Influence of defects of transverse stiffeners on the condition of bridge plate girders

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ABSTRACT: Bridge structures constructed of steel plate girders form a large and relatively old group of bridge infrastructure in Poland, subjected to various defects. A procedure of modelling and analysis of bridge plate girders with damaged stiffeners by means of Finite Elements Method (FEM) is presented in the paper. Assessment of load capacity and distribution of internal forces in intact and damaged structures is based on Linear Buckling Analysis (LBA) and Geometrically as well as on Materially Nonlinear Analysis with Imperfections (GMNIA). Results of the analyses performed for bridge plate girders of various geometry are presented and compared with the effects obtained for structures with damaged transverse stiffeners.

1 INTRODUCTION

In Poland, steel girders are used in about 50% of railway bridges and in about 20% of road bridges. The most popular types of steel girders applied in railway bridge superstructures are: steel plate girders (about 28%), truss girders (10%) and rolled beams (over 4%), as presented in Figure 1. Steel railway bridges are relatively old. The oldest structures were constructed over 150 years ago and about 45% of the bridge population is over 100 years old (Bień J., 2002).

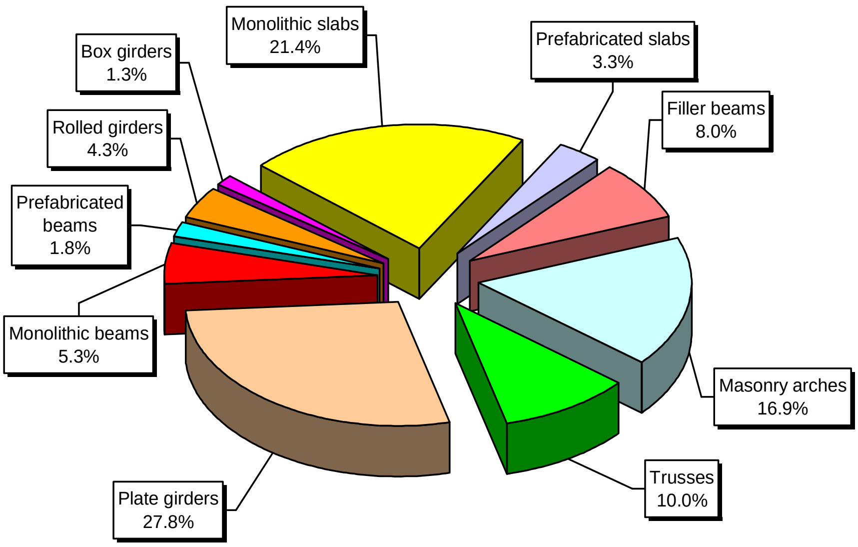

Figure 1. Basic types of main girders of railway bridge structures in Poland.

The most frequent defects of intermediate transverse stiffeners of bridge plate girders are material losses, as well as excessive deformations resulting from derailment of rolling stock or transportation of cargo exceeding the size of the loading gauge (for example, due to uncontrolled movements of the load during transport, see Figure 2). Such defects are usually dangerous and can lead to failure or even catastrophe.

Load capacity assessment of the bridge plate girders with defects of transverse stiffeners is a complex issue and can be analysed using a proper sequence of different types of Finite Elements Analysis (FEA): Linear Buckling Analysis (LBA) and Geometrically as well as Materially Nonlinear Analysis with Imperfections (GMNIA).

Results of nonlinear numerical analysis, presented in this paper, are mainly focused on the influence of typical defects of transverse stiffeners on the ultimate shear capacity of the investigated structures.

Figure 2. Typical defects of transverse intermediate stiffeners caused by the impact of vehicles crossing the bridge (Bień J., 2010)

2 DESIGN OF TRANSVERSE STIFFENERS ACCORDING TO EN 1993-1-5

When checking the buckling resistance, the effective section of a stiffener may be taken as the gross area comprising the stiffener plus the width of a plate equal to $15\varepsilon t_w$, but not more than the real dimension available – on each side of the stiffener avoiding any overlap of contributing parts to adjacent stiffeners, see Figure 3. Standard EN 1993-1-5 (2008) assumes that the transverse stiffeners provide a rigid support for a web panel with or without longitudinal stiffeners in order to be able to carry the load in different phases of work, up to the full development of tension field, which leads to achievement of the ultimate shear capacity of the girder.

