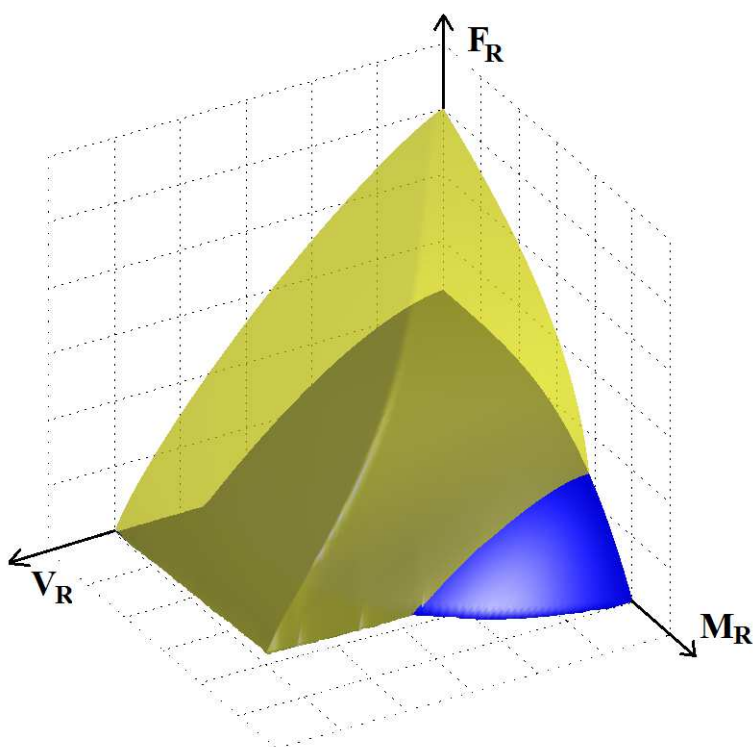




INTERACTING STABILITY BEHAVIOUR OF STEEL I-GIRDERS WITH CORRUGATED WEBS

Master's Thesis in the MSc in Computational Structural Engineering



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ABSTRACT

Corrugated sheets have been used for a long time in geotechnical engineering and for roof covering, but corrugated web girders have been started to use within the previous 30 years in industrial buildings and bridges, especially in composite bridges. During incremental launching of such structures the girder is subjected to the combination of different actions as bending moment, shear force and transverse force which result in a complex stress field in the girder and may occur interacting instability problem. Previous research activities related to the girders with corrugated webs showed that the structural behaviour of the corrugated web girders is different from the conventional I-girders, especially in the stress distribution in the flanges and in the web. On the other hand the stability behaviour of the corrugated web girders are also different from the flat web girders due to the corrugation profile. In spite of this fact there is a lack of investigations analyzing the effect of the complex loading situation. There is just a small number of previous investigations dealing with the combined loading situations related to corrugated web girders. Therefore the current diploma thesis focuses on the investigation of the structural behaviour of the girders with corrugated webs under all of the aforementioned actions and their combinations. Based on the experimental investigations from the international literature numerical models are developed and the structural behaviour of the girders with corrugated web is analysed under the complex loading situation due to the combination of bending moment, shear and transverse forces.

1. Introduction

Due to its advantages properties of corrugated sheets it has been used for a long time in geotechnical engineering and for roof covering. Corrugated web girders have been started to use within the previous 30 years in industrial buildings and bridges, especially in composite hybrid bridges in which the upper and lower flanges of the bridge deck are made of prestressed concrete however the webs are made of corrugated steel plates. During the construction stages, particularly in case of incremental launching of such structures the girder is subjected to the combination of different actions as bending moment, shear force and concentrated transverse force which result in a complex stress field in the structure what can lead to an interacting instability problem. Fig. 1 demonstrates the critical construction stages during launching. Figure 1 a) shows the most critical one when the superstructure is just before reaching the next pier and a large cantilever bending moment acts on the structure with accompanying large shear and transverse force at the previous pier. The other critical stage when the launching nose is moved through the pier and the superstructure gets above the pier. In this case the structure is suddenly subjected to a large shear force with an accompanying transverse force.

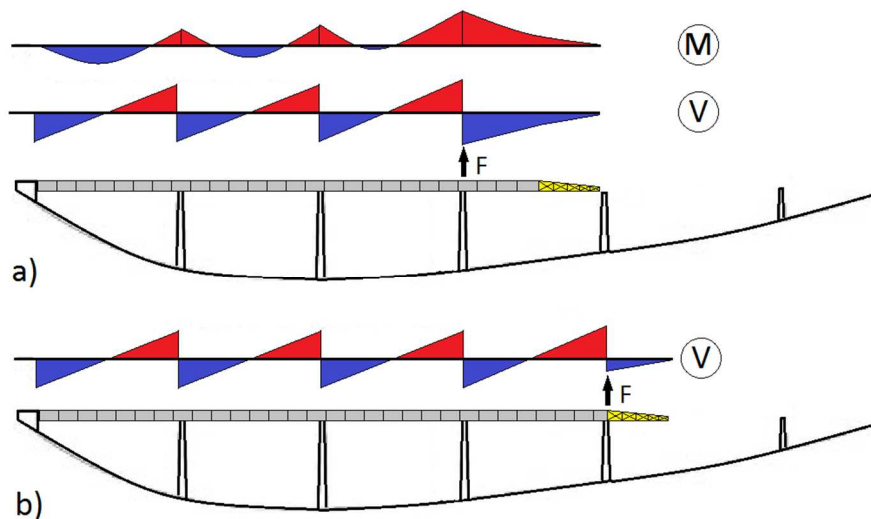


Fig. 1: Critical stages during incremental launching.

Previous research activities related to the girders with corrugated webs showed that the structural behaviour of the corrugated web girders is different from the conventional I-girders, especially in the stress distribution in the flanges and in the web. The local and global stability behaviour of the whole girder is also different due to the corrugation profile. In spite of this fact there is a lack of investigations analyzing the effect of such a complex loading situation. There is just a small number of papers dealing with the combined loading situations related to corrugated web girders. Therefore the current diploma thesis focuses on the numerical investigation of the structural behaviour of the girders with corrugated webs under all of the above mentioned actions and their combinations.

It has to be mentioned that there are many questions to be answered in case of corrugated web girders which is discussed in Section 2 with pointing out the incompleteness of the

investigations. The strength, stability and fatigue problems regarding to corrugated web girders are discussed separately and finally the evaluation is given with emphasizing the future and current aims.

A short outlook on the interacting stability behaviour of flat web girders is attached to this diploma thesis in Appendix C. Several researchers observed in the past that the corrugated web girders have several advantages in comparison with flat web girders which is an important statement because it does always have to be highlighted that the use of any structural layout is reasonable or not.

The notation system using in this diploma thesis regarding to corrugated web girders can be seen in Fig. 2.

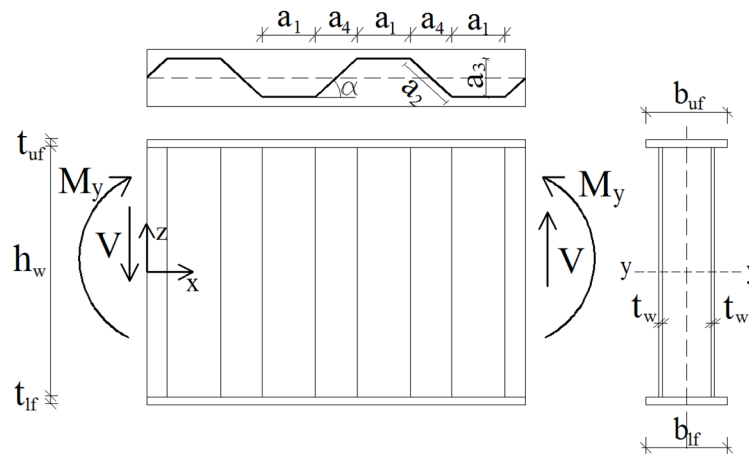


Fig. 2: Geometry of a girder with trapezoidally corrugated web.

Based on the information coming from the international literature on the topic of corrugated web girders a research program was prepared in Subsection 3.4 to execute the investigation precisely on the interacting behaviour of corrugated web girders subjected to combined bending moment (M), shear (V) and transverse force (F) and a detailed literature review was conducted on this topic in Section 3.2. It has to be mentioned that the number of corresponding papers in the literature is slightly more than ten which is a very low number and only one recommendation can be found related to the M - V interaction of corrugated web girders. Design method given in the EN1993-1-5 [1] Annex D was found to be too conservative by Kövesdi et al. [58]. Advanced numerical models were developed and verified in Section 4 on the basis of experiments found in the literature considering initial geometric imperfections and residual stresses and then the parametric study was executed to understand the interacting behaviour of corrugated web girders and to propose a possible interaction formula for this combined loading situation, namely M - V , M - F , V - F (Section 6) and M - V - F (Section 7) interactions, respectively. Finally comparison was taken place with other proposals found in the literature and statistical analysis was performed to determine the correctness of the new interaction equations.

2. Behavioural aspects of corrugated web girders

Due to the appearance of a new structural layout as in case of girders with corrugated webs a new structural behaviour can be observed. Many questions to be answered came out due to corrugated web and new investigations became necessary for better understanding and for standardization. The problems regarding to corrugated web girders are strength, stability and fatigue problems, respectively. In the international literature numerous papers have been published dealing with corrugated web girders. The current section is focusing on the previously analyzed structural behaviours related to corrugated web girders and discussing the unexamined problems up to now which have to be solved in the future. Thus it is useful as a starting point for the further investigations.

2.1. Stress distributions

Numerous experimental and numerical investigations have been done in the field of stress distribution and strength capacity of the web and flange of corrugated web girders. At first the bending moment resistance of corrugated web girders was investigated experimentally by Linder [2] in 1992 and continued by Aschinger and Linder [3] in 1997. It was stated that the bending moment resistance can be calculated from the contribution of the flanges considering the effect of the additional transverse bending moment in the flanges coming from the shear flow in the web. Thus the normal stress distribution in the flanges is nearly linear and the shear stress is considered as constant along the web depth. The first proposal for the calculation of the transverse bending moment was developed by Aschinger and Linder. Based on numerical results the stress distributions in the flanges and in the corrugated web was analyzed in detail by Elgaaly et al. [4] in 1997. It was presented that the ultimate moment carrying capacity can be calculated only from the contribution of the flanges and the shear force is carried only by the corrugated web and concluded that there is no interaction between them. A significant research programme was executed on the stress distribution in the flanges of corrugated web girders by Abbas et al. [6, 7, 8] in 2006 and 2007. The determination of the transverse bending moment was one of the scope of the programme. Further investigations were executed on the determination of the maximum transverse bending moment in the flanges by Balaz [9] in 2012 and Kövesdi et al. [10] in 2012. It was recognized that the loading conditions and the location of the supports in different positions compared to the corrugated web have a significant effect on the maximum transverse bending moment, thus an analytically derived calculation method was proposed for the most unfavorable situation by Kövesdi et al. The calculation analogie for different corrugation layouts can be seen in Fig. 3.

The normal stress distribution in the web was analyzed by Elgaaly et al. [4, 5] in 1997 and 1998. It was established that the normal stresses are almost zero along the web depth, merely in a small part adjacent to the flanges arise. Thus it was concluded that the contribution of the web to the bending moment or axial force carrying capacity is negligible. This phenomenon is the so called “accordion effect” which was confirmed by Huang et al. [11], Mori et al. [12], Hannebauer [13] and Kövesdi et al. [10] and it is implemented in the design formula of the EN1993-1-5 [1] Annex D regarding to corrugated web girders.

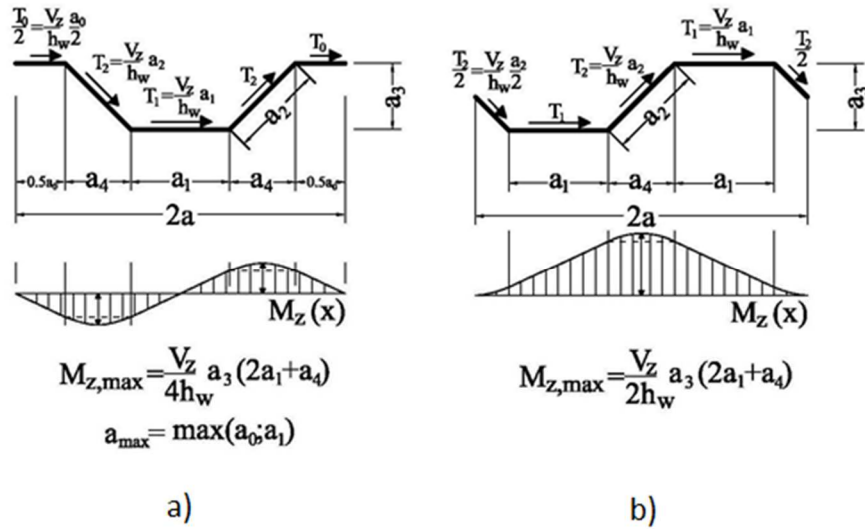


Fig. 3: Analytical formulas for the maximum transverse bending moment [9, 10].

2.2. Stability behaviour of the structural members

In case of thin-walled structures the failure modes are usually the loss of stability of a structural member or the combination of structural members or the whole structural element. By having a flat web I-girder the loss of stability may occur due to patch loading failure of the flange and web, the shear buckling failure of the web, the bending failure of the web considering the local flange buckling or the combination of these failure modes. In case of corrugated web girders the loss of stability can not happen due to the bending failure of the web because of the aforementioned “accordion effect” that the contribution of the corrugated web to the bending moment carrying capacity is negligible, thus the mention of bending failure due to the corrugated web is not relevant.

Patch loading resistance

The research on the patch loading resistance of corrugated web girders was started by Aravena and Edlund [15] in 1987 with experimental research program. The patch loading resistance was discussed theoretically based on experimental results of corrugated web girders by Kähönen [16] in 1988, thus a new design model (Eq. (1)) was published for the first time. The developed model is the improved version of the model for flat web girders which was developed by Rockey and Roberts [14] in 1979 in which the contribution of the web and the flange to the patch loading resistance is separated.

$$R = R_w + R_f \quad (1)$$

where $R_w = f_{yw} \cdot t_{wep} \cdot s_s \cdot k_w$ contribution of the web, (2)

$$R_f = 2 \cdot \sqrt{4 \cdot M_{pl,f} \cdot t_{wep} \cdot f_{yw}}$$
 contribution of the flange, (3)

where f_{yw} yield strength of the web,
 t_{wep} effective web thickness,
 s_s loading length,

k_w shear stress interaction factor,
 $M_{pl,f}$ plastic moment calculated from the flanges.

In 1997 the first numerical model was developed by Elgaaly and Seshadri [17] verified with experimental test results and a parametric study was executed with different geometric and material parameters. On the basis of their results a new design formula was proposed by having the observed failure modes as web crippling and web yielding. The proposed formula can be also decomposed into the contribution of the web and flange parts separately. Luo and Edlund [80] executed an additional numerical investigation using non-linear imperfect FE-analysis in 1996. Different material models, geometric and loading parameters were applied and studied in detail. It was detected that the most influential parameters are the loading length and the corrugation angle, therefore a new empirical resistance formula was developed based on design formula for flat webs by taking the most affected parameters into account with major importance.

$$P_u = 2 \cdot \sqrt{4 \cdot M_{pl,f} \cdot \chi \cdot t_w \cdot f_{yw} + f_{yw} \cdot \chi \cdot t_w \cdot s_s} \quad (4)$$

where χ reduction factor.

$$a_i \geq \left(\frac{h_w}{t_w} + 260 \right) \cdot \frac{t_w}{11.5} \text{ with } \alpha > 15^\circ \quad (5)$$

where a_i smallest web fold width,
 h_w web depth,
 t_w web thickness,
 α corrugation angle.

All of the aforementioned investigations on the patch loading resistance are focused on geometric parameter domain of frame structures. Therefore an extended numerical parametric study was executed by Kövesdi et al. [18] in 2008 in order to investigate the geometric parameter range used in bridge structures which is significantly important for an incrementally launched superstructure. The modified version (Eq. (4)) of the design proposal of Kähönen fits best into the numerical results due to long loading lengths and the global buckling of the web was prevented by a limit function (Eq. (5)) of the individual fold lengths proposed by Kuhlmann et al. [19] in 2008. It has to be emphasized that the new proposal considers only the pure patch loading resistance without any interaction factor of Kähönen. To harmonize the method with the EC principle for buckling analysis, new buckling curve was derived and the significant effect of the corrugation angle was included in the standardized formula. The applicability of the derived formula for bridge structures was confirmed by Kövesdi et al. [20] in 2010 and the sensitivity analysis of the influence of the initial imperfections was studied in detail by Kövesdi et al. [21]. It is important to note that there is no design formula recommended for the determination of the pure patch loading resistance in the EN1993-1-5 [1] standard for corrugated web girders.

Shear buckling resistance of the corrugated web

Numerous papers are dealing with the shear buckling resistance of the corrugated web girders. Due to the large number of research investigations just the main steps are discussed in this section. The first investigation was done by Easley and McFarland [22] in 1969. The suggested formulas are based on the orthotropic plate theory and the Ritz energy method and it is applicable only for the determination of the global shear buckling resistance of corrugated webs. Further buckling formulas were discussed related to corrugated plates by Easley [23] in 1975. It was stated that the failure modes may be governed by local or global shear buckling which can be calculated using isotropic (Eq. (6)) and orthotropic (Eq. (7)) plate buckling theory, respectively. After that several experimental, numerical and theoretical investigations have been performed in this field mostly dealing with the interaction between local and global shear buckling (Fig. 29 in Section 5.3). The existing formulas for the determination of the elastic local and global critical shear stresses are the followings.

$$\tau_{cr,L}^E = k_L \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{a_{max}} \right)^2 \quad (6)$$

$$\tau_{cr,G}^E = k_G \frac{\sqrt[4]{D_y D_x^3}}{h_w^2 t_w} \quad (7)$$

where

- k_L the local buckling coefficient,
- k_G the global buckling coefficient,
- E the Young's modulus,
- ν Poisson's ratio,
- t_w thickness of the web,
- a_{max} maximum fold width of the web panels,
- D_x longitudinal bending stiffness per unit length,
- D_y transverse bending stiffness per unit length.

However, the failure may be caused by the interaction between local and global shear buckling which is called interactive shear buckling strength and investigated at first by Leiva [24] in 1983 and Bergfelt et al. [25] in 1984. A first order (n=1) interactive formula was proposed (Eq. (8)) in which the material inelasticity and yielding are not considered.

$$\left(\frac{1}{\tau_{cr,I}} \right)^n = \left(\frac{1}{\tau_{cr,L}^E} \right)^n + \left(\frac{1}{\tau_{cr,G}^E} \right)^n \quad (8)$$

In 1988 Linder and Aschinger [26] proposed the same interactive formula but with n=2 as second order approximation and suggested a 70% limit for the nominal shear strength (reduction factor) for design. After that several authors have confirmed the applicability of the proposed interactive formula but with different orders. Elgaaly et al. [27] in 1996 suggested firstly to take the local and global inelastic critical stresses explicitly into account in the first order interactive formulae (Eq. (9)). This was confirmed by Luo and Edlund [28] in 1996, Johnson and Cafolla [29] in 1997 and Galambos [31] in 1998. In addition Johnson and Cafolla

proposed a design formula for the determination of the shear buckling stress of the corrugated profile.

$$\left(\frac{1}{\tau_{cr,I}}\right)^n = \left(\frac{1}{\tau_{cr,L}^{INE}}\right)^n + \left(\frac{1}{\tau_{cr,G}^{INE}}\right)^n \quad (9)$$

where $\tau_{cr}^{INE} = \sqrt{0.8\tau_y \tau_{cr}^E} \leq \tau_y$ is the inelastic critical stress, (10)

$\tau_y = \frac{f_y}{\sqrt{3}}$ is the shear yield stress using von Mises yield criterion, (11)

f_y uniaxial yield strength of the material.

