

4. CONCLUSIONS

Enhancing a steel I-beam by adding steel ribs connecting both flanges causes a significant growth of the critical moment of bending and in fact the load bearing capacity of an element. The most important dimension of the ribs is the length of plates, because that parameter decides of the plates' stiffness according to a mutual rotation between the top and the bottom flange. The longer the ribs are the greater load bearing capacity will be, because adding the ribs partially converts an I-beam in a box-section element which is a way more resistant to lateral-torsional buckling than I-shape. It is important to design ribs thick enough to avoid local instability of their own, but that parameter is not critical to improve the load bearing capacity of bending.

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