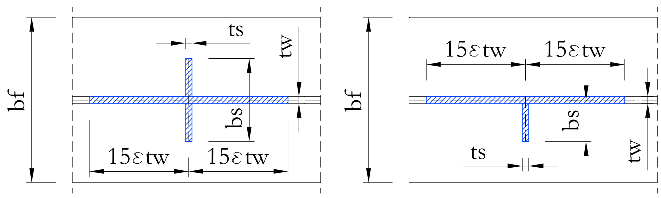

Figure 3. Effective cross-sections of stiffeners according to EN 1993-1-5 (2008)

To allow a rigid support for a web panel, intermediate transverse stiffeners should satisfy the criteria given below. The transverse stiffener should be analysed as a simply supported beam with an initial sinusoidal imperfection w_0 equal to min $(a_i,\ h_w)/300$, where a_i is the smallest adjacent panel length and h_w denotes the panel height, as indicated in Figure 4.

The transverse stiffener should carry the deviation forces from the adjacent compressed panels on the assumption that both adjacent transverse stiffeners are rigid and straight, as well as they have to be able to withstand possible external loads and axial force equal to

$$V_{Ed} - V_{cr} = V_{ED} - \frac{1}{\bar{\lambda}_w^2} f_{yw} h_w t_w / \left(\sqrt{3}\gamma_{M1}\right), \tag{1}$$

where: V_{Ed} – design transverse load in analysed panel, V_{cr} – elastic buckling transverse force in analysed panel, $\bar{\lambda}_w$ – relative slenderness of panel in shear, γ_{M1} – partial factor applied to checking of stability phenomena.

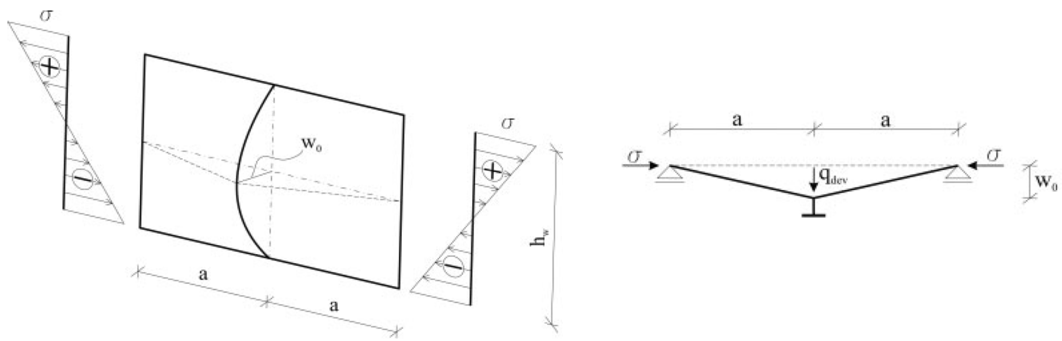

Figure 4. Intermediate transverse stiffener computational model according to EN 1993-1-5 (2008)

Using a second order elastic method analysis, it should be verified that both of the following criteria are satisfied at the ultimate limit state: the maximum stress in the stiffener should not exceed fy/γ_{M1} and the additional deflection should not exceed $h_w/300$. Unless a more advanced method of analysis is carried out, in order to prevent torsional buckling of stiffeners with open cross-sections, the following criteria should be satisfied:

$$\frac{I_T}{I_o} \geq \frac{f_y}{E}, \tag{2}$$

where I_T is the St. Venant torsional constant for the stiffener alone and I_0 is the polar second moment of area of the stiffener.