The same interaction equation considering the inelastic behaviour was proposed by Abbas et al. [35] in 2002 with $n=2$, Hiroshi et al. [39] in 2003 with $n=4$ and Driver et al. [44] with $n=2$. Abbas et al. concluded that the initial imperfections have a significant effect on the shear buckling capacity and it was confirmed by Driver et al. [44] and recommended that the amplitude of the initial imperfection may be taken as the thickness of the web as a realistic imperfection magnitude and suggested a shear design criteria for bridges by giving a new relationship between the normalized shear strength and the buckling slenderness ratio ($\rho-\lambda_s$),

where $\rho = \frac{\tau_{cr}}{\tau_y}$ is the normalized shear strength, (12)

$\lambda_s = \sqrt{\frac{\tau_y}{\tau_{cr}}}$ is the buckling slenderness ratio. (13)

In 2008 Yi et al. [46] studied the nature of the interactive shear buckling of corrugated webs, and concluded that the first order ($n=1$) interactive shear buckling equation in which the material inelasticity and material yielding are not considered (Eq. (8)) provides a good estimation of the shear strength of corrugated steel webs. In addition a new proposal for the relationship between $\rho-\lambda_s$ was provided. Moon et al. [47] presented in 2009 that the observation of Yi et al. was appropriate regarding not to consider the inelastic behaviour in the first order ($n=1$) interactive formulae (Eq. (8)), nevertheless a new proposal for the relationship between $\rho-\lambda_s$ was developed with a condition to maximize the shear buckling strength capacity without any shear buckling based on first order interactive shear buckling curve. Moon et al. also stated that the initial imperfections have a significant effect and the measured geometrical imperfections from specimens were presented. Notable experimental and numerical investigation was conducted by Nie et al. [50] in 2013. An imperfection sensitivity analysis on the order of the interactive formula of Yi et al. (from Leiva and Bergfelt et al.) was studied in detail (Eq. (8)). It was presented that the effect of the amplitude of the initial geometric imperfections on the post-buckling behaviour is negligible due to the membrane effect when the shear buckling governs the failure, but it has a significant effect when the shear yielding governs the failure. In consequence of the observations a new shear buckling curve ($\rho-\lambda_s$) was proposed which belongs to a realistic initial imperfection amplitude ($h_w/200$) with $n=0,7$ value for the order of the interactive formulae. Another notable research program was executed by Hassanein et al. [51] in 2013 on the applicability of the interactive formulae proposed by Yi et

al. (from Leiva and Bergfelt et al.). The quality of the web-to-flange junction was analyzed and stated that a realistic support condition is nearly fixed ($t_f/t_w \geq 3,0$). Using $n=0,6$ is appropriate for girders with fixed junctures and a new interaction shear buckling strength formula was proposed in case of fixed junction. Hassanein et al. concluded that for simple junction ($t_f/t_w < 3,0$) the first order ($n=1$) approximation is suitable.

There are three further methods for taking the interactive shear buckling behaviour into account. The first one (Eq. (14)) is suggested by El-Metwally [32] in 1998 which considers the inelastic buckling behaviour by having the yield strength of the material in the interactive formulae in an implicit way using second order ($n=2, u=1$) approximation. The value of “u” is a weighing factor of the yield term in the interactive formulae. It was confirmed by Sayed-Ahmed [43] in 2005 with $n=3$ and $u=1$, Sause et al. [49] in 2011 with $n=3$ and $u=2$ and Hassanein et al. [52] in 2013 with $n=3$ and $u=2$ especially for bridges. Sayed-Ahmed stated that global buckling mode is mostly affected by the corrugation angle and its post-buckling behaviour is negligible in comparison with the post-buckling behaviour of the local buckling mode where it is considerably high. Sause et al. presented that the geometric imperfections have a significant effect on the shear buckling strength capacity which have to be studied in the future and a new relation between $\rho-\lambda_s$ was also presented. Hassanein et al. investigated the corrugated webs used in bridge girders. It was stated that for bridge girders with corrugated webs the proposed formula of Sause et al. is the most suitable and confirmed the observation of Moon et al. that neither the stocky corrugated webs can not reach the yield shear strength. During the numerical analysis realistic initial imperfection amplitude was applied by taking the magnitude of the web thickness suggested by Driver et al.

$$\left(\frac{1}{\tau_{cr,I}} \right)^n = \left(\frac{1}{\tau_{cr,L}^E} \right)^n + \left(\frac{1}{\tau_{cr,G}^E} \right)^n + u \left(\frac{1}{\tau_y} \right)^n \quad (14)$$

The next interaction formula (Eq. (15)) was proposed by Eldib [41] in 2004 which considers the shear yielding in an implicit way but according to the failure mode just the unfavourable (lowest from the local and global) shear buckling strength is applied into the interactive formulae. If the local shear buckling failure mode governs then $n=4$ is used otherwise $n=1,5$. The reason is that in case of local shear buckling the post-buckling behaviour is relatively high but in case of global shear buckling it is negligible.

$$\left(\frac{1}{\tau_{cr,B}} \right)^n = \left(\frac{1}{\tau_{cr,B}^E} \right)^n + \left(\frac{1}{\tau_y} \right)^n \quad (15)$$

$$\text{where} \quad \tau_{cr,B}^E = \min(\tau_{cr,L}^E; \tau_{cr,G}^E). \quad (16)$$

The last one is the recommendation of the EN1993-1-5 [1] for the determination of the shear capacity of corrugated web girders which can be seen in Eq. (34) in Section 3.2. The calculation method is based on the proposal of Höglund [30] in 1997. Two different formulas were suggested for each local and global shear buckling and the unfavourable one is used. In 2002 Ullman [36] modified the method and provided the calculation methods for the reduction factors related to local and global shear buckling, respectively. It was explained that in case of

local buckling the post-buckling behaviour is considerable in comparison with the global buckling where it is negligible. According to the provided method the reduction factor for local buckling has no plateau but for global buckling there is. In addition, Ullman implemented an increase in the buckling coefficient for global buckling due to the restraintment of to the flanges if a certain stiffness criteria is fulfilled.

Most of the authors mentioned that more experimental testing and numerical investigations are needed in the field of shear capacity of corrugated web girders to specify the interactive formulas and thus a reliable design method could be prepared. Previously, experiments were carried out by Easley and McFarland [22], Leiva [24], Bergfelt et al. [25], Linder and Aschinger [26], Elgaaly et al. [27], Johnson and Cafolla [29], Peil [33], Yamazaki [34], Abbas [35], Pasternak and Hannebauer [37][40], Lee et al. [38], Gie et al. [42], Driver et al. [44], Scheer [45], Moon et al. [47] and Nie et al. [50].

As an outlook on the new research topics related to the shear buckling capacity of corrugated web girders the curved corrugated webs are in the scope of the paper of Eldib [48] in 2009. In comparison with existing bridges it was stated that a relevant amount of weight can be saved by the use of curved corrugated webs. Another completely new paper of Hassanein [53] in 2014 is dealing with haunched corrugated web girders which is more realistic for design since it is favourable due to design purposes. It was observed that the behaviour of the tapered bridge girders with corrugated webs differs from the prismatic ones. Therefore a new formula was adopted for those cases where the web-to-flange junction can be assumed as fixed, otherwise the formula proposed by Yi et al. may be suitable for use.

Local flange buckling

Flange buckling of corrugated web girders has importance in the bending moment resistance of the corrugated web girders. The loss of stability related to the flange buckling can occur in the compression flange due to local flange buckling between the inclined web folds or global buckling of the whole flange around the centerline of the corrugated web which is called torsional buckling of the flange. The latter one will be discussed in Section 2.4.

$$R = \frac{(a_1 + a_4)a_3}{(a_1 + 2a_4)b_f} < 0,14 \quad (17)$$

The local flange buckling of the compression flange is mainly influenced by the geometry of the corrugation profile. In the literature there is a small number of investigations regarding to local flange buckling. In 1997 Johnson and Cafolla [54] stated that under a certain value of the ratio given in Eq. (17), the average outstand of the flange may be used for the classification of the flange (the notation system can be found in Fig. 2 in Section 1). For practical girder geometries this ratio can not be fulfilled, thus researchers recommend to use the large and small flange outstands for the classification of the compactness of the compression flange as it is implemented in the commentary of the concerning EN1993-1-5 [55].

$$k_\sigma = 0,43 + \left(\frac{c}{a}\right)^2 \quad (18)$$

where c the largest outstand from the weld toe to free edge,
 a the greatest length of the assumed plate ($a=a_1+2a_4$, see Fig. 2).

It is assumed that the buckling coefficient related to the subdomains of the flange bounded by the inclined folds of the web corresponds to a simply supported plate – corrected with the flange subdomain's geometry – on the three edges and free on the fourth edge which leads to a conservative estimation of the buckling coefficient (Eq. (18)). Due to the lack of the investigations a conservative limit for the buckling coefficient ($k_\sigma=0,6$) is suggested to still guarantee the safety of the method (DAST [56]). It is important to mention that Sayed-Ahmed [43] in 2005 executed a parametric numerical study on this field in which the comparison was performed with the AISC-LRFD [81] and the CAN/CSA-S16.1-02 [82] which handle the classification in a less conservative way ($b_f/2/t_f$ as average outstand), but anyway the physical content is the same. It was concluded that the buckling coefficient ($k_\sigma=0,7$) may be chosen greater than the aforementioned limit in the commentary of the EN1993-1-5 [55]. He also stated that the classification of the section compactness should be based on the large outstand of the flanges.

2.3. Interacting stability behaviours

Although there are numerous researchers dealing with the investigation of the pure resistances namely bending (M), shear (V) and patch loading (F) resistances of corrugated web girders and there is just a small number of researchers dealing with the combined loading situation. It has to be mentioned that the number of corresponding papers in the literature is slightly more than ten which is a small number and only one recommendation can be found related to the M-V interaction of corrugated web girders in the EN1993-1-5 [1] Annex D. Another essential fact has to be announced that there are no investigations executed in the field of combined M-V-F interaction related to corrugated web girders.

Interaction of combined bending moment and shear force

In the international literature there are many papers dealing with the bending resistance and shear buckling resistance of corrugated web girders separately and there is just a few number of investigations dealing with the combined loading situation. This topic is presented in Section 3.2 in a more detailed way.

Based on the numerical results the stress distributions in the flanges and in the corrugated web was analyzed in detail by Elgaaly et al. [4] in 1997. It was presented that the ultimate moment carrying capacity can be calculated only from the contribution of the flanges and the shear force is carried only by the corrugated web and concluded that the interaction of combined bending moment and shear force can be neglected.

The EN1993-1-5 [1] recommends how to consider the effect of the shear force on the bending resistance. It suggests taking the reduction factor (f_T) into account as a multiplication factor of the pure bending resistance of a given girder. The reduction factor can be calculated from the

maximum additional transverse bending moment in the flanges coming from the shear flow in the web.

Sinusoidal corrugated web girders have been intensively studied on this research field by Pasternak and Hannebauer [37] in 2003. They investigated the effect of the shear force on the in-plane-bending resistance of the sinusoidal corrugated web girders. It was stated that there is no interaction between bending moment and shear force in case of sinusoidal corrugated web girders which is also noted in the EN1993-1-5 [1] Annex D, that the reduction factor (f_T) can be set to 1,0 in case of sinusoidal corrugated web girders. Another research was carried out by Kuchta [57] in 2006 investigating the structural behaviour of sinusoidal corrugated web girders under combined bending and shear. Based on the executed experimental investigations a numerical model was developed. As a result of the investigation an interaction equation was proposed with a small reduction due to the combined loading situation for girders with sinusoidal corrugated web.

Interaction of combined bending moment and patch loading

In the international literature there are a couple of papers dealing with the bending resistance and patch loading resistance of corrugated web girders separately and there is just a small number of investigations dealing with the combined bending and patch loading. This topic is presented in Section 3.2 in a more detailed way.

Elgaaly and Seshadri [17] first investigated the bending and transverse force interaction of the corrugated web girders in 1997. Based on a limited number of test results and numerical calculations, an interaction equation was proposed.

Kövesdi et al. [59] in 2011 modified the interaction curve provided by Elgaaly and Seshadri based on numerical results. In 2012 Kövesdi et al. [60], [61] presented the same proposal related to the interaction of combined bending moment and patch loading.

It has to be mentioned that there is no recommendation in the EN1993-1-5 [1] for the calculation of the combined bending moment and patch loading interaction in case of corrugated web girders.

Interaction of combined shear force and patch loading

In the international literature there are a couple of papers dealing with the pure shear and patch loading resistances of corrugated web girders separately and there is just a very low number of investigations paying attention to the combined shear and patch loading interaction. This topic is presented in Section 3.2 in a more detailed way.

Kähönen [16] first proposed a formula for the interaction of shear and transverse force in 1988. The patch loading resistance was discussed theoretically based on experimental results of corrugated web girders and the combined loading situation was taken with a shear stress interaction factor (k_w) into account (Eq. (2) in Section 2.2).

Elgaaly and Seshadri [17] investigated the shear and patch loading interaction of corrugated web girders in 1997. Five specimens were tested under local transverse force. The main aim of

their research was to determine the patch loading resistance of corrugated web girders. The pure patch loading resistance and the interaction between shear and patch loading was studied within the scope of a numerical research programme. Based on the numerical calculations, a design interaction curve was proposed.

Kövesdi et al. [62] in 2010 modified the interaction curve provided by Elgaaly and Seshadri based on numerical results. In 2012 Kövesdi et al. [60], [61] presented the same proposal related to the girders with corrugated web subjected to combined shear and transverse force.

It has to be mentioned that there is no recommendation in the EN1993-1-5 [1] for the calculation of the combined shear and patch loading interaction in case of corrugated web girders.

2.4. Global stability behaviour

In the international literature there are numerous papers dealing with lateral-torsional buckling (LTB) of corrugated web girders. In most of the papers analytical and numerical investigation have been conducted, however in some papers experimental tests and their results can be found.

The LTB of corrugated web girders was investigated experimentally and theoretically at first by Linder [64] in 1990. An analytically derived formula for ordinary flat web girders was proposed for the calculation of the critical moment regarding to simply supported conditions and under uniform bending by Chan and Lui [63] and Galambos [31]. The derived formula (Eq. (19)) was corrected and verified with the help of experimental results by Linder.

$$M_{ocr} = \frac{\pi}{L} \sqrt{EI_{y,co} G_{co} J_{co}} \sqrt{1 + W^2} \quad (19)$$

$$\text{where} \quad W = \frac{\pi}{L} \sqrt{\frac{EC_{w,co}}{G_{co} J_{co}}} \quad (20)$$

- L length of the girder,
- E Young's modulus,
- $I_{y,co}$ moment of inertia about the minor axis,
- G_{co} shear modulus,
- J_{co} pure torsional constant,
- $C_{w,co}$ warping constant.

Instead of warping constant used for flat web girders a new empirical formula for the warping constant (Eq. (21)) was proposed, but the other parameters in the critical moment formula were inherited from flat web girders.

$$C_{w,co} = C_{w,flat} + c_w \frac{L^2}{E\pi^2} \quad (21)$$

- where $C_{w,flat}$ warping constant for flat web girders,
- c_w correction factor for corrugated web girders according to Linder [64].

In 2005 Sayed-Ahmed [66] investigated different proposals for the moment gradient factors (Eq. (22)) numerically applying different moment distributions along the girder length. The critical moment formula for ordinary flat web girders suggested by Chan and Lui [63] and Galambos [31] (same as Eq. (19) but with parameters regarding to flat web girders) was used with an equivalent web thickness correction. Simply supported corrugated web girders were analyzed under 5 different linear bending moment distributions along the girders and 3 types of three-point bending in which the differences were the load position in the mid-span cross-section (top, middle, and bottom). During the investigations the numerically obtained results for the moment gradient factors were compared with the existing moment gradient factors for flat web girders proposed by Chen and Lui [63] and Galambos [31] for different end moments (Eq. (25)) and for three-point bending and uniform loading (Eq. (26)) and proposed by the AISC-LRFD [81] for different intermediate moment values (Eq. (27)). Calculation of the critical moment with the moment gradient factor can be seen in Eq. (22).

$$M_{cr} = C_b \cdot M_{ocr} \quad (22)$$

where C_b moment gradient factor,

M_{ocr} critical moment for simply supported beam under uniform bending (Eq. (19)).

The results showed a very good agreement with the proposals regarding to the moment gradient factors, but the numerically calculated critical moments did not coincide with the critical moment formula since it is underestimated and it is too conservative for design purposes. Thus a simply equivalent web thickness correction is not satisfactory for taking the real LTB behaviour of corrugated web girders into account. The main conclusion of the paper was that the lateral-torsional buckling capacity of corrugated web girders is greater than those of girders with flat web.

Theoretical, experimental and numerical investigation were carried out by Abbas et al. [6, 7, 8] in 2006 and 2007. It was stated and proved that under in-plane loading of corrugated web girders due to the effect of shear flow in the web, additional transverse bending moment is generated into the flanges which causes a torsional moment component in the flanges. Therefore out-of-plane twist of the girder occurs simultaneously with in-plane deflections due to in-plane loading.

In 2008 Hannebauer [13] confirmed the statement of Sayed-Ahmed that the global buckling capacity of trapezoidal corrugated web girders are greater than those of flat web girders. However, numerical investigation regarding to sinusoidal corrugated web girders were also carried out but it was concluded that the lateral-torsional buckling capacity of these girders are not greater than those of flat web girders in comparison with the EN1993-1-5 [1] buckling curves.

As a next step Moon et al. [70] executed a notable theoretical and numerical study under uniform in-plane bending of simply supported corrugated web girders in which a new warping constant (Eq. (23)) was proposed and verified and the moment of inertia about the minor axis is considered with no contribution of the web. It was concluded that the warping constant

suggested by Linder is not applicable for greater corrugation angles than 15 degree and the LTB capacity of corrugated web girders is higher in case of greater corrugation angles than those of flat web girders. Hence the buckling curve provided by the EN1993-1-5 [1] is applicable but conservative in case of elastic and inelastic buckling as well considering initial imperfections. It is important to mention that the provided shear modulus by Johnson and Cafolla [29] in 1997 for corrugated web girders was suitable.

$$C_{w,co} = \frac{1}{3} \sum (W_{ni}^2 + W_{nj} W_{ni} + W_{nj}^2) \cdot t_{ij} L_{ij} \quad (23)$$

where W_{ni} normalized unit warping at point “i”,

t_{ij}, L_{ij} thickness and length of the individual interconnected plate elements.

In 2010 Nguyen et al. [71] investigated theoretically and numerically the LTB of corrugated web girders under uniform moment gradient. He derived the same warping constant (Eq. (23)) suggested by Moon et al. and concluded that the elastic LTB strength of an I-girder with trapezoidal web corrugations is not only affected by a warping constant and shear modulus but also by the moment of inertia about the minor axis which is the quadratic function of the divergency of the corrugated web which varies from zero to the half of the corrugation depth. Thus to simplify the calculation, the moment of inertia about the minor axis is given by the average value of the moment of inertia using zero and half corrugation depth. It was observed that the value of the elastic LTB strength of the I-girder with trapezoidal web corrugations under uniform moment gradient is about 21% to 29% larger than the I-girder with flat web.

Kazemi [74] in 2010 analyzed numerically the effect of an elastic lateral restraint in the mid-span on the inelastic LTB behaviour of corrugated web girders under pure bending and concluded that the effect of the restraint depends not only on the stiffness of the restraint but also on the modified slenderness of the girder. Method for the calculation of the optimal stiffness of the restraint was proposed under uniform bending. Kazemi [73] conducted a numerical parametric study as well in 2010 and concluded that the corrugated web girders have a higher resistance against LTB than those of with flat webs.

Zhang et al. [75] in 2011 presented a method for calculating the critical buckling moment under uniform bending relying on the assumptions first presented by Lindner [64], stating that the moment of inertia about the weak axis and the torsion constant of the girder with corrugated web can be assumed to be equal to those of with flat web, and that the increased critical buckling moment is caused only by an increased warping constant (Eq. (24)) based on a prismatic girder with a flat, eccentric web.

$$C_{w,co} = C_{w,flat} + \frac{t_w h_w^3 e^2}{12} \quad (24)$$

where e eccentricity of the web.

In 2013 another notable investigation was performed by Moon et al. [76] on the LTB of corrugated web girders under linear moment gradient using simply supported boundary conditions. During the investigation the observations of Abbas et al. were considered that under

in-plane loading out-of-plane twist and in-plane deflection are eventuated, therefore different corrugation layouts, initial imperfections and residual stresses were analyzed to determine an applicable calculation method for the moment gradient factor in case of linear moment gradient. Three existing method for the determination of the moment gradient factor were analyzed. The first one (Eq. (25)) is provided by Chen and Lui [63] and Galambos [31], the second one (Eq. (27)) is provided by AISC-LRFD [81] and the last one (Eq. (28)) is proposed by Lim et al. [65] which was also produced for flat web girders by using Bubnov-Galerkin method. Calibration and modification were done by Chapra [67] in 2008 to provide the applicability of the moment gradient factor of Lim et al. for corrugated web girders. The results showed that the moment gradient factor of Lim et al. gives a very good agreement with the EN1993-1-5 [1] “b” buckling curve for linear moment gradient. During the numerical investigation the inelastic buckling behaviour was considered with initial geometric imperfections and residual stresses.

$$C_b = 1,75 + 1,05 \frac{M_A}{M_B} + 0,3 \left(\frac{M_A}{M_B} \right)^2 \leq 2,3 \quad (25)$$

where M_A smaller end moment,

M_B greater end moment.

$$C_b = \begin{cases} A/B \text{ loads acting at the top flange,} \\ A \text{ loads acting at the shear centre,} \\ AB \text{ loads acting at the bottom flange.} \end{cases} \quad (26)$$

where $A = 1,35$
 $B = 1 + 0,649W - 0,18W^2$ for concentrated loads,

$A = 1,12$
 $B = 1 + 0,535W - 0,154W^2$ for uniformly distributed loads.

$$C_b = \frac{12,5M_{\max}}{3M_a + 4M_b + 3M_c + 2,5M_{\max}} \quad (27)$$

where M_i absolute value of the moment at the quarter and mid-span points,

$M_{\max}$ absolute maximum bending moment along the girder.

$$C_b = \frac{2}{\sqrt{(1 - (M_1 / M_2))^2 + R_1 (1 + (M_1 / M_2))^2}} \quad (28)$$

where R_1 coefficient (for corrugated web girders it is set to 0,1442 [67]),

M_i end moments on the simply supported girder.

It was also observed that in case of elastic buckling using odd number of half corrugation waves a bifurcation type buckling behaviour governs the failure, however nonlinear type behaviour was observed for those of with even number of half corrugation waves. Another important observation was drawn out that in case of inelastic buckling taking the initial imperfections and

residual stresses into account, the difference between the LTB capacities is not considerable by having even or odd number of half corrugation waves.

Further numerical investigation was executed in 2013 by Tohamy et al. [77] on the LTB behaviour of corrugated web girders in comparison with those of flat web girders in case of simply supported boundary conditions. Higher LTB resistance was again attributed to the corrugated web girders according to the numerical results. It was concluded that smaller the corrugation wave length or/and thicker the web, higher is the LTB capacity. It was also observed that the highest LTB capacity was reached when the corrugation angle is equal to 30 degree. Another notable conclusion was drawn out that using equivalent in weight flat and corrugated web girders the less deflection belongs to the corrugated one.

Although many theoretical investigation was done coupled with numerical analysis, nevertheless just a very few number of experimental tests have been executed namely by Linder [64], Hannebauer [13], Jamali [69], Denan et al. [72] and Divahar et al. [79]. Further experimental testing is needed in this field coupled with theoretical and numerical investigations, because different boundary and loading conditions have not been taken into account in this field yet. However, there are another paper can be found dealing with the applicability of corrugated web girders into frame structures by Sabat [78] in which the global behaviour of the frame was investigated as a case study. Promising results were concluded by Sabat that higher resistances can be achieved using corrugated webs.

It has to be mentioned that there is no direct recommendation in the EN1993-1-5 [1] for the calculation of the LTB strength of corrugated web girders, however as far as the authors know, the reduction factor (f_T) in the EN1993-1-5 Annex D related to the bending moment resistance was calibrated based on experimental results on specimens tested under combined bending and shear, where the failure modes were the LTB of the specimens.

2.5. Fatigue behaviour

From optimal solution point of view of the desired steel structures due to the economical design demands need to be as low in weight as it can be. The design demands indicate the use of thin-walled structures for which the loss of stability is the critical keypoint of the structure. Therefore stiffeners are applied on the structure by welding which reduce the life time of the structure due to fatigue. Based on the advantageous stability behaviour of the corrugated web girders less stiffeners are needed which may leads to a more favourable fatigue behaviour. There is a relatively low number of papers can be found in the literature dealing with the fatigue problems of corrugated web girders. The available papers are mostly investigating experimentally the fatigue behaviour of such girders, but some of them are focusing on the numerical analysis as well. For the sake of completeness not only the papers regarding to trapezoidal but sinusoidal corrugated web girders are being discussed in this subsection which is not entirely relevant in case of fatigue while the geometry of the structure has a significant influence on the stress concentrations promoting the appearance of cracks causing propagation and finally the collapse of the structure.

Sinusoidal corrugated web girders

The first investigation related to sinusoidal corrugated web girders was executed by Harrison [83] in 1965. Two specimens were tested under four-point-bending. One of the girders failed from a notch in the flame cut edge of the flange and the other was failed from cracks initiated from the web-to-flange fillet weld and propagated across the flange. In 1979 Korashy and Varga [84] tested the fatigue behaviour of eleven girders stiffened using discrete web corrugations under four-point-bending. It was reported that the initiation of the cracks arised at the toe of the web-to-flange fillet weld where the inclination of the web was close to the maximum and propagated into the flange causing fracture. Not only homogeneous but inhomogeneous hybrid girders were tested with different steel qualites in the flanges and in the web. No significant differences were reveled in the fatigue life between the two types. It was observed from the results that the wave-stiffened girders had at least 25% higher fatigue strength than those of with flat webs and even with stiffeners welded only to the web.

In 2006 crane girders with sinusoidal corrugated webs were tested under shear, transverse force and moving load to analyze the fatigue behaviour by Machacek et al. [94]. It was reported that under shear loading the initiation of the cracks arised from the weld toes at the web-to-flange or web-to-stiffener connections and propagated quickly to the fracture. As a result, proposal was presented for the fatigue detail categories under shear and transverse forces for design purposes in case of girders with sinusoidal corrugated webs.

Trapezoidal corrugated web girders

Rodriguez [86] in 2000 reported on tests of six girders with trapezoidal web corrugations. For the web-to-flange welds continuous one-sided fillet welds were applied. Five girders failed to fracture where the cracks initiated close to the stiffeners placed at the load and reaction points to prevent web crippling. One girder failed by crack initiation from the web-to-flange fillet weld toe at a location outside of the pure bending region where combined bending and shear interacted. It is important to confirm again that Rodriguez applied one-sided web-to-flange fillet welds which have never been used in bridges at all, thus this research is not relevant for further metioning and study.

In 2001 six specimens with trapezoidal corrugations were tested under four-point-bending by Ibrahim [87]. For all girders the web-to-flange fillet weld was 5 mm in size and the bend radius between the adjacent web folds was 27 mm. All the test specimens failed by fatigue cracking within the constant moment region. The initiation of the cracks arised at the web-to-flange weld toe along an inclined fold and propagated in the tension flange. For five girders the point of the crack initiation was at the end of the inclined fold at the beginning of the bend region. This point has been called the S-point (see Fig. 5) in case of trapezoidal corrugated web girders. In case of one girder the initiation of the crack arised at the weld toe in the middle of an inclined fold which was attributed to the welding stop-start.

Experimental and numerical research program was executed by Anami and Sause et al. [89, 90, 91] in 2006. Eight specimens with trapezoidal corrugations were tested under four-point-

bending, the weld was 8 mm in size and the bend radius was 120 mm. Six girders failed due to fracture at the web-to-flange weld toe in the tension flange within the pure bending region and two girders did not fail at all. In case of five fractured girders the fatigue cracks initiated and propagated from the S-point and one fractured girder failed due to cracks arising at the parallel web fold which was attributed to the welding stop-start process. It was concluded that corrugated web girders have longer fatigue life than ordinary flat web girders with stiffeners, but smaller than those of without stiffeners. An author observation was reported that the welding method influences the life time since robotic GMAW welding has almost one and a half greater resistance than semiautomatic GMAW welding. It was reported that the highest measured stress range reached 121% compared with the nominal stress range at the stress concentration point.

The observations from test results was verified by numerical analysis that the crack initiation arises in the S-point at the bend radius between the adjacent web folds. During the numerical analysis the effect of the corrugation angle and the bend radius were meant to be investigated. It was stated that increasing the corrugation angle or/and decreasing the bend radius the fatigue life decreases effectively. The effect of the corrugation depth and the fold lengths are negligible.

Another investigation was conducted by Ibrahim et al. [92, 93, 94] in 2006 on the fatigue behaviour of trapezoidal corrugated web girders. Experimental testing and numerical analysis was also included in the research program. Six specimens were tested under four-point-bending. For all specimens the weld size was 5 mm and the bend radius was 27 mm. All the fatigue cracks initiated at the pure bending zone from the S-point and propagated perpendicular to the longitudinal stress. It was revealed that the corrugated web girders have 30-50% longer fatigue life than those of with flat webs even with short cutted stiffeners from the tension flange. Moreover the results showed that the fatigue life category for corrugated web girders is smaller than those of with unstiffened flat webs. Furthermore, the researchers reported that the welding stop-start point results in a high stress concentration that can reduce fatigue life. In comparison of the nominal stress with the highest measured stress the differences was 117 in percentage.

The authors suggested to develop as simple numerical model as possible. Modelization of only one corrugated wave can be satisfactory since their outcomes did not show any notable differences. The numerical analysis again verified that the crack initiation arises in the tension flange in the S-point. It was also pointed out that the most influential geometric parameters are the corrugation angle and the bend radius between the web folds.

Experimental investigation was executed on trapezoidally corrugated web girders by Kövesdi et al. [96, 97] in 2009. Total number of six specimens with the same corrugation profile were tested, two under four-point-bending and four under three-point-bending. The weld size was 3 and 6 mm and the bend radius between the web folds was 60 mm. Solely three specimens failed due to fatigue cracks which initiated from the S-point. It was pointed out that the smaller the weld size, the longer is the fatigue life, therefore the use of as minimal weld size in the design as possible was recommended. Based on the nominal stresses the fatigue detail category was proposed to 90 according to the EN1993-1-9 [99] considering the previous experimental results from the literature as well. Moreover the papers emphasized that the highest measured stress concentration reached 124,6% compared with the measured nominal stress in the undisturbed

zone of the flange in case of four-point-bending and 116,4% in the case of three-point-bending. Furthermore it was observed that the combined bending and shear loading has a significant influence on the fatigue life.

Notable experimental and numerical investigations were carried out analysing the fatigue behaviour of corrugated web girders by Wang et al. [99] in 2014. Fillet welded joint with corrugated plate specimens were fabricated as can be seen in Fig. 4 and Fig. 5. It was assumed that the virtual girder is subjected to pure bending and it was employed that the contribution of the web to the in-plane bending moment capacity is negligible which is not true in a small region of the web close to the flanges but for the sake of simplicity it was assumed.

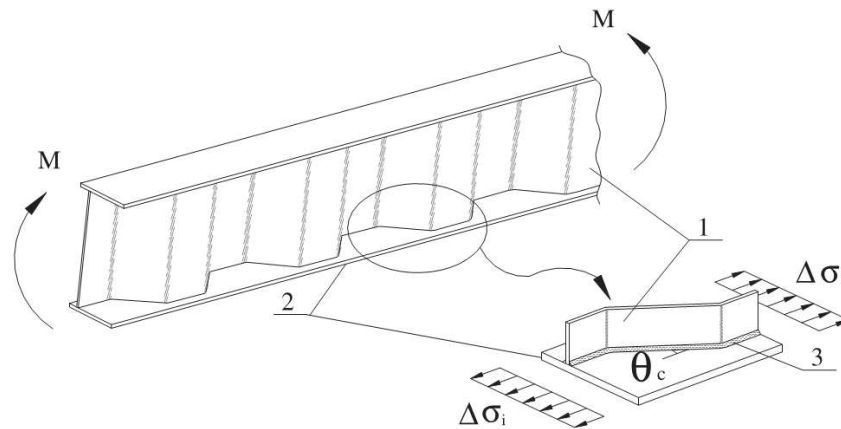


Fig. 4: Illustration of a fillet welded joint with corrugated plate [99].

Several specimens were tested with corrugation angles equal to 30, 45 and 60 degree and with different bend radiuses between the web folds. It was observed that the cracks initiated close to the S-point depending on the corrugation angle and the centre of the bend radius. By increasing the corrugation angle or/and the bend radius of the web the crack initiation translated slightly from the S-point to the bended region which is shown in Fig. 5. Moreover the fatigue strength of the test joints is improved with the decrease of the corrugation angle or with increase of the bend radius of the web. Furthermore, in comparison the results on the S-N curve with the previous results of real girders from the literature it was revealed that the same fatigue behaviour was observed in strength and in crack initiation and propagation as well.

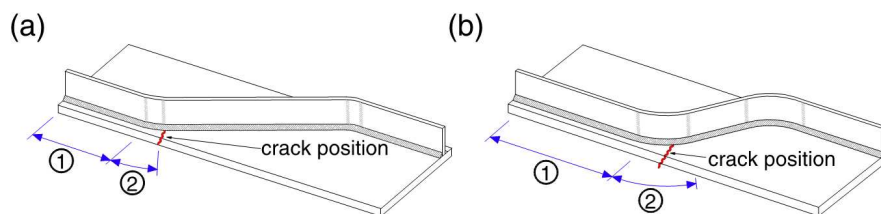


Fig. 5: Fatigue crack positions of specimens with varying corrugation angel [99].

Based on the experimental results numerical models were developed to investigate the stress concentrations around the S-point. Parametric study was executed to analyze the effect of the different geometric parameters in detail.

Hybrid girders with concrete flanges and corrugated steel plate web

Beyond pure steel corrugated web girders inhomogeneous hybrid girders have been also in the scope of some researches. Hybrid bridges are mostly composed with prestressed concrete flanges and trapezoidal corrugated steel webs. Several connection types are existing between the concrete flange and the steel web which have to be optimized considering fatigue as well.

The first investigation in this topic was conducted by Takesita et al. [85] in 1997. The main conclusion of this paper pointed out that the fatigue cracks initiate from the web-to-flange weld toe even if the web is embedded into the concrete flange. Therefore it was suggested that in most cases it is easier to analyze the fatigue strength of the structure by substituting the concrete flanges to steel which leads to a safe result.

Experimental investigations have been performed on hybrid bridges with corrugated steel webs and concrete flanges by Sakai et al. [88] in 2002. The purpose of the tests was to analyze the fatigue behaviour of the connection between the web plates at the elongation cross-section. A new connection type was proposed to reduce the stress concentration and to improve the fatigue life. The fatigue tests revealed that up to 60 million cycles, no fatigue cracks occurred.

Despite the detailed standardization of ordinary I-girders, due to the lack of investigations on the fatigue behaviour of corrugated web girders there is no recommendation in the concerning EN1993-1-9 [99]. Further experimental investigations are needed coupled with numerical analysis. Each researchers stated that longer fatigue life can be achieved with corrugated web girders instead of using stiffened flat web girders.

2.6. Evaluation and current and future aims

As it turned out, several questions have to be answered in case of corrugated web girders. In the Eurocode 3 standard there are only recommendations for the calculation of the bending resistance considering local flange buckling and the shear buckling resistance regarding to steel corrugated web girders. This summary had the aim to discuss the problems to be solved coupled with an in-depth literature overview in this topic.

The stress distributions have been investigated by several researchers in the past. It is known that the shear force is carried by the web and the bending moment is carried by the flanges. The stress distributions have been studied in detail, thus the observed additional transverse bending moments in the flanges coming from the shear flow in the web have been discussed theoretically as well which is the specific characteristic of such structures. It was stated by later papers that there is no relationship observed between the transverse bending moment and the bending resistance reduction, if plastic design is applied.

Several researchers were dealing with the investigation of the pure patch loading resistance of corrugated web girders, however there is no recommended design formula in the EN1993-1-5 [1]. Analytically derived formulas have been already existing from different researchers. By harmonizing the proposal of Kähönen [16] with the EC3 principle for buckling analysis it can be applicable for bridge structures where the patch loading length is relatively long.

The EN1993-1-5 [1] Annex D provides recommendation for the calculation of the shear buckling resistance of corrugated web girders. The design formula does not consider the interactive shear buckling phenomena, thus the local and global shear buckling are handled separately and the unfavourable one has to be considered. Papers from the literature are dealing with the interactive shear buckling resistance, thus different interactive formulas have been published coupled with different buckling curve proposals.

The local flange buckling is considered in the EN1993-1-5 [1], however it gives conservative results because of the assumed boundary conditions on the three edges of the flange's "subpanels" between the inclined web folds. Therefore the buckling coefficient should be specified with further investigations.

Although there are numerous researcher activities dealing with the investigation of the pure resistances namely bending (M), shear (V) and patch loading (F) resistances of corrugated web girders and there is just a small number of researchers dealing with the combined loading situation. It has to be mentioned that the number of corresponding papers in the literature is slightly more than ten regarding to M-V, M-F and V-F interaction which is a small number and only one recommendation can be found related to the M-V interaction of trapezoidal corrugated web girders in the EN1993-1-5 [1] Annex D which was found to be too conservative by Kövesdi et al. [58]. Another essential fact has to be announced that there are no investigations executed in the field of combined M-V-F interaction related to corrugated web girders.

Several researchers have achieved significant results in the investigation of the lateral-torsional buckling of the corrugated web girders within the last 10 years. Only simply supported girders were in the scope of the investigations, therefore the further investigations regarding to the different boundary and loading conditions are needed. There is no direct recommendation in the EN1993-1-5 [1] for the calculation of the lateral-torsional buckling strength of corrugated web girders, however as far as the authors know, the reduction factor (f_T) in the EN1993-1-5 [1] Annex D related to the bending moment resistance was calibrated based on experimental results on specimens tested by combined bending and shear, where the failure mode was lateral-torsional buckling of the specimens.

Due to the low number of investigations on the fatigue behaviour of corrugated web girders there is no recommendation in the EN1993-1-9 [99]. Further experimental investigations are needed coupled with numerical analysis. Each researchers stated that longer fatigue life can be achieved with corrugated web girders instead of using stiffened flat web girders. Only one proposal [96] can be found in the literature for the fatigue detail category regarding to the concerning EN1993-1-9 [99]. It suggests detail category 90 which is in between the stiffened and unstiffened flat web girders' detail category.

As a conclusion it can be stated that one of the most interesting and not solved research topic regarding to corrugated web girders is the determination of the interacting behaviour under combined bending moment, shear and transverse force. However a few authors have already discussed the interaction between M-V, M-F and V-F which may be the individual interaction planes of the aforementioned M-V-F 3D interaction surface. Therefore the current MSc diploma thesis focuses on the numerical investigation of the above mentioned interactions.

3. Interacting stability behaviour

Based on the conclusions of the Section 2 it can be stated that one of the most necessary investigation in the field of corrugated web girders is on the interacting stability behaviour. The importance and the basic conditions of this topic are discussed in Section 3.1. The previous research activities in this field and their results are presented in Section 3.2 while the aims and the research strategy of the current diploma thesis are introduced in Section 3.3 and 3.4, separately.

3.1. General statement

Typical layout of a hybrid superstructure can be seen in Fig. 6. In case of hybrid bridges the upper and lower flanges of the bridge deck are made of prestressed concrete however the webs are made of corrugated steel plates. During the incremental launching procedure only the steel corrugated webs with steel flanges are launched which can be seen in Fig. 7. By using corrugated steel plates as web in the launching girder makes it possible to reduce temporary loads, materials and construction equipments. Precast concrete panels are placed on the lower flanges of the steel launching girder thus the needs for formwork of lower slab and scaffolding are omitted.

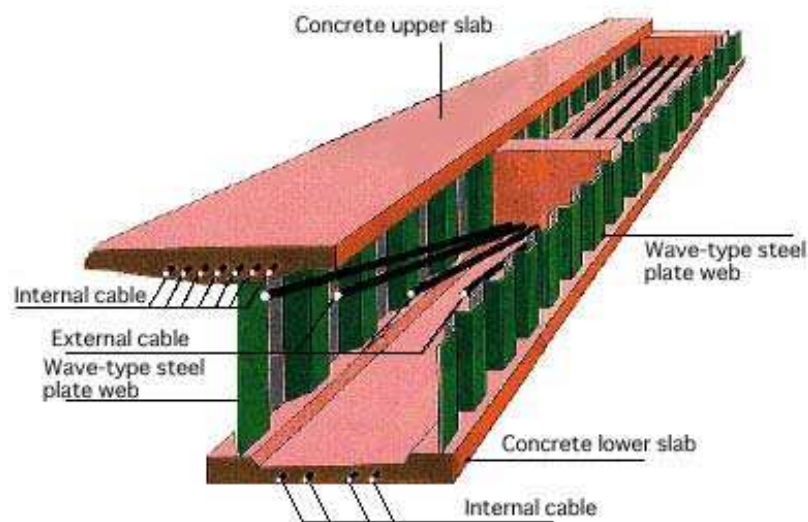


Fig. 6: Typical layout of a hybrid superstructure.

During the construction stages the girders are subjected to the combination of different actions as bending moment, shear force and patch loading which result in a complex stress field in the structure and may occur interacting instability phenomena. The most critical stages when the launching superstructure is just before reaching the next pier and a large cantilever bending moment acts on the structure with accompanying large shear and transverse force at the previous pier.

In the past numerous investigations have been carried out in case of corrugated web girders for design purposes. Many questions have come out due to corrugated web and new investigations became necessary for better understanding and for standardization. Numerous experimental and

numerical investigations can be found in the international literature which is practicably discussed in Section 2 related to girders with corrugated webs. These investigations are dealing with the strength, stability and fatigue problems of such structures.



Fig. 7: Incrementally launched superstructure with steel corrugated web.

In spite of the aforementioned numerous papers there is a very low number of papers dealing with the interacting stability behaviour of corrugated web girders. The concerning literature is slightly more than ten papers which is a small number. Therefore the investigation has high priority. It is commonly accepted among researchers and designers that the flanges of corrugated web girders are carrying the major part of the bending moment and normal forces and the corrugated web plate is carrying the major part of the shear force. It is also known from previous investigations that the shear flow in the corrugated web indicates transverse bending moment in the flanges resulting additional normal stresses. Some papers are dealing with the normal stress distribution in the flanges due to shear force and a few number of papers are dealing with the reduction in the in-plane-bending moment capacity due this aforementioned additional transverse bending moment in the flanges, thus namely the interaction between the bending moment and shear force of such girders.