The effective section of intermediate stiffeners, acting as rigid supports for web panels, should have a minimum second moment of area, $I_{s,min}$:

$$I_{s,\min} \geq 1.5 h_w^3 t_w^3 / a^2 \quad \text{for} \quad a / h_w < \sqrt{2}, \tag{3}$$

$$I_{s,\min} \geq 0.75 h_w t_w^3 \quad \text{for} \quad a / h_w \geq \sqrt{2}. \tag{4}$$

3 PROCEDURE OF GMNIA ANALYSIS

The design procedure of intermediate stiffeners of plate girders according to EN 1993-1-5 (2008) does not allow for a precise stress (effort) evaluation of the individual sections of a stiffener, as well as a precise stability assessment of the entire element. The situation is significantly more complicated in the case of bridge plate girders with damaged transverse stiffeners.

To solve the problem, a procedure for determining the effect of transverse stiffeners defects on the ultimate shear capacity of a plate girder as well as on the entire stiffener component stability, has been proposed. The procedure utilizes the results of the geometrically as well as materially nonlinear analysis with imperfections (GMNIA) combined with the general rules applied in design of beam elements subjected to bending and compression according to the code EN 1993-1-5. Performed numerical analyses made it possible to evaluate the structure components in all phases of work – elastic, nonlinear plastic-buckling, ultimate capacity and collapse, including geometric imperfections applied to individual girder elements and assumption of elastic-plastic bilinear material model with isotropic hardening.

The general methodology (Kużawa & Bień, 2012) for analysis of bridge plate girders in subsequent phases of load until failure, is presented in Figure 5. FEM numerical models for the LBA and GMNIA analysis, performed in Abaqus system, were constructed of shell elements, type S4R – 4 node, six degrees of freedom at each node, with a reduced integration scheme (Chróścielecki J., 2004). The load was modelled using kinematic excitation, realized by means of coupling ties offered by Abaqus system (SIMULIA, 2010). Computations were performed using Newton–Raphson iterative algorithm and Huber-Mises yield criterion was employed for the numerical simulations, leaving aside the issues related to the strain velocity (viscosity).

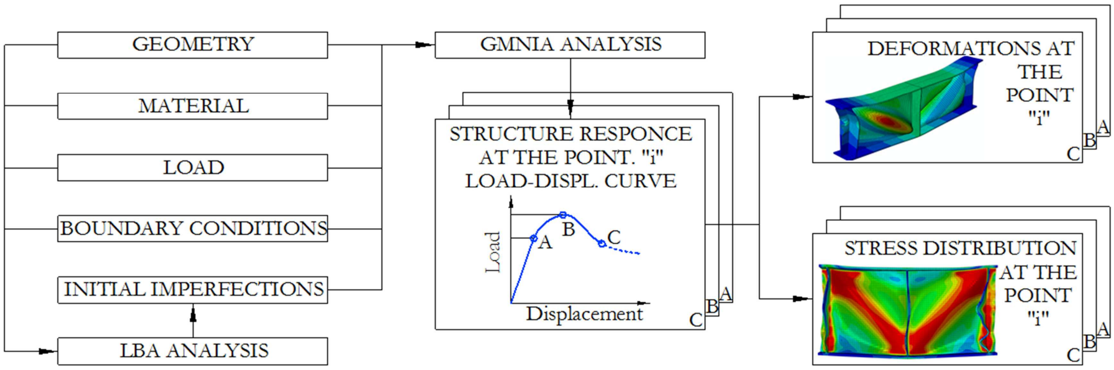

Figure 5. Procedure of bridge plate girders analysis in subsequent phases of load (Kużawa & Bień, 2012)

4 SELECTED RESULTS OF GMNIA ANALYSIS OF INTACT GIRDERS

More than thirty simply supported 6 m long plate girders, with different geometrical parameters: h_w, t_w, t_f, b_s and t_s (Table 1), were analysed. General girder geometry and denotations of above mentioned dimensions are shown in Figure 6. Numerical models of the structure were evaluated, by means of GMNIA analysis, using kinematic enforcement *U2*, as shown in Figure 6. Entire value of kinematic enforcement *U2* = 20 mm was obtained in approximately 40 steps for each model. The applied procedure enables an analysis of the global structure stability, as well as of the individual plate girder elements in particular characteristic phases of work: elastic, nonlinear plastic-buckling, ultimate capacity and collapse. Selected results of the analysis of intact structures are presented according to slenderness parameter h_w/t_w, modes of preliminary geometrical imperfections and modes of collapse.

Initial imperfections, applied in GMNIA analysis, were set as a superposition of various shapes of modes obtained by means of Linear Buckling Analysis (LBA), indicated in Figure 7.

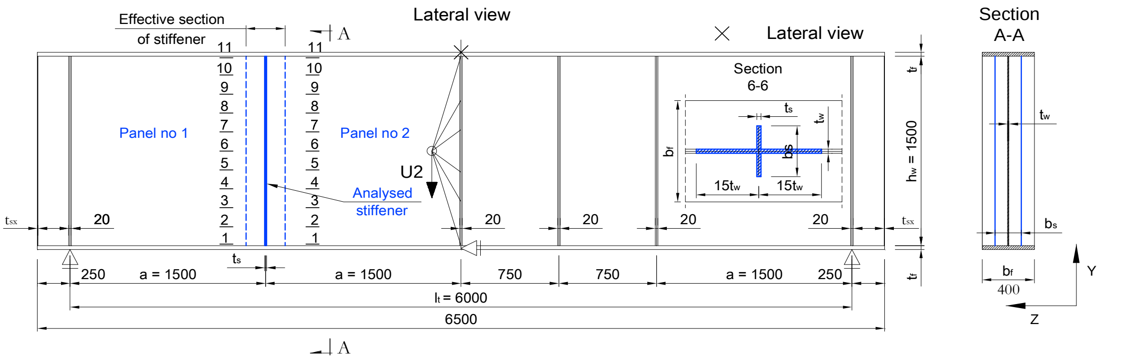

Figure 6. Geometry of analysed structure [mm]

Table 1. Geometrical parameters of analysed structures

$h_w = a$ [mm]	h_w/t_w [-]	t_f/t_w [-]	b_s [mm]	t_s/t_w [-]
1500	[100, 125, 150, 185]	4	[100, 200, 300, 400]	[0.5, 0.8]

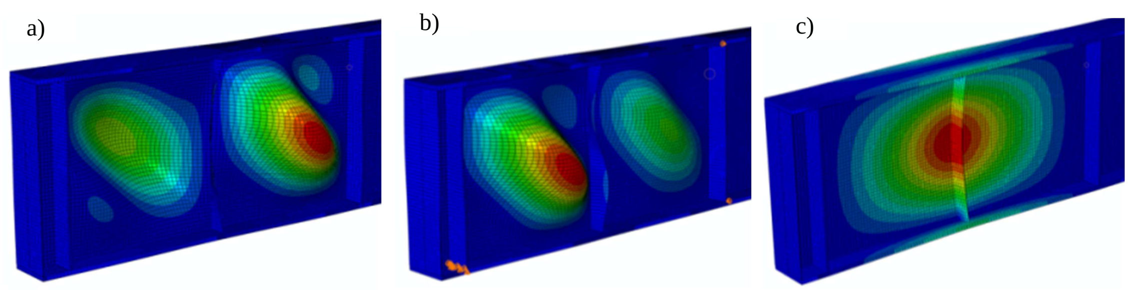

Figure 7. Selected components of initial deformations of girders: a) mode 1, b) mode 2, c) mode 3

Load-displacement curves, for models with different slenderness parameter h_w/t_w, are presented in Figure 8. The lowest ultimate load was obtained for structures having the intermediate transverse stiffeners width b_s = 100 mm and slenderness parameter h_w/t_w = 185. For girders with parameters h_w/t_w = 185, the ultimate load value was lower than the average ultimate load – about 9.38 % for t_s/t_w = 0.5 and 2.14 % for t_s/t_w = 0.8. Among all other models (b_s > 100 mm), the maximum difference in the calculated ultimate load value for particular girders did not exceed 1.5 %. In collapse phase, it was observed that the decrease in the value of limit load along the increase of kinematic enforcement is overlooked and intermediate transverse stiffeners, despite many local deformations, are able to carry the load resulting from tension fields, being developed in adjacent panels. For models having a stiffener width b_s = 100 mm, decrease in the ultimate load is much more rapid in comparison with other models, because of a global loss of stability of the intermediate stiffeners.