Researchers dealing with the patch loading resistance of corrugated web girders agree with the obvious fact that both, the flange and the corrugated web contribute to the patch loading resistance of the girder. It is also expressed in the proposed formulas for the determination of the patch loading resistance. Due to the contribution of both structural members an accompanying shear force or bending moment can reduce the effort of capacity into the patch loading resistance. Therefore the investigations of the combined bending moment and patch loading and the combined shear force and patch loading are in the scope of a few papers discussed in Section 3.2. In real cases the three actions act in the same time which result in a complex stress field in the structure. There are no papers found in the literature dealing with corrugated web girders subjected to combined bending moment (M), shear (V) and transverse forces (F). Therefore the current diploma thesis focuses on the numerical investigation of the structural behaviour of the girders with corrugated webs under all of the aforementioned actions and their combinations.

In case of conventional flat web girders the proposed interaction planes and the whole 3D interaction surface are discussed which can be read in the Appendix C. Proposals regarding to corrugated web girders for the distinct interaction planes (M-V, M-F and V-F) of the 3D interaction surface (M-V-F) have been already existing. However the validity of these interaction planes have to be studied numerically and evaluated at first and then the 3D interaction surface have to be in the scope of the current diploma thesis to give a proposal for the determination of the M-V-F interaction which is essential in case of an incrementally launched superstructure with steel corrugated webs.

3.2. Previous research activities

Calculation methods for the pure resistances

Determination formulas for the pure bending and shear buckling resistances have been already existing in the EN1993-1-5 [1] Annex D. However there are no recommendations in the standard for the calculation of the pure patch loading resistance of corrugated web girders, therefore in the current diploma thesis the determination of the pure patch loading resistance is based on the proposal of Kövesdi et al. [18] which is specialized for bridge type girders.

According to the EN1993-1-5 [1] Annex D the pure bending resistance can be determined from the contribution of the flanges by the Eq. (29).

$$M_{Rd} = \frac{b_f \cdot t_f \cdot \chi \cdot f_{yf}}{\gamma_{M1}} \cdot \left(h_w + \frac{t_{uf} + t_{lf}}{2} \right) \quad (29)$$

where b_f, t_f the smallest width and thickness from the lower or upper flanges,
 t_{uf}, t_{lf} upper and lower flange thicknesses,
 h_w web depth,
 χ reduction factor due to local flange buckling,
 γ_{M1} partial factor for buckling (1,1 for bridges and 1,0 for buildings),
 f_{yf} flange yield strength.

The EN1993-1-5 [1] Annex D suggests using buckling coefficient assuming an equivalent rectangular plate supported along three edges. The buckling coefficient can be calculated according to the Eq. (18) in Section 2.2. The flange buckling is considered in EN1993-1-5 by the buckling coefficient given above in Eq. (18) with an upper limit value of 0,6 discussed in Section 2.2. After the determination of the buckling coefficient the relative slenderness and the reduction factor related to the outstanding compression elements can be determined on the same way as for the conventional I-girders (acc. to the EN1993-1-5 [1] 4.4.1 and 4.4.2).

Design method of the shear buckling resistance of corrugated web girders is implemented in the EN1993-1-5 [1] Annex D. The formula for the determination of the design shear capacity can be seen in Eq. (30) without introducing of the calculation of the reduction factor.

$$V_{bw,Rd} = \chi_c \frac{f_{yw}}{\gamma_{M1} \sqrt{3}} h_w t_w \quad (30)$$

where χ_c the lesser reduction factor regarding to local and global buckling,
 f_{yw} yield strength of the web,
 γ_{M1} partial factor for buckling (1,1 for bridges and 1,0 for buildings),
 h_w web depth,
 t_w web thickness.

The patch loading resistance is determined in the current master's thesis according to the proposal of Kövesdi et al. [18] which can be calculated according to Eq. (31).

$$P_u = \frac{2 \cdot \sqrt{4 \cdot M_{pl,f} \cdot \chi \cdot t_w \cdot f_{yw} + f_{yw} \cdot \chi \cdot t_w \cdot s_s}}{\gamma_{M1}^{**}} \quad (31)$$

where χ reduction factor,
 f_{yw} yield strength of the web,
 t_w web thickness,
 s_s loading length,
 $M_{pl,f}$ plastic moment calculated from the flanges,
 γ_{M1}^{**} partial factor equal to 1,35 (for quality S235) derived by Kövesdi [60, 68].

This procedure is proposed for only bridge type girders thus two conditions have to be fulfilled, one for the smallest web fold width and one for the corrugation angle (Eq. (5) in Section 2.2). There are other limits of this method, because the analyzed parameter range for h_w/t_w ratio was between 200 and 500, and the analyzed parameter range for s_s/h_w was between 0,4 and 0,8, but anyway there is no existing general proposal for corrugated web girders.

Previous studies on the bending moment and shear interaction

In the international literature there are numerous papers dealing with the in-plane-bending moment resistance and shear buckling resistance of corrugated web girders separately and there is just a few number of investigations paying attention to the combined bending moment and shear force.

Based on numerical results verified by experiments the stress distributions in the flanges and in the corrugated web was analyzed by Elgaaly et al. [4] in 1997. It was presented that the ultimate moment carrying capacity can be calculated only from the contribution of the flanges and the shear force is carried only by the corrugated web and stated that there is no observed interaction between the bending moment resistance and the shear buckling resistance of girders with trapezoidal corrugated web.

EN1993-1-5 [1] recommends how to consider the effect of the shear force on the bending resistance. It suggests taking the reduction factor (f_T) into account as a multiplication factor of the pure bending resistance of a given girder. The reduction factor can be calculated from the maximum additional transverse bending moment in the flanges coming from the shear flow in

the web. The reduced moment capacity can be calculated according to the Eq. (32) and the interaction diagram can be seen in Fig. 8.

$$M_{V,Rd} = f_T \cdot M_{Rd} \quad (32)$$

$$\text{where: } f_T = 1 - 0,4 \sqrt{\frac{\sigma_x(M_z)}{f_{yf}}} = 1 - 0,4 \sqrt{\frac{6M_{z,\max}}{f_{yf} b_f^2 t_f}} \quad (33)$$

f_{yf} yield strength of the flange plate,

γ_{M0} partial factor for stress check,

$\sigma_x(M_z)$ maximum value of the additional normal stresses coming from transverse bending moment,

$M_{z,\max}$ maximum value of the transverse bending moment (see Fig. 3).

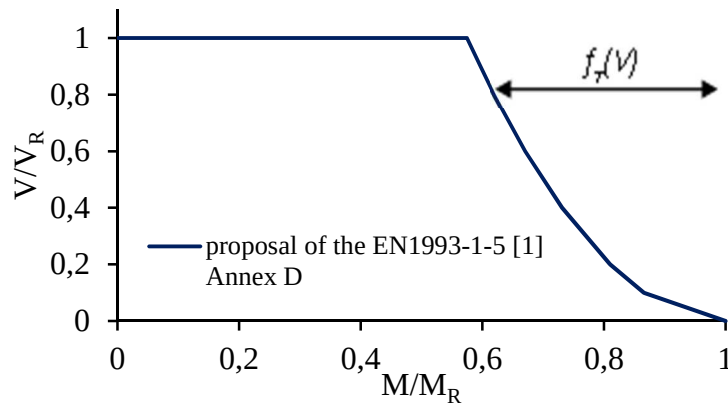


Fig. 8: M-V interaction diagram according to EN1993-1-5 [1].

It has to be mentioned that according to the author's knowledge, the reduction factor (f_T) in the EN1993-1-5 [1] Annex D regarding to the bending moment resistance was calibrated based on experimental results on specimens tested under combined bending moment and shear force, where the failure modes were lateral-torsional buckling of the specimens. Thus the aforementioned design procedure is not a direct design method for the M-V interaction because it considers the lateral-torsion buckling of these girders which belongs to the global stability problems of girders with corrugated web.

Sinusoidal corrugated web girders have been intensively studied on this research field by Pasternak and Hannebauer [37] in 2003. They investigated the effect of the shear force on the in-plane-bending resistance of the sinusoidal corrugated web girders. It was concluded that there is no interaction between bending moment and shear force in case of sinusoidal corrugated web girders. This statement is also noted in the concerning EN1993-1-5 [1] Annex D, that the reduction factor (f_T) can be set to 1,0 in case of sinusoidal corrugated web girders.

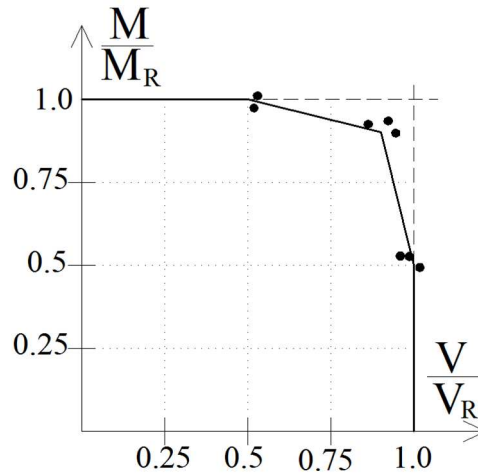


Fig. 9: M-V interaction diagram for sinusoidally corrugated web girders [57].

Another research was carried out by Kuchta [57] in 2006 investigating the structural behaviour of sinusoidal corrugated web girders under combined bending moment and shear force. Based on the executed experimental investigations a numerical model was developed. As a result of the investigation an interaction equation was proposed with a small reduction due to the combined loading situation for girders with sinusoidal corrugated web. Kuchta did not take the effect of the transverse bending moment in the flanges into account, thus the magnitude of the additional normal stresses in the flanges were not considered. Furthermore the possible shear buckling of the web or local flange buckling phenomenas were also not considered instead the plastic resistances were determined. The proposal of Kuchta [57] for the M-V interaction can be seen in Fig. 9 where the maximum reduction in the bending and shear resistance is 8,33%.

Previous studies on the bending moment and patch loading interaction

Elgaaly and Seshadri [17] investigated the bending and transverse force interaction of the trapezoidal corrugated web girders in 1997. Based on a limited number of experimental test results and numerical calculations, a “moving” interaction equation was proposed which is demonstrated by the Eq. (34) below. During their investigation unstiffened flange model was used which means that the interaction of the web and flange buckling is allowed as a collapse mode of the structure.

$$\left(\frac{M}{M_R}\right)^{1,25} + \left(\frac{F}{F_R}\right)^{1,25} \leq 1,0 \quad (34)$$

where M_R bending resistance,
 F_R patch loading resistance.

Based on the numerical results Kövesdi et al. [59] found the proposal of Elgaaly and Seshadri too conservative and modified the indices of the “moving” interaction curve in 2011. During the investigation stiffened flange model was used which is shown by Fig. 16 and 17 and described in Section 4.5. The applied geometric parameter domain of the models was wider than before on which the typical failure modes were the web crippling and bending. This new proposal can be seen in Eq. (35). In 2012 Kövesdi et al. [60], [61] presented the same proposal

related to the interaction of combined bending moment and patch loading. In Fig. 10 the proposals for the M-F interaction diagram can be seen according to Elgaaly and Seshadri [17] and Kövesdi et al. [59].

$$\left(\frac{M}{M_R}\right)^{1,2} + \left(\frac{F}{F_R}\right)^8 \leq 1,0 \quad (35)$$

where M_R bending resistance,
 F_R patch loading resistance.

It has to be mentioned that there is no recommendation in the EN1993-1-5 [1] for the calculation of the combined bending moment and patch loading interaction in case of corrugated web girders.

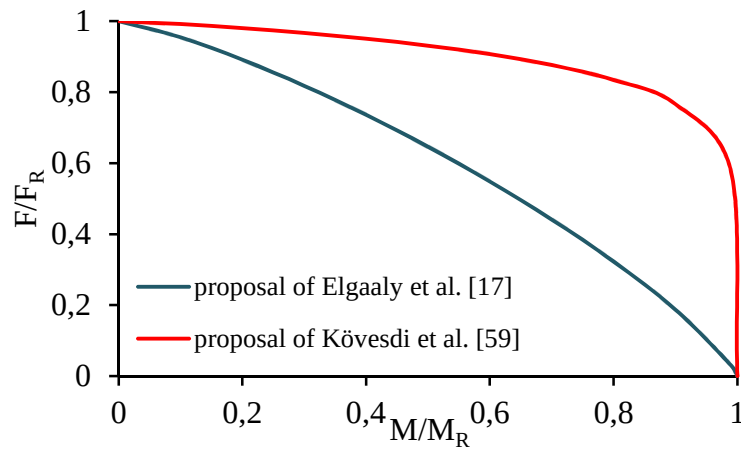


Fig. 10: Proposals for the M-F interaction curve.

Previous studies on the shear and patch loading interaction

Kähönen [16] proposed a formula for the interaction of shear and transverse force in 1988. The pure patch loading resistance was discussed theoretically based on experimental results of corrugated web girders and the combined loading situation was taken with a shear stress interaction factor (k_w) into account (see Eq. (2) in Section 2.2).

Elgaaly and Seshadri [17] investigated the shear and patch loading interaction of corrugated web girders in 1997. Five specimens were tested under local transverse force. The main aim of their research was to determine the patch loading resistance of corrugated web girders. The pure patch loading resistance and the interaction between shear and patch loading was studied within the frame of a numerical research programme in which the geometric parameter ranges of the analyzed girders were limited. Based on the numerical calculations, a “moving” interaction curve was proposed as shown in Eq. (36).

$$\left(\frac{V - 0,5F}{V_R}\right)^{1,25} + \left(\frac{F}{F_R}\right)^{1,25} \leq 1,0 \quad (36)$$

where V_R shear resistance,
 F_R patch loading resistance.

The proposal of Elgaaly and Seshadri was found to be appropriate enough based on the numerical investigation executed by Kövesdi et al. [62] in 2010. However the interaction curve was modified and a new lower limit interaction curve was provided by Kövesdi et al. based on their numerical results. The new interaction formula is demonstrated by the Eq. (37). The applied geometric parameter domain was wider than the analyzed parameter range of Elgaaly and Seshadri. In 2012 Kövesdi et al. [60], [61] presented the same proposal related to the girders with corrugated web subjected to combined shear and transverse force. In Fig. 11 the proposals for the V-F interaction diagram can be seen according to Elgaaly and Seshadri [17] and Kövesdi et al. [62].

$$\left(\frac{V - 0,5F}{V_R} \right)^{1,2} + \left(\frac{F}{F_R} \right)^{1,2} \leq 1,0 \quad (37)$$

where V_R shear resistance,
 F_R patch loading resistance.

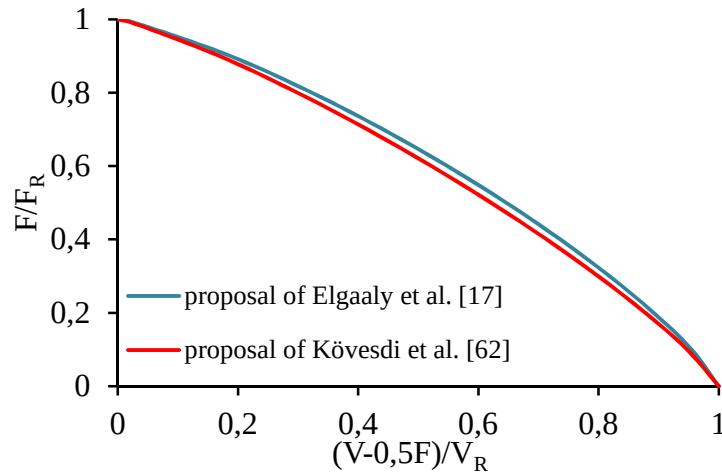


Fig. 11: Proposals for the V-F interaction curve.

It has to be mentioned that there is no recommendation in the EN1993-1-5 [1] for the calculation of the combined shear and patch loading interaction in case of corrugated web girders.

Evaluation of the proposals

EN1993-1-5 [1] Annex D provided formulas for the determination of the pure bending moment and shear buckling resistances of corrugated web girders with an adequate accuracy verified by several researchers. However there is no formula provided for the determination of the pure patch loading resistance of corrugated web girders in the Eurocode. There can be found some proposal regarding to corrugated web girders which is discussed in Section 2.2. In the recent MSc thesis the proposal of Kövesdi et al. [18] is used inspite of the fact that it was developed for bridge type girders and not for a general corrugated web girder geometry.

The combined bending moment and shear force interaction have been investigated by three research groups in the past regarding to trapezoidal corrugated web girders. At first Elgaaly et al. [4] investigated the combined bending and shear interaction of corrugated web girders and

concluded and stated that the interaction behaviour can be neglected. In contrast with that the EN1993-1-5 [1] Annex D recommends an interaction formula which predicts a significant interaction behaviour considering the additional normal stresses in the flanges coming from the shear flow in the web. This interaction formula was investigated by Kövesdi et al. [58] in 2014 and concluded that the proposal of the EN1993-1-5 is too conservative and the interaction behaviour on the numerically analyzed geometric parameter range was less than 5% with respect to the numerically calculated pure resistances but it can be neglected when adequate design formulas for the pure resistances are applied.

The Eurocode standard does not provide formulas for the determination of the combined bending moment and patch loading and combined shear force and patch loading interactions related to corrugated web girders. Both interactions have been studied by two research groups. At first Elgaaly and Seshadri [17] investigated the combined M-F and V-F interactions of trapezoidal corrugated web girders. Based on limited number of experimental tests and numerical investigation, proposals for both interactions were provided. On the basis of the numerical results of Kövesdi et al. [59] which was performed on a wider geometric parameter range, a new proposal for the M-F interaction behaviour of trapezoidal corrugated web girders was proposed. It was stated that the proposal of Elgaaly and Seshadri was too conservative. On the other hand the proposal for the V-F interaction curve of Elgaaly and Seshadri was found to be acceptable according to the numerical results of Kövesdi et al. [62] nonetheless a lower bound interaction curve was proposed with a small modification.

As a conclusion it can be stated that a small number of previous investigations are dealing with the combined loading situations of corrugated web girders from which the combined M-V-F interaction has never been investigated at all. Thus the investigation in this field must be of high priority.

3.3. Research strategy

In the previous sections the problems regarding to corrugated web girders are discussed with pointing out the most untapped topic in this field which is the investigation of the 3D interaction surface with respect to the interaction of combined bending moment, shear and transverse force. To solve this problem the individual interaction planes are studied at first. In the literature there can be found proposals for the individual interaction planes, namely M-V, M-F and F-V interaction planes, however there is no existing proposal for the M-V-F interaction surface. Therefore the research strategy of the current diploma thesis is the following:

- 1) An in-depth literature overview on the topic of the previous investigations on the M-V, M-F and F-V interaction behaviour are collected and discussed in detail in Section 3.2.
- 2) Advanced numerical models are developed with the help of the ANSYS v15.0 [101] finite element package using four-point shell elements considering the geometric and material nonlinearities and initial geometric imperfections and residual stresses. Different loading and boundary conditions are applied dealing with the individual interaction planes. (Section 4)

- 3) Based on the test results in the literature the numerical models are verified (Section 4.9).
- 4) Numerical parametric study is executed on the developed model using different geometric parameters to investigate at first the pure resistances (Section 5) and the individual interaction planes, namely M-V, M-F and V-F interaction planes (Section 6) and then the 3D interaction surface (Section 7).
- 5) There is no proposal existing for the M-V-F interaction domain, therefore at first the numerical results are compared with the composed 3D interaction surface from the previous proposals for the individual interaction planes. Then demanded correction can be made on the interaction domain (Subsection 7.2).
- 6) Statistical analysis is conducted to determine the applicability of the new M-V-F interaction equation (Subsection 7.3).
- 7) The comparison with the interaction behaviour of flat web girders is executed (Section 8).

After the execution of the aforementioned strategy the proposal for the determination of the M-V-F interaction surface is given for design purposes which is essential in case of an incrementally launched bridge structure with steel corrugated webs or in case of industrial buildings and halls with corrugated web girders.

4. Numerical model development and verification

The developed finite element models (geometric, mesh, load and boundary conditions) are introduced in this section. The finite element models are developed in ANSYS 15.0. The experimental background, the analysis type, convergence study, applied imperfections and the verification of the numerical models are also presented in this Section.

4.1. Experimental background

The behaviour of corrugated web girders under the combined loading situation are in the scope of investigation, therefore the verification of the numerical model with respect to the pure ultimate load carrying capacities, namely the bending moment, shear and patch loading capacities has high importance in this research program. To verify the advanced numerical model 16 available test results of specimens have been compared with the numerically computed results. Six bending tests of Elgaaly et al. [4], one shear test of Moon et al. [47], Hannebauer [13] and Driver et al. [44], two patch loading tests of Elgaaly and Seshadri [17] and five patch loading tests of Kövesdi et al. [20] were analyzed numerically and compared with the ultimate load carrying capacities of the test results. The evaluation of the comparison can be seen in Section 4.9.