Deformation and stress distribution for girders with parameters b_s = 300 and h_w/t_w = 150 were analysed taking into account: combination of mode shapes no 1 and 3 of initial deformations (model A) and mode shapes no 2 and 3 (model B) − see Figure 7. Deformation and stress distribution for models A and B during collapse are presented in Figure 9.

Distribution of axial force in the effective cross-section of analysed stiffeners, in particular load phases, for model A and B, are presented in Figure 10. Distribution and values of internal forces depend on the slenderness parameter h_w/t_w, as well as on the level of development of tension field in both analysed panels (stress distribution and deformations) determined by mode shapes of initial imperfections. In the collapse phase, for the initial deformations directed equally in panels no 1 and 2, the obtained values of axial forces are almost two times higher than for the initial deformations oppositely directed in those panels.

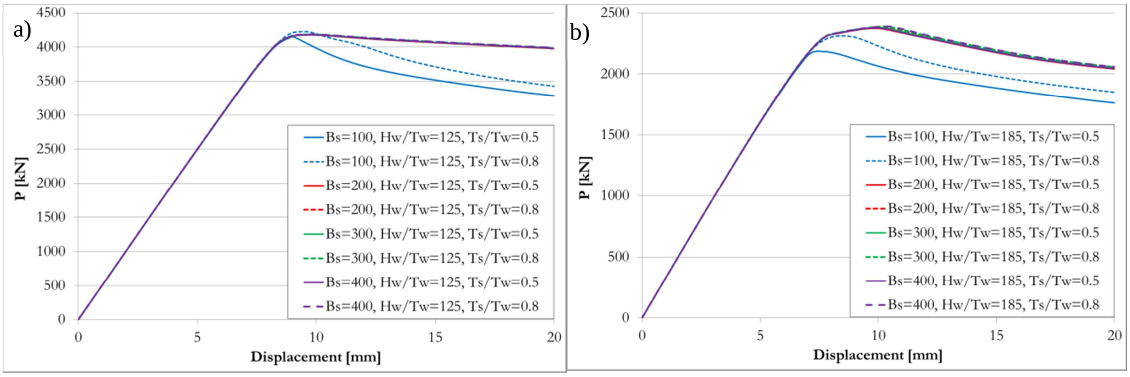

Figure. 8. Load-displacement curves for models with different slenderness parameter: a) h_w/t_w = 125, b) h_w/t_w = 185

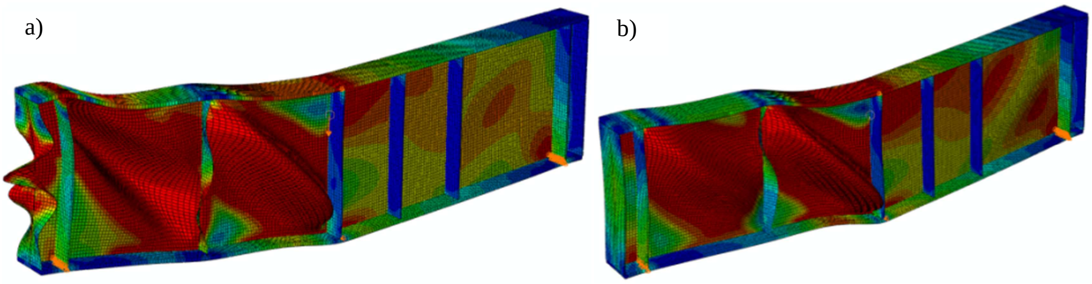

Figure. 9. Deformation and Mises stress distribution for models A (a) and B (b) with parameters b_s = 300 and h_w/t_w = 150