4.2. Geometric model

The geometrical model with the defined surfaces can be seen in Fig. 12 in which four corrugations are applied with inclined web folds at both ends of the beam. In this case the length of the beam is 4800 mm, thus the length of one corrugation is 1200 mm in which the length of the parallel fold is 400 mm and the depth of the corrugation is 200 mm with an angle of 45° .

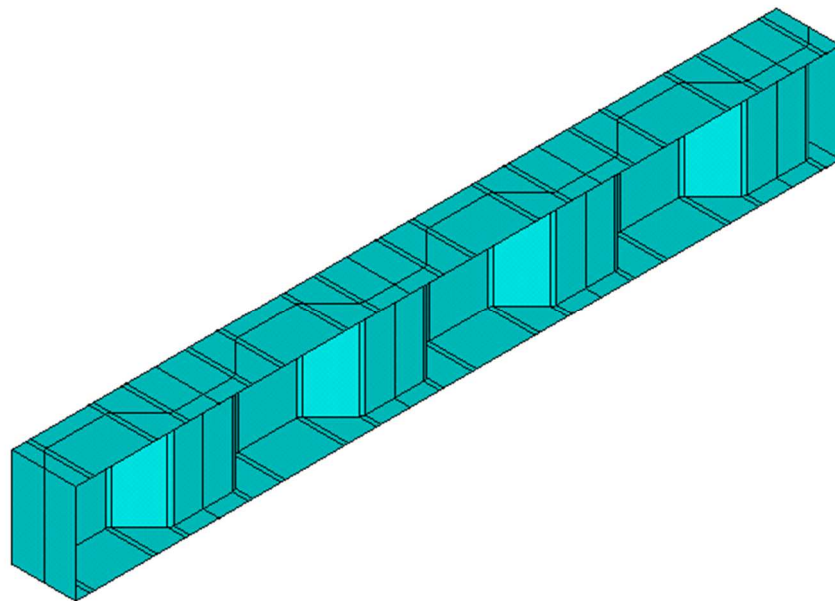


Fig. 12: Geometric model of corrugated web girders.

4.3. Material model

The character of the applied material model are a linear elastic - hardening plastic material model using von Mises yield criterion. This multilinear material model has in the plastic domain an isotropic hardening behaviour. The material is assumed to behave linearly elastic and to obey Hook's law with a Young' modulus equal to 210000 MPa up to the yield stress (f_y). The yield plateau is modelled up to 1% strains. Thereafter and until it reaches the ultimate stress (f_u), the material is assumed to behave linear hardening with a reduced modulus. The ultimate strength is defined by 12% strains. The material is perfectly plastic when it reaches the ultimate stress.

Different yield and ultimate stress values are applied depending on the analyzed girders. For the verification of the numerical model the measured values of the yield and ultimate stresses are contained in the 9th and 10th columns of Table 1, Table 2 and the 2nd, 3rd, 4th and 5th columns of Table 4, respectively (Subsection 4.9). In the numerical parametric study the characteristic value of the yield and ultimate strength are used in the computations instead of the measured values. The 9th and 10th columns of Table 6 contains the yield and ultimate stresses used in the parametric study (Section 5.1).

4.4. Finite element mesh

Figure 13 presents the finite element mesh. In the finite element model the SHELL181 element was applied which can be seen in Fig. 14.

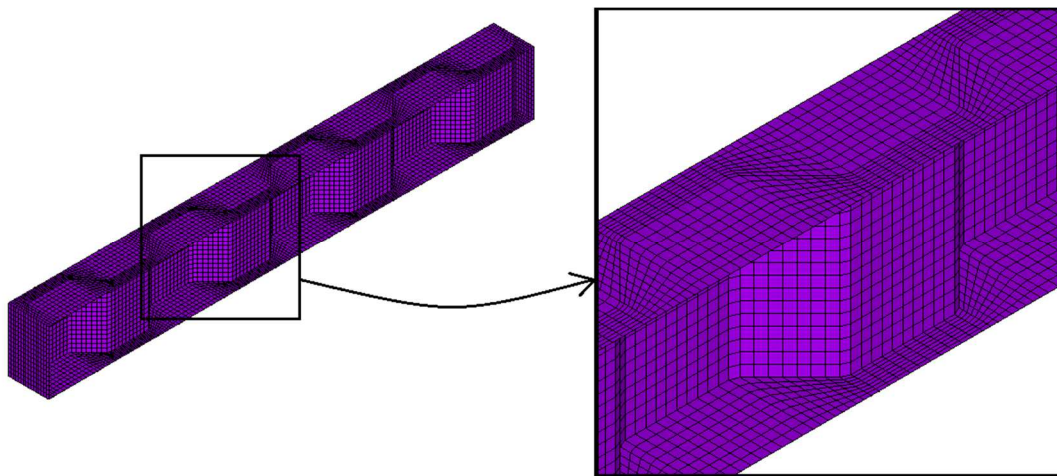


Fig. 13: Finite element mesh.

The attributes of the SHELL181 finite element are the followings:

- four-node thin shell element,
- bending and membrane capabilities, so both in-plane and normal loads are permitted,
- six degrees of freedom at each node, translations in the nodal x, y and z direction and rotations around the nodal x, y and z-axis,
- large strain and large deflection capabilities are also included,
- it can be applicable for stability analysis (critical stress level determination),
- applicable for linear or nonlinear analyses.

In this geometry with mesh size 40 mm the number of applied nodes is 8026, thus the degrees of freedom of the whole model is 48156.

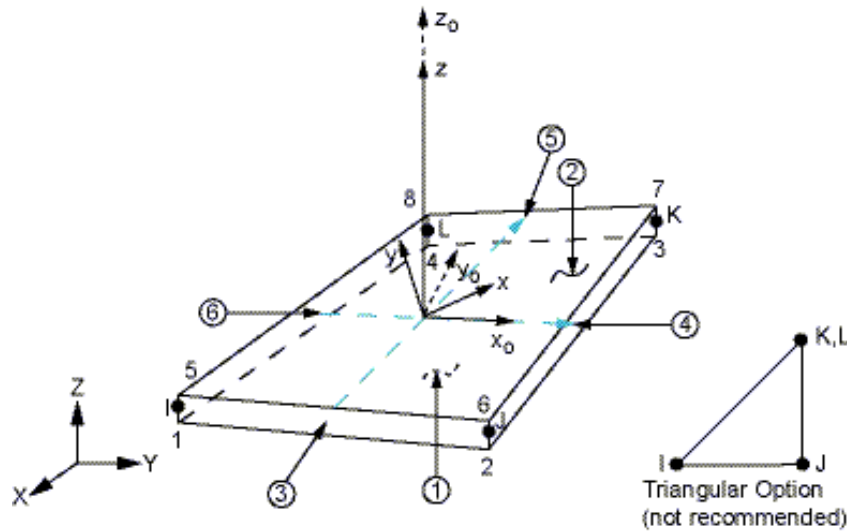


Fig. 14: SHELL181 finite element [101].

4.5. Loads and boundary conditions

The different boundary and loading conditions of the finite element model specialized for the investigation of each interaction planes and the interaction surface are presented. Different boundary and loading conditions have been applied for the investigation of each M-V, M-F, V-F and M-V-F interaction behaviour of corrugated web girders. In case of all girders diaphragms are placed to the ends of a beam. Fig. 15 shows the boundary and loading conditions in case of the investigation of the bending moment and shear force interaction behaviour.

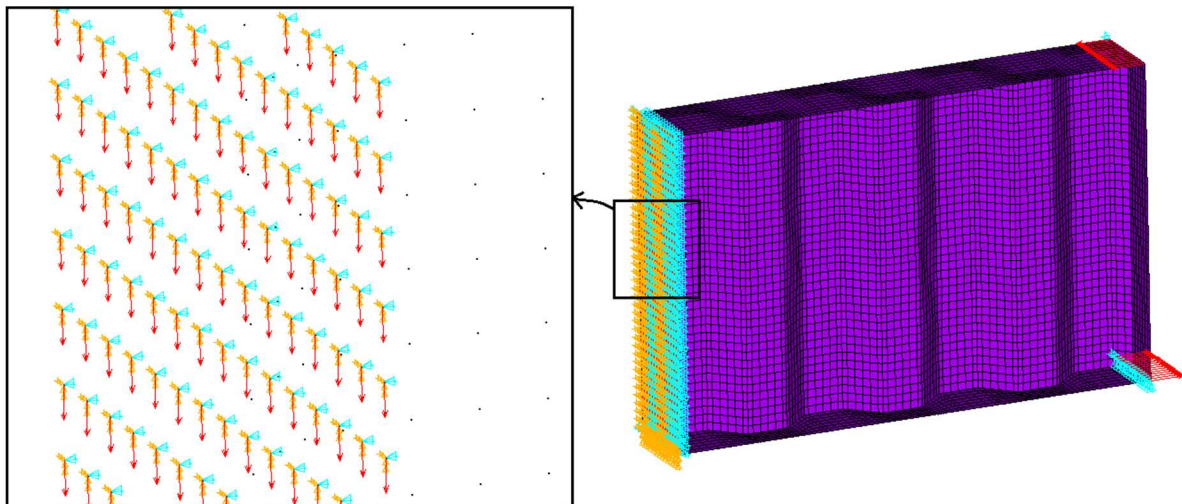


Fig. 15: Boundary conditions in case of the investigation of the M-V interaction.

It is assumed that the virtual girder is subjected to three-point bending with accompanying bending moments acting at the ends of the girder. One can observe in Fig. 15 that only the half of the virtual girder is modelled in order to reduce the computational time. The shear force is distributed and applied on the left end diaphragm and the bending moment is applied as

equivalent normal forces distributed in the flanges on the right end of the girder. On the left hand side the girder is supported against displacement in the longitudinal direction and rotation around the transverse and lateral directions, on the right hand side the girder is supported in the transverse and lateral directions.

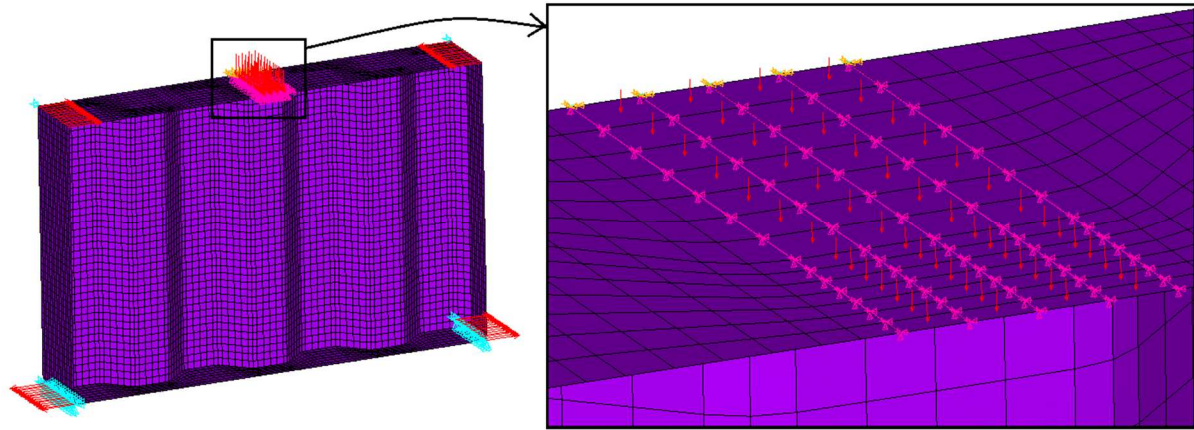


Fig. 16: Stiffened flange model for the investigation of the M-F interaction.

For the investigation of the bending moment and patch loading interaction the applied boundary and loading conditions can be seen in Fig. 16. The girder is subjected to three-point bending with accompanying bending moments acting on both ends. It has to be emphasized that stiffened flange model is used during the parametric study which excludes the local flange buckling due to patch loading. In the case of bridge launching flange buckling can not occur, because of the stiffening effect of the concrete flange and/or loading device. Thus, for the numerical analysis the nodes of the flange have been coupled in the lateral direction with rigid elements. The rotation of these rigid lines are then restrained around the longitudinal and lateral directions to avoid flange buckling. In this way the stiffening effect of the concrete flange and/or loading device is taken into account and the failure mode remains pure web crippling or local web buckling (Fig. 16).

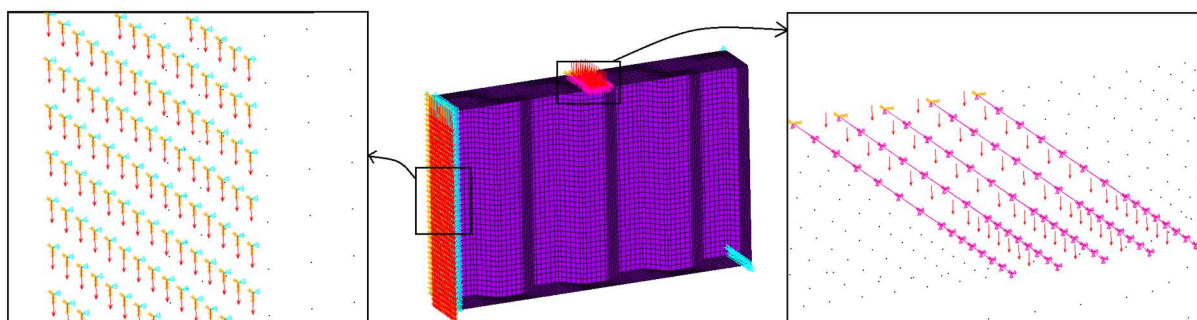


Fig. 17: Boundary conditions in case of the investigation of the V-F interaction.

In Fig. 17 the boundary and loading conditions are shown in case of the investigation of the shear and transverse force interaction behaviour. One can observe that on the left diaphragm the shear force is acting while in the midspan the patch load is acting. On the right hand side the girder is supported vertically, on the left hand side the girder is supported longitudinally and against rotation around the transverse and lateral directions. Stiffened flange model is used during patch loading (Fig. 17).

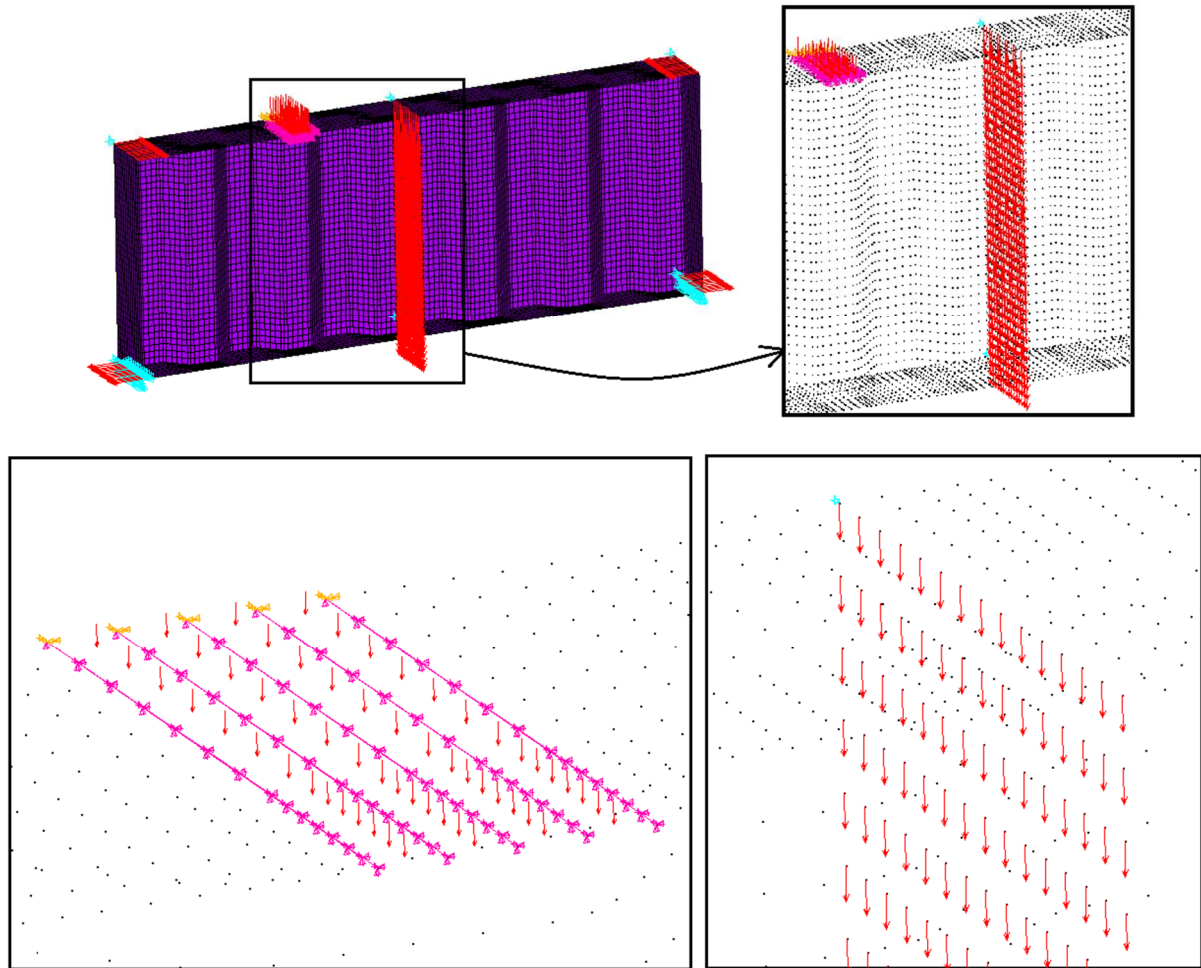


Fig. 18: Boundary conditions in case of the investigation of the M-V-F interaction.

In Fig. 18 the boundary and loading conditions are shown in case of the investigation of the M-V-F interaction behaviour. The girder is simply supported and subjected to bending moments at both ends. The shear force is applied on the midspan diaphragm while the patch load is acting close to the quarter point of the beam placed above a parallel web fold. The beam is supported laterally at the end points and at the midspan.

4.6. Analysis type

The analyzed numerical models are developed using the finite element software ANSYS 15.0 [101]. The numerical modelling is based on a full shell model using four node thin shell elements. The ultimate loads under the combined loading situations are determined by geometrical and material nonlinear analysis using equivalent geometric imperfections (GMNIA). The full Newton-Raphson approach is used in the nonlinear analysis since it is well-suited for handling large deformations. The nonlinear solver convergence was analyzed using different time stepping attributes such as the time step size and maximum and minimum time step sizes. Due to the convergence study the time step sizes are set properly in the numerical simulations. During the simulations the default convergency criteria built in ANSYS is used.

4.7. Applied imperfections

The aim of the current research is to determine the resistances of the pure and combined loading situations under bending, shear and transverse force of trapezoidal corrugated web girders. As it is discussed in Section 2 the pure failure modes in most of the cases are governed by buckling, thus the initial imperfection has an important role in the numerical computation procedure. Initial imperfections can be geometric and structural imperfections (residual stresses) from which both can be taken with equivalent geometric imperfections into account. There are three different alternatives to define the equivalent geometric imperfection which are allowed according to the EN1993-1-5 [1].

- 1) The first possibility is to apply manually the commonly observed imperfection shapes after the fabrication processes. In this case the imperfection shape is practically defined by a functional description such as sine function or other periodic functions. This approach is the most complex one in comparisons with the other types.
- 2) The second possibility is to apply the eigenmode imperfections. The standard allows using the first eigenmode as an imperfection shape, which corresponds to the buckling shape of the analyzed panel. The eigenmodes are based on mathematical background so it is fully objective and the modelling technique has been already well clarified. The main advantage is that the usage of the first eigenmode imperfection shape leads to a safe design.
- 3) The third one is the ultimate failure shape as a possible imperfection shape type. In this case the applied imperfection shape comes from the previous geometrically perfect computation (GMN analysis) then the originally perfect geometry is updated by the ultimate failure shape of the structure as an imperfect shape. This imperfection type is not a commonly used imperfection may be because it is time-consuming and may lead to a too conservative design approach.

In most of the cases the initial imperfections of the specimens are not known for the verification of the developed numerical model. The measuring procedure is very time-consuming and does not contain the residual stresses. Using periodic functions as equivalent geometric imperfections can be a possible solution, but it leads to a very complex and slow approach which requires a high degree of attention. Therefore it is not suitable for parametric study and for combined loading situations.

Using the ultimate failure shape as equivalent initial imperfection is also very time-consuming approach because all of the nonlinear computations have to be performed at least twice to determine the appropriate ultimate failure shape. Furthermore it is very rare that the applied imperfection coincide with the ultimate failure shape imperfection, thus this approach leads to the most conservative design.

The application of the first eigenmode shape is the most common one because it contains the relevant failure mode. In the current research work different failure modes are studied separately and also on a combined way, the best and always a safe side solution is ensured by

using eigenmode imperfections. This imperfection type can handle the change of the failure mode in the interaction domain. Maybe the usage of the first eigenmode imperfection shape leads to a conservative design, but it can be assured that all of the calculations are on the safe side. Therefore all the numerical parametric studies are executed using the first eigenmode imperfection shape. The first eigenmodes due to pure bending, shear and transverse force can be seen in Fig. 19.