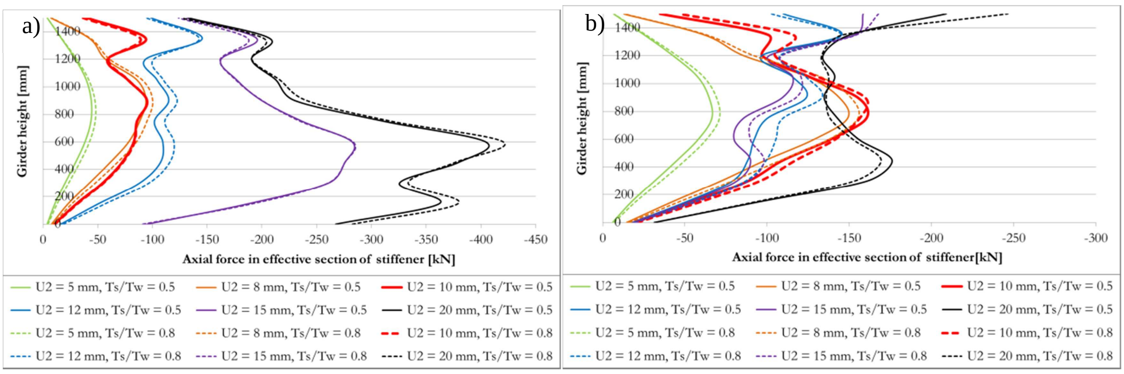

Figure. 10. Distribution of axial force in effective cross-section of analysed stiffener in particular load phases for: a) model A, b) model B

In Figure 11, distribution of bending moments with the vector parallel to the longitudinal axis of the girder, in particular load phases for models A and B, is shown. For model A – having equally directed deformations implemented in panels no 1 and 2, bending moment distribution has the shape of full sine wave. For model B – having an oppositely directed deformations implemented in panels no 1 and 2 − the distribution has the shape of 1.5 sine wave and extreme bending moments are located near the middle of the girder height. In comparison with model B, the maximum value of the bending moment in the stiffener effective section of model A is almost two times higher in the ultimate load phase. It should be taken into account that bending moments are crucial internal forces in terms of capacity utilization of transverse stiffeners of bridge plate girders, and the applied system of initial geometric imperfections for the analysed area of structure is a key issue for obtaining the unfavourable load case.

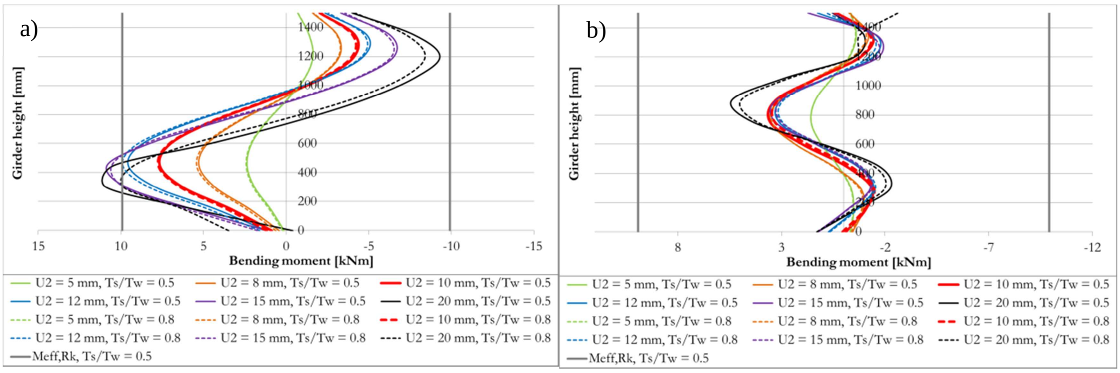

Figure. 11. Distribution of bending moment in effective cross-section of analysed stiffener in particular load phases for: a) model A, b) model B

5 SELECTED RESULTS OF GMNIA ANALYSIS OF DAMAGED GIRDERS

A comparison of selected results of the analysis of intact and damaged structure with material losses of the transverse stiffener (Figure 12a) is discussed below. Applied discretization of girder with the presence of defects, initial geometric imperfection in panel no 1 and 2, as well as collapse mode shape are shown in Figure 12.