The imperfection amplitudes for the pure resistances are discussed in some papers and in the EN1993-1-5 [1]. In case of pure patch loading there is no recommendation in the EN1993-1-5, therefore an imperfection sensitivity analysis was performed by Kövesdi et al. [21]. In this diploma thesis the first eigenmode (web crippling), the sine-wave, the ultimate shape and the modified sine-wave imperfections were investigated and compared. In case of the modified sine-wave imperfection the amplitude of the sine-wave is the highest at the web crippling zone and descending away from the attacked flange. In conclusion of that paper the fold length divided by the scaling factor equal to 200 is proposed as imperfection magnitude in case of the application of the first eigenmode imperfection.

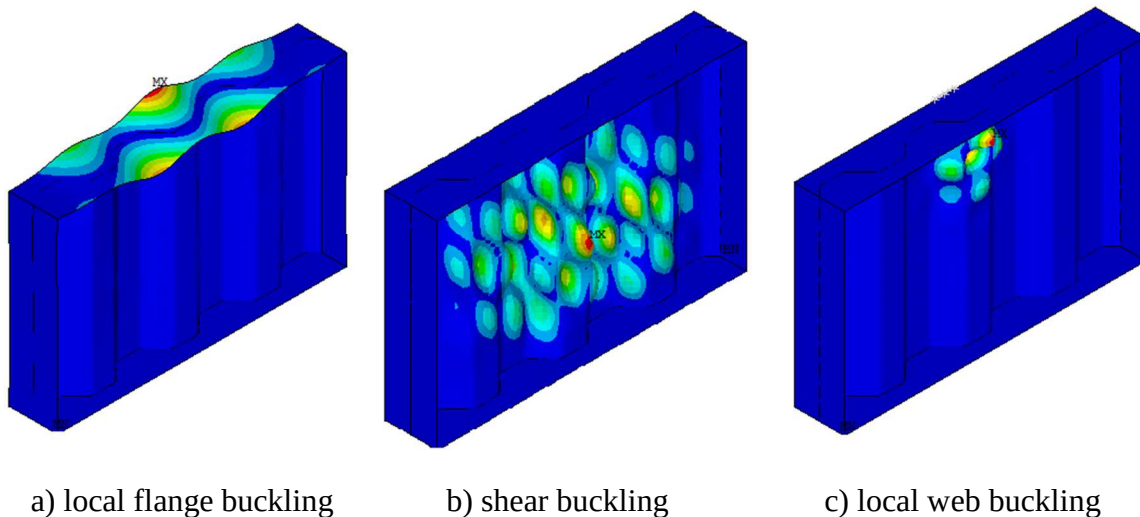


Fig. 19: First eigenmodes due to pure a) bending, b) shear force and c) patch loading.

There is no recommendation in the EN1993-1-5 [1] for the imperfection amplitude for corrugated webs. However imperfection sensitivity analysis has been also carried out by different researchers investigating the pure shear buckling resistance of corrugated web girders. Driver et al. [44] proposed that the magnitude of the applied imperfection amplitude can be taken as the thickness of the web which was confirmed by Hassanein et al. [52]. Another magnitude for the imperfection amplitude was proposed by Yi et al. [46] and Nie et al. [50]. They suggested using web depth divided by the scaling factor equal to 200 as equivalent initial geometric imperfection amplitude. The last one is used because in most of the cases it gives greater imperfection magnitudes.

In case of pure bending the recommendation of the EN1993-1-5 Annex C [1] for local panel or subpanel buckling is used in which the flange greatest outstand divided by the scaling factor

equal to 200 is applied. This magnitude is applied in the parametric study on the M-V interaction behaviour by Kövesdi, Jáger, Dunai [58].

Even though an imperfection sensitivity analysis is performed using the eigenmode imperfection shape. As a result, the previously discussed proposals are judged to be appropriate for further use. During the numerical analysis of the combined loading situations the appropriate imperfection magnitude is applied with respect to the dominant first eigenmode.

4.8. Convergence study

Mesh convergency study is executed to ensure the model reliability. According to Moon et al. [76] one web fold should be modelled by minimum of 3-4 elements along the fold length to determine the capacity of the girder with adequate accuracy. Due to the imperfect analysis as a principle 6-10 elements have to be used to ensure accuracy. It means that in case of the analyzed girders the applied average finite element size varies between 5 - 40 mm.

For representing the process of the convergence study, one geometry is chosen which can be seen in Section 4.2 (Fig. 12) and 4.4 (Fig. 13). Proving the accuracy of the numerical model different mesh sizes were applied and analyzed. During the investigation the computational time is also measured. Figure 20 shows that using smaller mesh sizes the obtained values for the numerically computed bending moment resistance tends to the accurate solution.

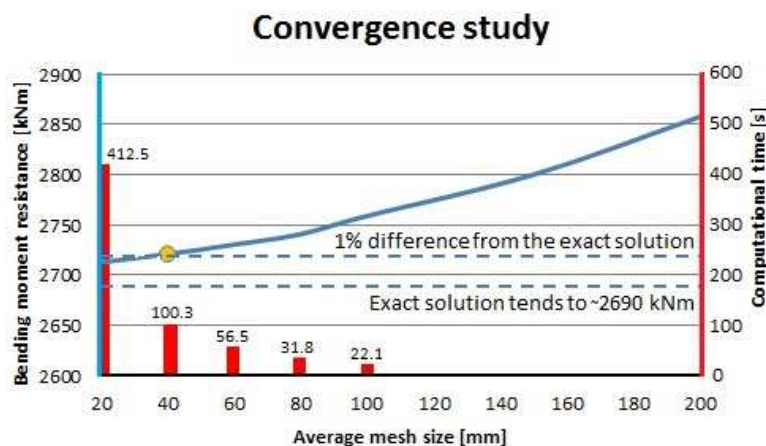


Fig. 20: Convergence study of pure in-plane-bending.

The calculated bending moment resistances by different mesh sizes are represented by the blue line. According to the Microsoft Excel software in this case the exact solution tends to the bending moment resistance equal to 2690 kNm which is demonstrated by the lower dashed line. The one percentage difference from the assumed exact solution is demonstrated by the upper dashed line. The intersection point of the upper dashed line with the continuous blue line is close to the mesh size 40 mm which is marked by a yellow dotted line in Fig. 20. It is shown by red columns that the computational time is more time-consuming in case of mesh size 20 mm and this is the main reason why mesh size 40 mm was judged to be accurate enough for the application in the numerical computation.

4.9. Verification of the numerical model

In this Section the verification of the numerical model is presented. Tests of different researchers available in the international literature are used for the verification of the finite element model. In the followings the corresponding pure ultimate load carrying capacities of the numerical models of the specimens, namely the numerically computed bending moment, shear buckling and patch loading capacities of the corresponding specimens are shown separately and compared with the test results.

Verification of the bending moment resistance

Six bending tests of Elgaaly et al. [4] were analyzed numerically and compared with the ultimate load carrying capacities of the test results. In Fig. 21 the numerical model of Elgaaly's specimen (Fig. 21/a) and its typical bending failure mode (Fig. 21/b) can be seen.

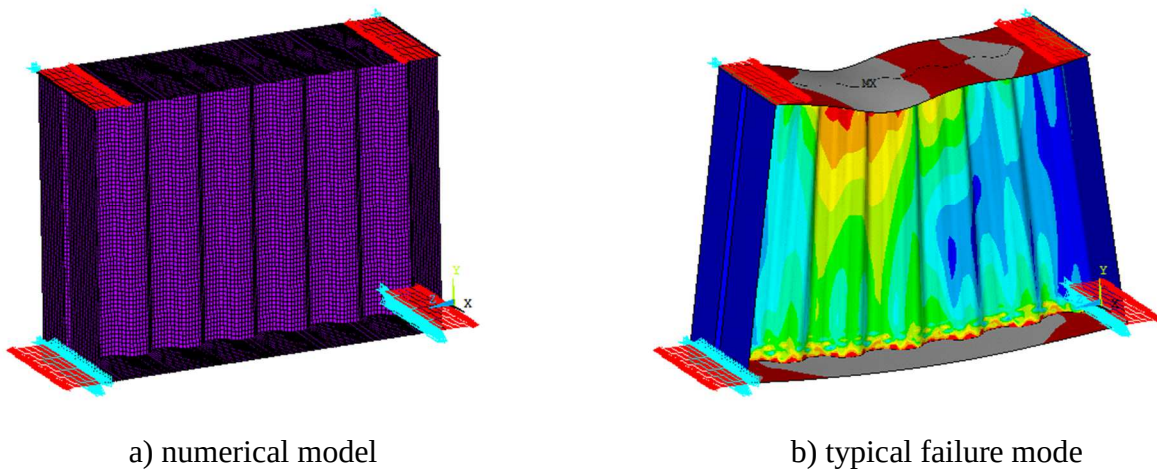


Fig. 21: Elgaaly's specimen and its typical bending failure illustrating the von Mises stresses.

In Table 1 the geometric parameters of the six specimens can be seen with the average measured yield and ultimate stresses (9th and 10th column) which were applied in the numerical material model (Section 4.3). The 11th column and 12th column of Table 1 contain the experimentally measured (M_{test}) and the numerically computed ultimate bending moment capacities (M_{num}), respectively. The 13th column contains the comparison of the aforementioned capacities.

Table 1: Comparison of the experimental and numerical results.

	t_f [mm]	b_f [mm]	h_w [mm]	t_w [mm]	α [°]	a_1 [mm]	a_2 [mm]	f_y [MPa]	f_u [MPa]	M_{test} [kNm]	M_{num} [kNm]	M_{test} / M_{num}
Bending tests of Elgaaly et al. [4]												
1	12,7	152,4	304,8	0,607	50	19,80	18,53	289	682	180,9	183	0,99
2	12,7	152,4	304,8	0,607	45	38,10	35,92	289	682	193,3	182,3	1,06
3	12,7	152,4	304,8	0,607	55	41,90	40,70	289	682	175	178	0,98
4	12,7	152,4	304,8	0,607	62,5	49,80	57,25	289	682	175,2	177,9	0,99
5	12,7	152,4	304,8	0,607	55	41,90	40,70	376	682	237,8	234,9	1,01
6	12,7	152,4	304,8	0,607	62,5	49,80	57,25	376	682	223,2	232,9	0,96

The results show that the maximum difference is 4% (0,96) on the unsafe and 6% (1,06) on the safe side. It may be attributed to the measurement devices using in the experiments and also to the developed numerical model and to the different applied imperfections. Despite the differences of the measured and computed results a good agreement can be observed, therefore the numerical model is judged to be accurate enough for the application for the further numerical parametric study.

Verification of the shear buckling resistance

One shear test of Moon et al. [47], Hannebauer [13] and Driver et al. [44] were analyzed numerically and compared with the ultimate load carrying capacities of the test results. In Fig. 22 the numerical model of Moon's specimen (Fig. 22/a) and its shear buckling failure mode (Fig. 22/b) can be seen. The real failure mode of the Moon's specimen is shown in Fig. 23.

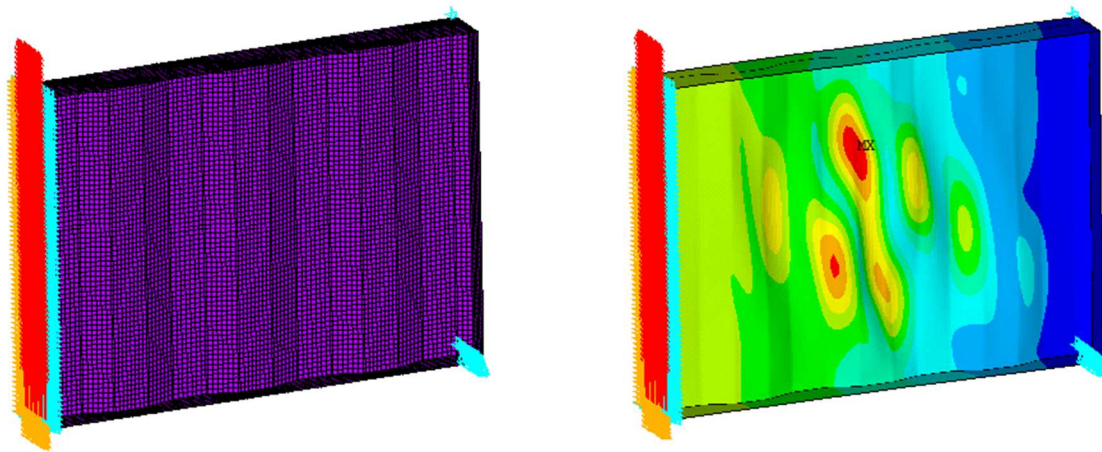


Fig. 22: Numerical model of the half of Moon's specimen and its shear buckling failure.

In Table 2 the geometric parameters of the three specimens can be seen with the average measured yield stresses (9th column) which were applied in the numerical material model (Section 4.3). The 11th column and 12th column of Table 2 contain the experimentally measured (V_{test}) and the numerically computed ultimate shear capacities (V_{num}), respectively.



Fig. 23: Moon's specimen after shear buckling failure [47].

Table 2: Comparison of the experimental and numerical results.

	t_f [mm]	b_f [mm]	h_w [mm]	t_w [mm]	α [°]	a_1 [mm]	a_2 [mm]	f_y [MPa]	f_u [MPa]	V_{test} [kN]	V_{num} [kN]	V_{test} / V_{num}
Shear test of Moon et al. [47]												
1	30	300	2000	4	23	220	195	296	-	1053	1007	1,05
Shear test of Hannebauer [13]												
2	15	200	500	2,5	40	30	62	270	-	206,5	193,2	1,07
Shear test of Driver et al. [44]												
3	50	450	1500	6,3	37	300	250	485	-	2155	2009	1,07

The results show that the maximum difference is 7% (1,07), and all of the numerically computed results are on the safe side, thus the numerical model is judged to be appropriate enough for the application for the further numerical parametric study.

Verification of the patch loading resistance

Two patch loading tests of Elgaaly and Seshadri [17] and five patch loading tests of Kövesdi et al. [20] were analyzed numerically and compared with the ultimate load carrying capacities of the test results. In Fig. 24 the numerical model of Kövesdi's specimen (Fig. 24/a) and its local web buckling failure mode (Fig. 24/b) can be seen. The real failure mode of the Kövesdi's specimen is shown in Fig. 25.

By using unstiffened flange model in the numerical analysis the collapse mode is not only pure web crippling or local web buckling, but also the interaction of the web and flange buckling. This collapse mode corresponds to the experiments of Elgaaly and Seshadri [17]. During incremental launching of superstructures the flange buckling can not occur, because of the stiffening effect of the concrete flange and/or loading device and/or the lateral restraint of the bridge girder above the bearings. Thus, for the numerical analysis the nodes of the flange have been coupled in the lateral direction with rigid elements (Figs. 16 and 17 in Subsection 4.5) which is called stiffened flange model. The rotation of these rigid lines are then restrained to avoid flange buckling. In this way the stiffening effect of the concrete flange and/or loading device and/or the lateral restraint of the bridge girder above the bearings are taken into account and the failure mode remains pure web crippling or local web buckling.

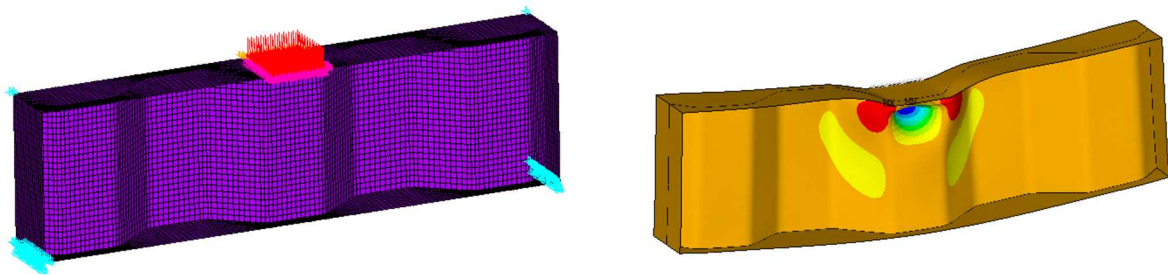


Fig. 24: Numerical model of Kövesdi's specimen and its local web buckling failure.



Fig. 25: Local web buckling of Kövesdi's specimen [20].

Table 3: Geometric parameters of the test specimens.

	Location of load	L [mm]	h_w [mm]	t_w [mm]	b_f [mm]	t_f [mm]	a_1 [mm]	a_2 [mm]	α [°]	s_s [mm]
Patch loading tests of Elgaaly and Seshadri [17] using unstiffened flange model										
1	Parallel fold	750	376	2	120	10	130	157	40	146
2	Inclined fold	750	376	2	120	10	130	157	40	104
Patch loading tests of Kövesdi et al. [20] using stiffened flange model										
3	Inclined fold	1500	500	6	225	20	210	212	39	90
4	Parallel fold	1875	500	6	225	30	210	212	39	200
5	Parallel fold	1875	500	6	225	20	210	212	39	200
6	Parallel fold	1140	500	6	225	20	210	212	39	200
7	Parallel fold	1875	500	6	225	20	210	212	39	380

In Table 3 the geometric parameters of the seven specimens are presented. In Table 4 the average measured yield and ultimate stresses of the flanges and the web can be seen (2nd to 5th columns) which were applied in the numerical model (Section 4.3). The 6th column and 7th column of Table 4 contain the experimentally measured (F_{test}) and the numerically computed ultimate patch loading capacities (F_{num}), respectively. The 8th column contains the comparison of the aforementioned capacities.

The results show that the maximum difference is 1% (0,99) on the unsafe and 5% (1,05) on the safe side. The Table 4 shows that almost all of the numerically computed results are on the safe side, thus the numerical model is judged to be appropriate enough for the application for the further numerical parametric study.

Table 4: Results of the tests and the numerical analyses.

	f_{yf} [MPa]	f_{uf} [MPa]	f_{yw} [MPa]	f_{uw} [MPa]	F_{test}	F_{num}	F_{test}/F_{num}
1	389	563	379	413	131,28	125,54	1,05
2	389	563	379	413	102,35	103,16	0,99
3	379	517	373	542	754,20	715,77	1,05
4	379	517	373	542	1192,01	1182,41	1,01
5	379	517	373	542	949,02	944,46	1,00
6	379	517	373	542	1077,72	1064,55	1,01
7	379	517	373	542	1280,99	1235,58	1,04

5. Validity of the pure resistance formulas

In the current section the pure resistances, namely the bending moment, shear buckling and patch loading resistances of corrugated web girders are investigated. Several researchers were dealing with the investigation of the pure resistances. The developed design formulas for each resistances used in this thesis are presented in Section 3.2. The main question of this section is the determination of the correctness of these proposals to make sure that its further use is reasonable or not.

5.1. Numerical research program, strategy and goals

Experimentally and numerically investigated girder geometries have been collected from the international literature into a created and developed database by Kövesdi, Jäger, Dunai [58] in 2013. In the frame of the study program of Kövesdi, Jäger, Dunai [58] typical girder geometries with trapezoidal and sinusoidal corrugated webs were collected and evaluated as different classes of the corrugation layouts. An equivalent trapezoprofile are created in case of sinusoidal web corrugation to evaluate the geometrical properties. The evaluation was carried out separately for trapezoidal and sinusoidal corrugated web girders.

The created database contains total of 1941 number of different girder geometries which composed from the above classes of corrugation layouts:

1. 1522 sinusoidal corrugated web girder geometries from the catalog of the company Zeman [102], from the paper of Hannebauer [13] and Kuchta [57],
2. 407 experimentally and numerically analyzed trapezoidal corrugated web girder geometries,
3. 12 different girder geometries from existing bridge structures with trapezoidal corrugated webs.

In case of several bridges only the geometry of the corrugation profile and the web depths are available from the literature. In these cases fictitious flange sizes are assumed, which are usually used for these kind of bridges.

Table 5: Evaluation of the geometries collected from the international literature [58].

	corrugation angle (α) [°]			fold length a_1 [mm]			fold length a_2 [mm]		
	min	max	average	min	max	average	min	max	average
Sinusoidal corrugation	20	38	37,9	26	100	26,5	65	220	66,1
Trapezoidal corrugation	10	90	39,4	20	750	245,1	14	707	237,1
Existing bridges	25	39	33,3	250	430	334,8	250	430	340,3

The statistical evaluation of the different classes of corrugation layouts, namely the sinusoidal corrugation and trapezoidal corrugation profile of the existing bridges can be seen in Table 5. It can be observed in Table 5 that in comparison of the average values regarding to the three classes the differences between the fold lengths are significant while the differences between the corrugation angles is small .

From the database 11 typical bridge and 9 typical building type geometries have been chosen for the analysis of the ultimate load carrying capacity (current Section) and the interacting stability behaviour (Section 6 and 7) of corrugated web girders. The analyzed geometries and material properties are summarized in Table 6.

The numerical model is verified by test specimens from the international literature (Section 4.9), therefore it is applicable for the determination of the in-plane bending, shear buckling and patch loading resistances of the analyzed girders.

Table 6: Analyzed girder geometries in the numerical parametric study.

	b_f [mm]	t_f [mm]	h_w [mm]	t_w [mm]	α [°]	a_1 [mm]	a_2 [mm]	f_y [MPa]	f_u [MPa]
Typical bridge geometries									
1	300	30	2000	4	23	220	200	296	460
2	300	20	1000	4	45	150	150	355	510
3	400	40	1500	6	45	200	200	355	510
4	400	40	1000	10	45	200	200	355	510
5	450	40	2500	9	32	330	330	355	510
6	450	40	2500	9	45	330	330	355	510
7	450	40	2500	9	60	330	330	355	510
8	300	20	2500	9	32	330	330	355	510
9	300	30	2500	9	32	330	330	355	510
10	350	30	2000	8	32	330	330	355	510
11	350	30	1200	9	36,8	250	250	355	510
Typical building geometries									
12	200	15	500	2,5	40	30	62	270	430
13	300	20	1000	4	45	50	50	355	510
14	250	16	750	5	45	65	65	355	510
15	250	16	750	5	60	65	65	355	510
16	250	16	750	5	30	65	65	355	510
17	250	16	750	5	45	120	120	355	510
18	250	16	750	5	45	200	200	355	510
19	250	16	750	5	45	40	40	355	510
20	220	12	1000	1,5	45	140	70	270	360

The parameter ranges used in this diploma thesis slightly differs from the parameter ranges used in [59, 60, 61, 62]. By this research program the analyzed parameter ranges are extended as follows:

- h_w/t_w : 100 – 500
- a_1/t_w : 8 – 93,33
- α : 23° – 60°
- s_s/h_w : 0,042 – 0,865

During the investigation the applied geometric layouts of the corrugated web are shown in Fig. 26. According to Kövesdi, Jáger, Dunai [10] the most unfavourable layouts of the girder are when it is supported under the inclined fold of the corrugated web. Figure 26 shows these cases with odd number of half corrugation waves (girder type 3) and with even number of half corrugation waves (girder type 4). In this case the additional transverse bending moment in the flanges coming from the shear flow in the web is maximized (Fig. 3/a and 3/b in Section 2.1). It was stated in [10] that the most unfavourable one is the girder type 4, but girder type 3 has been used in case of patch loading since in this diploma thesis the patch load is acting at the midspan of the girder above the parallel web fold regarding to M-F loading.

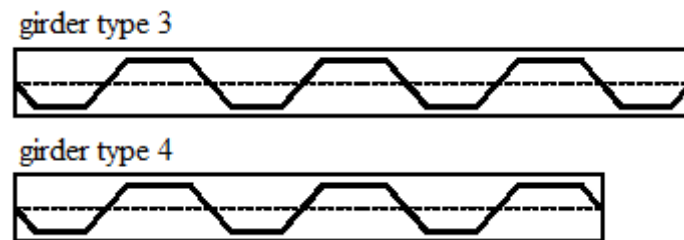


Fig. 26: Geometric layouts with inclined web folds (view from above) [10].

For each of the 20 girders the individual ultimate capacities are computed numerically and compared with the analytically calculated resistances of the design proposals of the EN1993-1-5 [1] Annex D in case of bending moment and shear force (Eq. (29) and (30) in Section 3.2) and Kövesdi et al. [18] in case of patch loading (Eq. (31) in Section 3.2). All of the girders were loaded under three different patch loading lengths which were between $\sim 0,5$ to $\sim 3,5$ of the parallel fold length (a_1) depending on the meshing. Thus every girder in Table 6 was analyzed in total of five times, one time under pure bending moment, one time under shear force and three times under transverse force applying different loading lengths. Therefore in the current section a total number of 100 numerical computations had been carried out.

With the help of the results statistical evaluations are executed to determine the accuracy of each proposals of the pure resistances for the analyzed parameter ranges.

5.2. Investigation of the bending resistance

Twenty girder geometries which are included in Table 6 are analyzed under pure bending. The typical observed failure mode can be seen in Fig. 27 where the failure occurs in the compression flange subpanel at the greatest outstand. This phenomena is the so called local buckling of the flange in the largest outstand of the flange subpanel from the web-to-flange weld toe to the free edge of the flange.

After the sensitivity analysis and calibration of the imperfection magnitude, during the numerical computation scaling factor equal to 200 was used on the greatest outstand of the flange as it is proposed in the EN1993-1-5 [1] Annex C for panel or subpanel buckling. Of course this imperfection magnitude was applied in the verification procedure as well (Table 1 in Section 4.9). In case of bridge type girders partial factor $\gamma_{M1}=1,10$ and in case of building type girders partial factor $\gamma_{M1}=1,00$ is substituted in the denominator of the Eq. (29) (Section 3.2).

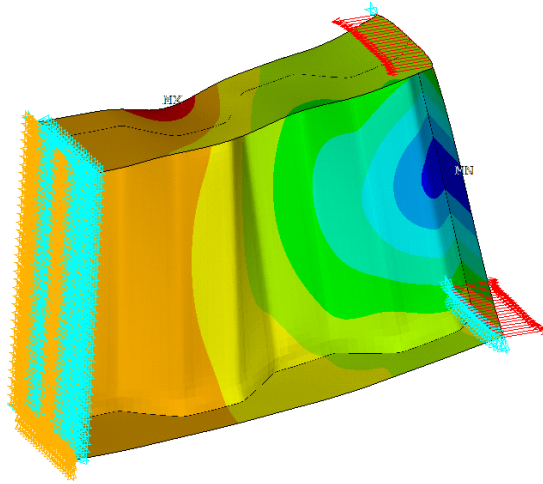


Fig. 27: Observed failure mode under pure bending.

Figure 28 shows the comparison of the results. The horizontal axis characterizes the amplitude of the corrugation profile with the ratio of the flange width divided by the corrugation depth (b_f/a_3) while the vertical axis represents the ratio of the numerically computed and analytically calculated resistances. One can observe that only one girder geometry is on the unsafe side with 0,986 ratio. This one is the girder number 18 in Table 6 (Section 5.1).

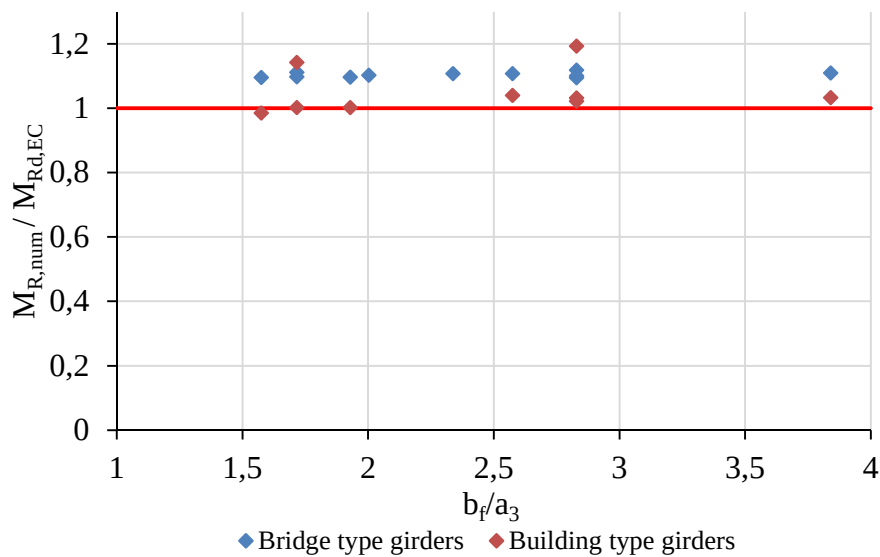


Fig. 28: Comparison of the numerical and analytical solutions.

The statistical evaluation of the results can be seen in Table 7. The average ratio is 1,08 with standard deviation equal to 0,053 which means that the recommendation of the EN1993-1-5 [1] is adequate enough for the further use. According to the Student's T-test distribution the reliability interval is $8 \pm 2,5\%$ on the safe side using 95% confidence interval with two-tails 2,5% upper and lower fractile of the analyzed sample.

Table 7: Statistical evaluation of the numerical results.

Average	1,081
Standard deviation	0,053
Coefficient of variation	0,049
Minimum	0,986
Maximum	1,194
Reliability interval of the mean considering normal distribution using 2,5% upper and lower fractile	0,023
Reliability interval of the mean considering Student's T-test distribution using 2,5% upper and lower fractile	0,025

5.3. Investigation of the shear buckling resistance

The same twenty girder geometries which are included in Table 6 were analyzed under shear force. In case of shear buckling three types of failure modes are existing, namely local buckling of the individual web folds (Fig. 29/a), global buckling of the whole web panel (Fig. 29/c) and interactive shear buckling (Fig. 29/b) which is the combination of the aforementioned two phenomena. Detailed description can be read in Section 2.2 about the shear buckling phenomena of corrugated web girders.

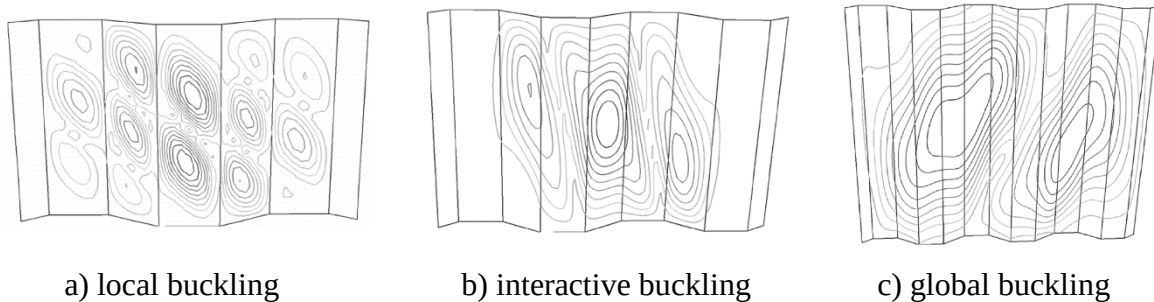


Fig. 29: Typical shear buckling modes [48].

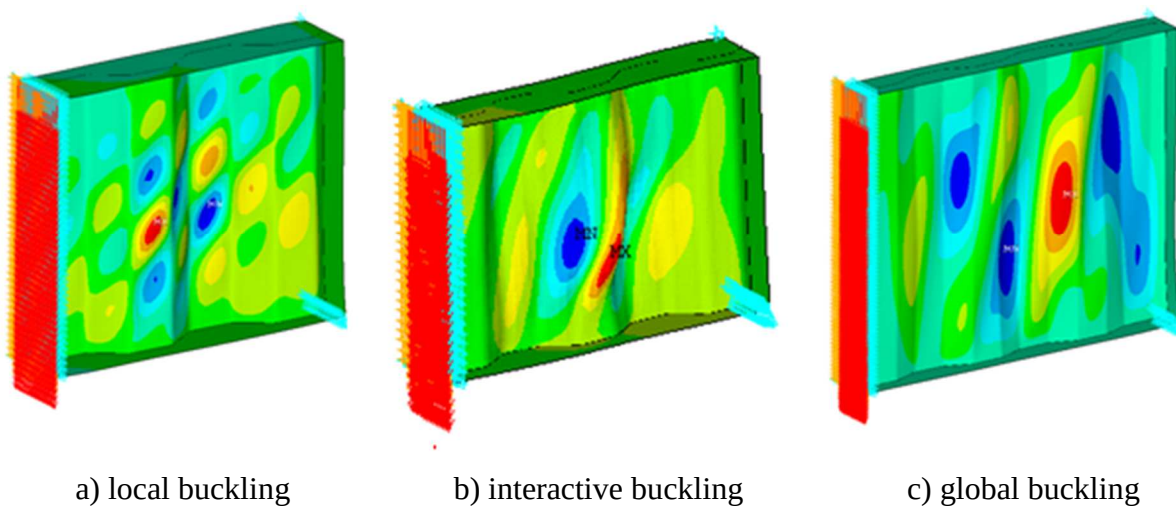


Fig. 30: Observed failure modes under shear force.

The typical observed failure modes can be seen in Fig. 30. In the Fig 30/a typical local shear buckling of the individual web folds can be observed while Fig. 30/c demonstrates the global

shear buckling of the web panel and Fig. 30/b represents the interactive shear buckling of the adjacent web folds. The type of shear buckling failure mostly depends on the corrugation profile.

After the sensitivity analysis and calibration of the imperfection magnitude, during the numerical computation scaling factor equal to 200 was used on the web depth as it is proposed by Yie et al. [46] and Nie et al. [50]. Of course this imperfection magnitude was applied in the verification procedure as well (Table 2 in the Subsection 4.9). In case of bridge type girders partial factor $\gamma_{M1}=1,10$ and in case of building type girders partial factor $\gamma_{M1}=1,00$ is substituted in the denominator of the Eq. (30) (Section 3.2).

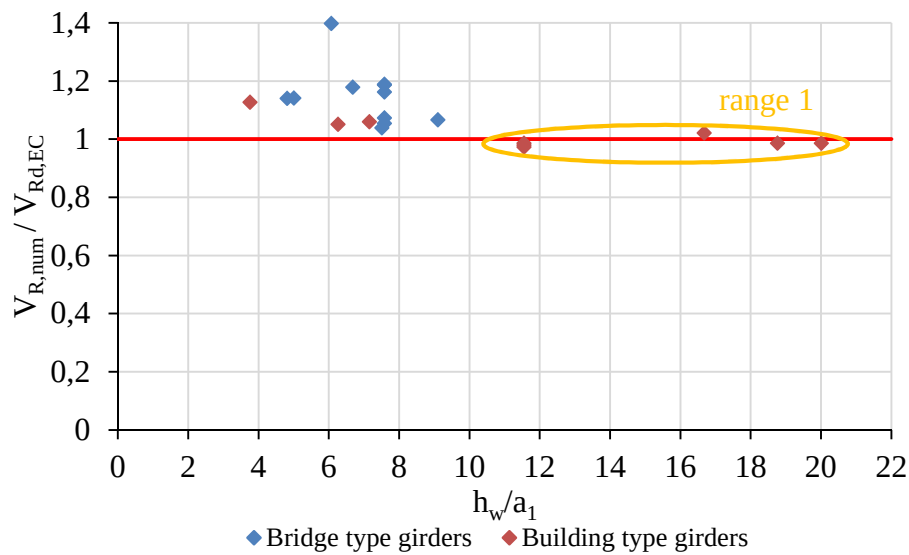


Fig. 31: Comparison of the numerical and analytical solutions.

Figure 31 shows the comparison of the results. The horizontal axis characterizes the web folds as subpanels of the corrugation profile with the ratio of the web depth divided by the parallel fold length (h_w/a_1) while the vertical axis represents the ratio of the numerically computed and analytically calculated resistances. One can observe that only some girder geometries are on the unsafe side with minimum ratio of 0,974. Figure 31 shows that in case of greater h_w/a_1 ratios (range 1) the results are very close to or under the red line (unsafe side) which is the case of building type girders where the fold lengths are short. This is the case of girder number 13, 14, 15, 16 and 19 in Table 6 (Section 5.1) where the $a_1=a_2$ parameters were relatively small. It may be attributed to the large imperfection magnitude, because in this diploma thesis it depends only on the web depth ($h_w/200$).

The statistical evaluation of the results can be seen in Table 8. The average ratio is 1,091 with standard deviation equal to 0,104 which means that the recommendation of the EN1993-1-5 [1] is adequate enough for the further use since during the verification procedure the numerically computed results show higher resistances than the experimental test results (Table 2 in Section 4.9). According to the Student's T-test distribution the reliability interval is $9,1 \pm 4,8\%$ on the safe side using 95% confidence interval with two-tails 2,5% upper and lower fractile of the analyzed sample.

Table 8: Statistical evaluation of the numerical results.

Average	1,091
Standard deviation	0,104
Coefficient of variation	0,095
Minimum	0,974
Maximum	1,399
Reliability interval of the mean considering normal distribution using 2,5% upper and lower fractile	0,045
Reliability interval of the mean considering Student's T-test distribution using 2,5% upper and lower fractile	0,048

5.4. Investigation of the patch loading resistance

The same twenty girder geometries which are included in Table 6 were analyzed under patch loading applying three different loading lengths for each. In case of patch loading with stiffened flange model (Fig. 16 and 17 in Section 4.5 and explanation in Section 4.9) three types of failure modes are existing, namely web crippling, local web buckling and global buckling of the whole web panel from which the last one can be avoided by the appropriate geometric layout of the corrugation profile (Eq. (5) in Section 2.2). Detailed description can be read in the Section 2.2 about development of the determination of the patch loading resistance of corrugated web girders.

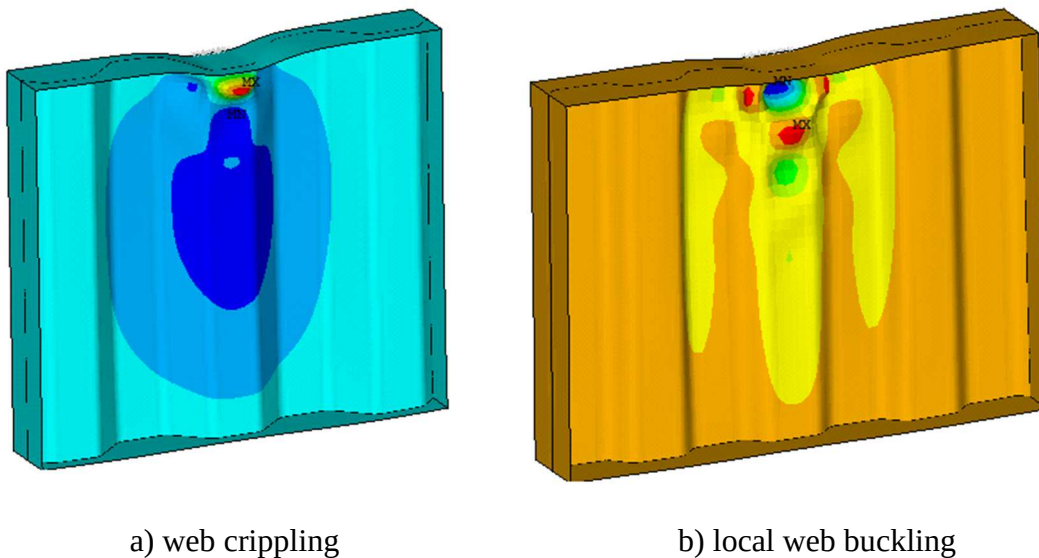


Fig. 32: Observed failure modes under patch loading.

The typical observed failure modes can be seen in Fig. 32. In Fig. 32/a typical web crippling failure can be observed while Fig. 32/b demonstrates the local web buckling failure of the corrugated web. The type of patch loading failure mostly depends on the corrugation profile.

After the sensitivity analysis and calibration of the imperfection magnitude, during the numerical computation scaling factor equal to 200 is used on the parallel fold length as it is proposed by Kövesdi et al. [21]. Of course this imperfection magnitude was applied in the verification procedure as well (Table 3 and 4 in Section 4.9). In case of the analyzed girder

geometries (bridge and building type as well) in Table 6 the proposed partial factor $\gamma_{M1}^{**}=1,35$ is substituted into the denominator of the Eq. (31) (Section 3.2) suggested by Kövesdi [60, 68].

Figure 33 shows the comparison of the results. The horizontal axis represent the influence of the rate of the loading length by the ratio of the loading length divided by the parallel fold length (s_s/a_1) while the vertical axis represents the ratio of the numerically computed and analytically calculated resistances. It can be observed that only three dots are close to or under (range 1) the red line (unsafe side) with minimum ratio of 0,989. This is the case of girder number 20 in Table 6 (Section 5.1) where the thickness of the web is extraordinarily thick ($t_w=1,5$ mm) thus the quick failure of this building type girder may be attributed to the large imperfection magnitude, because in this diploma thesis it depends only on the parallel fold length of the web panel ($a_1/200$).

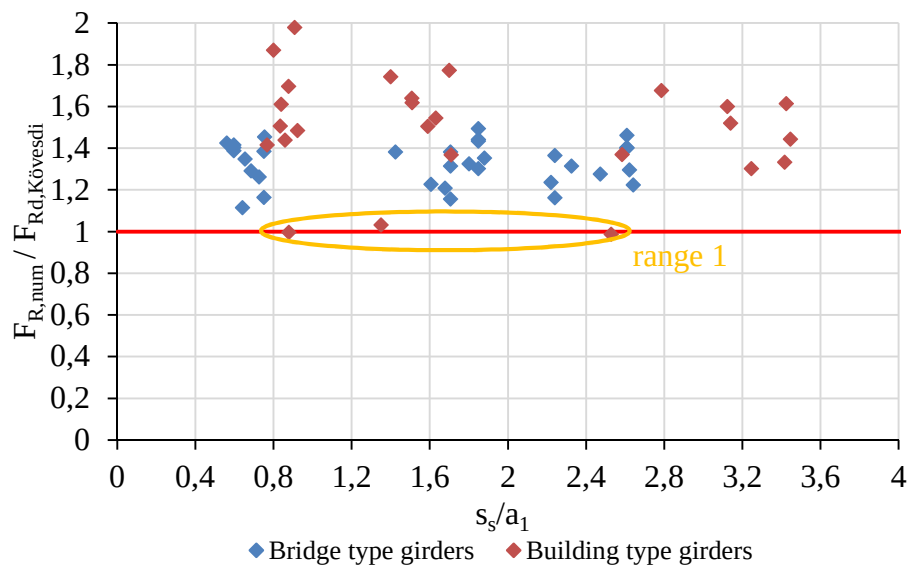


Fig. 33: Comparison of the numerical and analytical solutions.