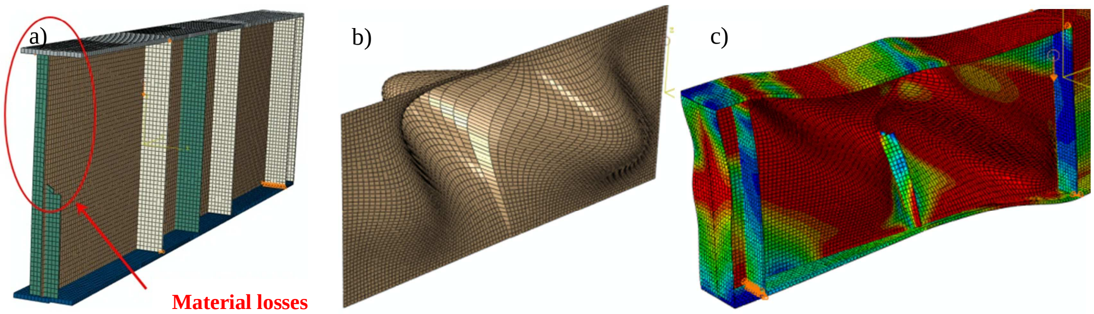

Figure. 12. Discretization of damage structure (a), initial geometric imperfections in panel no 1 and 2 (b) deformation and Mises stress distribution in collapse phase (c)

The internal forces in effective cross-section of intermediate transverse stiffener for intact and damaged structure, in subsequent load phases, are compared in Figure 13. In the ultimate load phase, the maximum values of internal forces for the damaged structure are on average twice as high as for the intact girder. The ultimate load capacity obtained for structure with the presence of the considered material losses in the intermediate transverse stiffener is 6.16 % lower than the ultimate load capacity obtained for the intact structure.

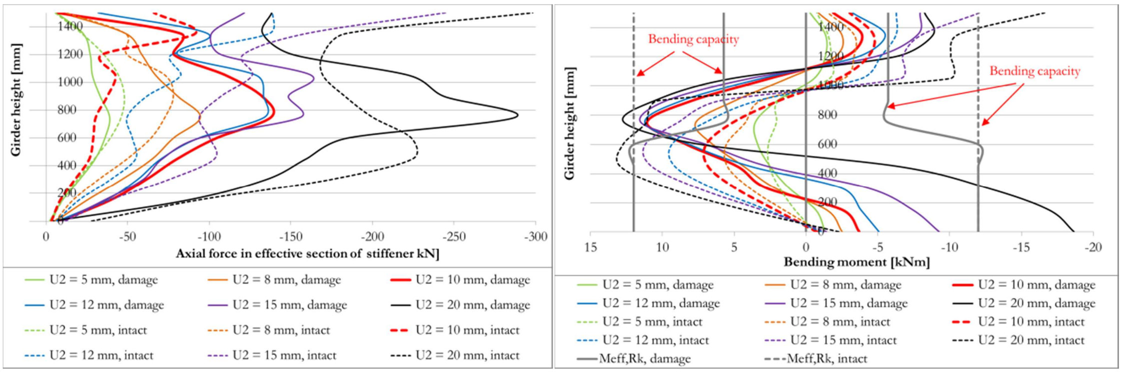

Figure. 13. Distribution of global internal forces in transverse stiffener effective cross-sections in particular load phases

6 SUMMARY

Based on the obtained results, it can be concluded that the proposed procedure enables a precise assessment of the ultimate load capacity of plate girder spans, taking into account possible defects of intermediate transverse stiffeners. The developed methodology can be used in the bridge management process to prevent construction failure and unreasonable replacement of bridge structure due to defects arriving during its operation. Presented algorithm can be also applied to acquisition of the knowledge and creation of the specific knowledge base, forming a background for expert tools supporting the load capacity assessment of bridge plate girders with defects.

7 ACKNOWLEDGEMENT

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