Omission of girder number 20 the results show that in case of building type girders the received ratios are higher in average than in case of bridge type girders. It has to be mentioned that the design method of Kövesdi et al. [18] for the determination of the patch loading resistance (Eq. (31)) has been developed for bridge type girders. It can be observed that using small s_s/a_1 ratios the differences become larger between the computed and calculated resistances, but anyway most of the results are on the safe side.

The statistical evaluation of the results can be seen in Table 9. The average ratio is 1,405 with standard deviation equal to 0,196 which means that the proposal of Kövesdi et al. gives conservative results, but without any other alternatives for both bridge and building type girders it is appropriate for further use. According to the Student's T-test distribution the reliability interval is $40,5 \pm 5,1\%$ on the safe side using 95% confidence interval with two-tails 2,5% upper and lower fractile of the analyzed sample.

Table 9: Statistical evaluation of the numerical results.

Average	1,405
Standard deviation	0,196
Coefficient of variation	0,139
Minimum	0,989
Maximum	1,981
Reliability interval of the mean considering normal distribution using 2,5% upper and lower fractile	0,050
Reliability interval of the mean considering Student's T-test distribution using 2,5% upper and lower fractile	0,051

5.5. Evaluation

The proposals for the pure resistances, namely the bending moment (Eq. (29) in Section 3.2), shear buckling (Eq. (30)) and patch loading resistances (Eq. (31)) were investigated in this Section and the competencies and reliability intervals were determined with the help of statistical evaluations of the sample included in Table 6.

In case of pure bending moment the proposal of the EN1993-1-5 [1] shows a good agreement with the numerically computed resistances by giving a lower limit estimate. Only one geometry of the sample is lying under its calculated value with the difference of 1,4%, thus the proposal is judged to be adequate enough for further use.

The proposal of the EN1993-1-5 [1] for the determination of the shear buckling resistance of corrugated web girders was investigated and the results show a good agreement with the numerically computed resistances. The proposal gives a good estimate in case of h_w/a_1 less than 10. If this ratio is greater than 10 the results are very close to or a bit under the proposal of the EN1993-1-5 which is attributed to the applied imperfection depending only on the web depth (h_w). The maximum difference on the unsafe side is 2,6% which is de facto attributed to the large imperfection magnitude. Therefore the proposal is found to be adequate enough for further use.

In case of patch loading the proposal of Kövesdi et al. [18] gives a conservative estimation for the determination of the patch loading resistance. Only one geometry is close to or under the used proposal with maximum difference of 1,1% on the unsafe side. Without any other alternatives for both bridge and building type girders it is appropriate for further use.

6. Numerical investigation of the distinct interaction planes

In the current section the distinct interaction planes of the 3D interaction surface (M-V-F), namely the M-V, M-F and V-F interaction planes of corrugated web girders are investigated. Just a few number of papers have been dealing with the investigation of the distinct interaction planes which can be read in Sections 2.3 and 3.2. The main questions of this section which are being focused on is to determine the competency of these proposals to make sure that the further use is reasonable or to give new proposals to estimate the observed interaction behaviours by applying better lower limit approximations.

6.1. Numerical research program, strategy and goals

In this section the results of the numerical research program executed on the girder geometries found in Table 6 (Section 5.1) are introduced in detail. The pure resistances, in other words the intersection points of the distinct interaction curves with the corresponding axes are investigated in Section 5. From then on the intermediate points of the girder geometries under combined M-V, M-F and V-F interactions are determined numerically and compared with the proposals developed by different authors (Section 3.2). During the analysis the applied corrugated web layouts can be seen in Fig. 26.

As it was introduced in the previous section all of the girders were loaded under three different patch loading lengths which were between $\sim 0,5$ to $\sim 3,5$ of the parallel fold length (a_1) depending on the meshing. In case of the investigation of the M-F and V-F interaction 20 intermediate points were computed for each, because of the three different applied patch loading lengths and 8 intermediate points were computed for the characterization of the M-V interaction behaviour. Thus every girder in Table 6 was analyzed in total of 48 times, 20 times under combined M-F, 20 times under combined V-F and 8 times under combined M-V loading. Therefore in the current section a total number of 960 numerical computations had been carried out.

With the help of the results statistical evaluations are taken place in every cases to determine the correctness of each proposals and if it is necessary new proposals are given.

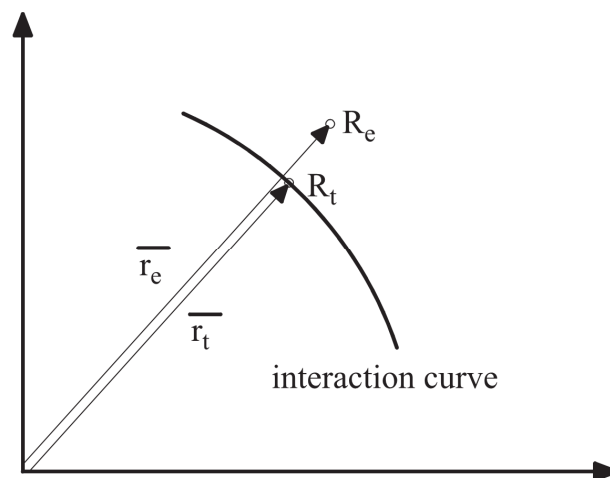


Fig. 34: Basis of the statistical evaluation [61].

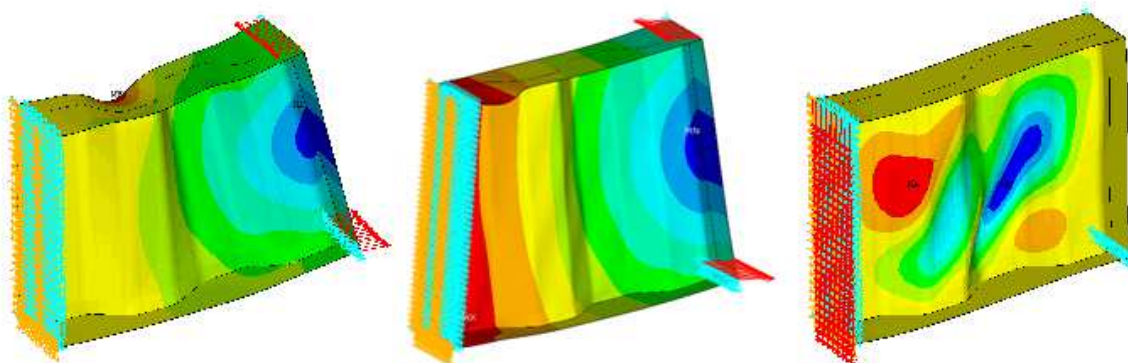
The statistical evaluation is based on the distance between the numerically computed results and the proposed interaction curve shown by Fig. 34. In the figure R_e and R_t represent the numerically computed results and its projection to the interaction curve, respectively, since the r_e and r_t represent the vectors pointing to the R_e and R_t points. The ratio of the lengths of the vectors characterizes the conformity of the proposed interaction curve.

In most of the cases the exponents of the “moving” interaction curves differ which means that no analytical solution is existing for the problem. The only way is to solve the problem numerically by using a nonlinear equation solver.

6.2. Investigation of the M-V interaction plane

Parametric study can be carried out due to the the verified numerical model to investigate the combined bending moment and shear force interaction behaviour of corrugated web girders as a distinct interaction plane of the M-V-F interaction surface. It has to be emphasized that the presented investigation of this section was performed by Kövesdi, Jäger, Dunai [58] in 2013 and published in 2014.

The ultimate load carrying capacities of the girders, namely the bending moment and shear buckling resistances have been determined numerically for each girder geometries found in Table 6 in Section 5.1. The aforementioned capacities are the intersection points of the interaction curves with the axes. In this part of the recent research program 8 intermediate points between the ultimate capacities are determined numerically applying different bending moment and shear force ratios on the structure. Thus a total number of 160 numerical computations have been performed on the 20 girder geometries. The typical observed failure modes are shown in Fig. 35. Figure 35/a shows a typical bending failure where local flange buckling governs the collapse mode of the structure, Fig. 35/c represents the shear buckling failure of the web. The combination of the aforementioned two failure modes can be seen in Fig. 35/b.



a) pure bending failure b) failure under combined loading c) shear buckling failure

Fig. 35: Observed typical failure modes.

One can observe in Fig. 35/b that the failure occurs in the compression flange at the middle of the parallel fold of the web, where the flange outstand is the greatest and the bending moment is also relatively large. This location is at the first parallel fold viewed from the left side. In this cross-section the bending resistance of the analyzed girders can be determined from the applied

bending moment and the applied accompanying shear force. From the analyzed 20 girder geometries two are chosen for illustrating their results. In Fig. 36 the results of a typical bridge girder geometry and a typical building girder geometry can be seen. In the figure the horizontal axis represents the bending resistances and the vertical axis represents the shear resistances of both analyzed girders and the points show the numerically computed results. The proposed interaction equation of the EN1993-1-5 [1] is also presented in both diagrams by brown dashed line.

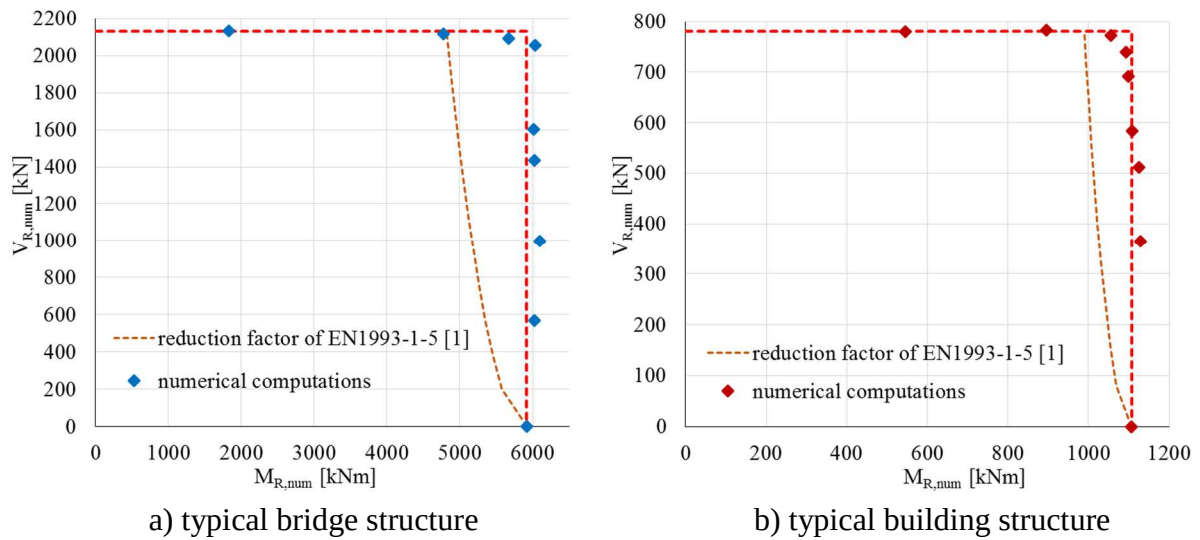


Fig. 36: Typical M-V interaction behaviour.

The results of the combined bending and shear interaction analysis show that just a small interaction behaviour can be observed. In both type of girder geometries in Fig. 36/a and Fig. 36/b the numerically computed results follow the red dashed lines representing the pure bending and shear buckling resistances. It can be observed in both cases that the bending resistance reduction formula of the EN1993-1-5 [1] gives unnecessarily conservative results.

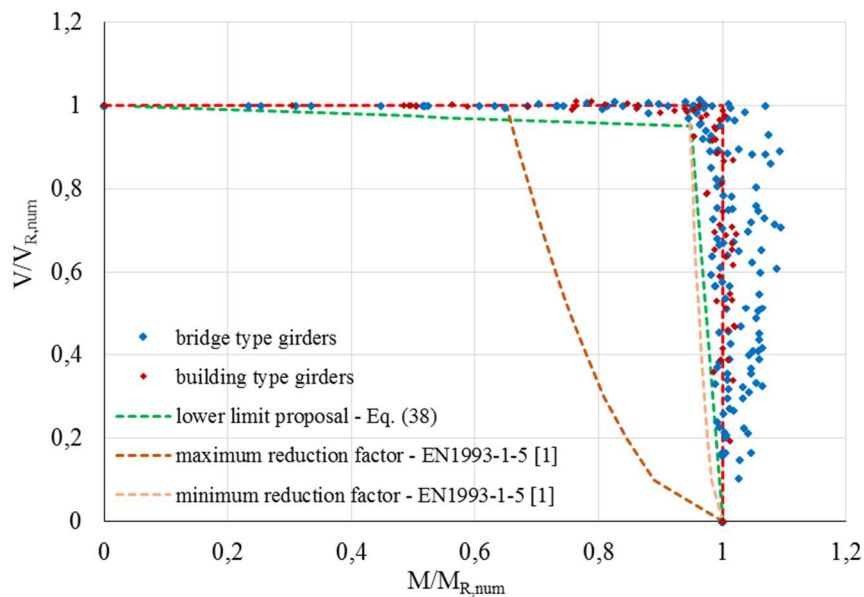


Fig. 37: Observed M-V interaction behaviour with respect to the computed resistances.

All the numerical results of the analyzed 20 girder geometries are presented in Fig. 37 in which the horizontal axis represents the ratio of the numerically computed bending resistances while the vertical axis represents the numerically computed shear buckling resistances. In the figure the blue dots demonstrate the results regarding to the bridge type girders and the red dots demonstrate the results regarding to the building type girders. The numerical results show that the maximum bending moment resistance reduction and the maximum shear buckling resistance reduction due to the combined loading situation are 4,62% and 2,95%, respectively, thus the multiplication factors equal to 0,95 and 0,95 (5%-os interakció) on the analytically calculated bending moment and shear buckling resistances can be applicable if the M-V combined loading situation is subjected to the structure. One can observe again that recommendation of the EN1993-1-5 [1] gives too conservative estimate (light and dark grey dashed lines). To approximate the results the following lower limit proposal is offered which is shown by the green dashed line:

$$\max \begin{cases} \frac{1}{20} \frac{M}{M_R} + \frac{V}{V_R} \leq 1,0 \\ \frac{M}{M_R} + \frac{1}{20} \frac{V}{V_R} \leq 1,0 \end{cases} \quad (38)$$

where M_R numerically computed bending resistance,
 V_R numerically computed shear buckling resistance.

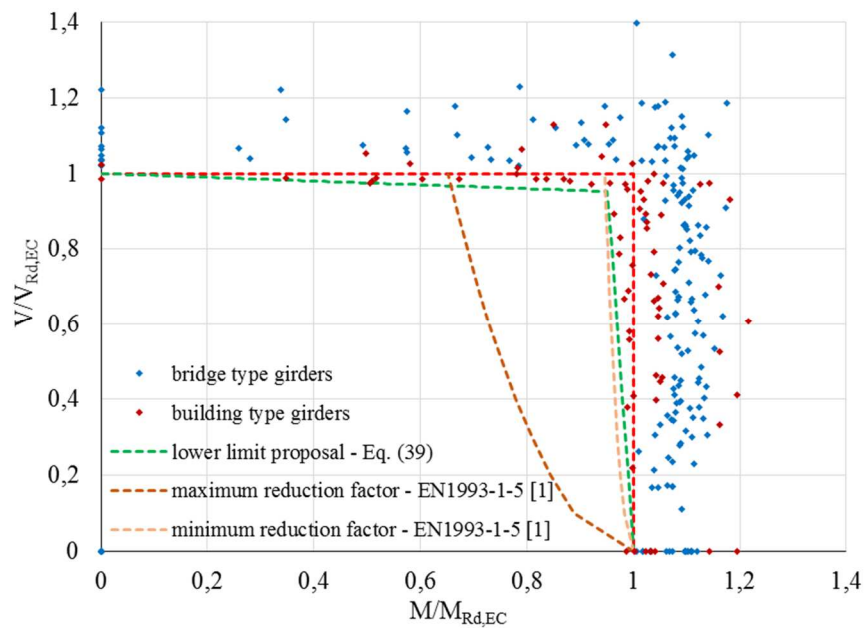


Fig. 38: Observed M-V interaction behaviour with respect to the calculated resistances.

In Fig. 38 the results are plotted with the calculated pure resistances according to the EN1993-1-5 [1] which are presented by the Eq. (29) and Eq. (30) in the Subsection 3.2. One can observe that the results drawn out a bit greater interaction behaviour than in Fig. 37. This is attributed to the observation regarding to Fig. 31 (Subsection 5.3) that in case of small a_1 the computed shear resistances are close to or a bit under the analytically calculated resistances due to the large applied imperfections ($hw/200$) which depends in this diploma thesis only on the web

depth. But anyway the aim of this research program is to provide safe side proposals for the interaction behaviours and not to give new proposals for the determination of the pure resistances. Therefore all of the results are taken into account during the creation of the new proposals.

According to the results plotted in Fig. 38 the maximum reduction is around 5%, therefore the current lower limit proposal for the M-V interaction diagram (green dashed line) is shown by the Eq. (39) below for design purposes. The results also demonstrate that the recommended reduction factor of the EN1993-1-5 [1] leads to an unnecessarily conservative design.

$$\max \left\{ \begin{array}{l} \frac{1}{20} \frac{M}{M_{Rd,EC}} + \frac{V}{V_{Rd,EC}} \leq 1,0 \\ \frac{M}{M_{Rd,EC}} + \frac{1}{20} \frac{V}{V_{Rd,EC}} \leq 1,0 \end{array} \right. \quad (39)$$

where $M_{Rd,EC}$ calculated bending resistance according to the EN1993-1-5 [1],
 $V_{Rd,EC}$ calculated shear buckling resistance according to the EN1993-1-5 [1].

The statistical evaluation regarding to the Eq. (38) and Eq. (39) can be seen in Table 10. By using the numerically computed pure resistances (column 2 in Table 10) the average is 1,034 with standard deviation equal to 0,029. If the standard formulas for the pure resistances are applied the average becomes 1,112 with standard deviation equal to 0,065. It is well-marked that the minimum value (0,974) is equal to the minimum value found in Table 8 in the Subsection 5.3. Thus the minimum value belongs to the pure resistance and not to the intermediate points, because all of the intermediate points are out of the current proposal. Therefore the current proposal can be applicable for design purposes.

Table 10: Statistical evaluation of the numerical results.

	lower limit proposal using R,num (Eq.(38))	lower limit proposal using Rd,EC (Eq.(39))
Average	1,041	1,112
Standard deviation	0,027	0,065
Coefficient of variation	0,026	0,058
Minimum	1,000	0,974
Maximum	1,137	1,449

These research proves the results of Kuchta [57] who suggested also just a small resistance reduction due to the combined M-V loading situation and proves the statement of Elgally et al. [4] that the interaction behaviour of trapezoidally corrugated web girders could be neglected.

The observed resistance reduction is not larger than the web contribution in the bending resistance or the flange contribution in the shear buckling resistance. But these contributions are neglected in the design procedures of the corrugated web girders.

Based on the numerical calculations another important observation is also noticed that the interaction behaviour does not depend on the corrugation profile, but the additional normal

stress distribution in the flanges are largely dependent on the corrugation profile of corrugated web girders [58]. According to the results of the numerical calculations there is no relationship existing between the transverse bending moment in the flanges indicated by the shear force and the in-plane-bending resistance reduction, if plastic design is used.

6.3. Investigation of the M-F interaction plane

Parametric study can be carried out due to the verified numerical model to investigate the combined bending moment and transverse force interaction behaviour of corrugated web girders as a distinct interaction plane of the M-V-F interaction surface.

The ultimate load carrying capacities of the girders, namely the bending moment and patch loading resistances have been determined numerically in Section 5 for each girder geometries found in Table 6. The aforementioned capacities are the intersection points of the interaction curves with the axes. During the investigation three different patch loading lengths have been applied on the structure for better understanding of the interaction behaviour.

In this part of the recent research program 20 intermediate points between the ultimate capacities are determined numerically applying different bending moment and patch loading ratios on the structure. From the 20 computations regarding to one girder geometry, 10 points belong to the shortest applied patch loading lengths, and 5-5 belong to the longer ones. Thus total number of 400 numerical computations have been performed on the 20 girder geometries in this subsection. The typical observed failure modes are shown in Fig. 39.

Figure 39/a shows a typical bending failure where local flange buckling governs the collapse mode of the structure, Fig. 39/c represents the patch loading failure of the web panel. The combination of the aforementioned two failure modes can be seen in Fig. 39/b.

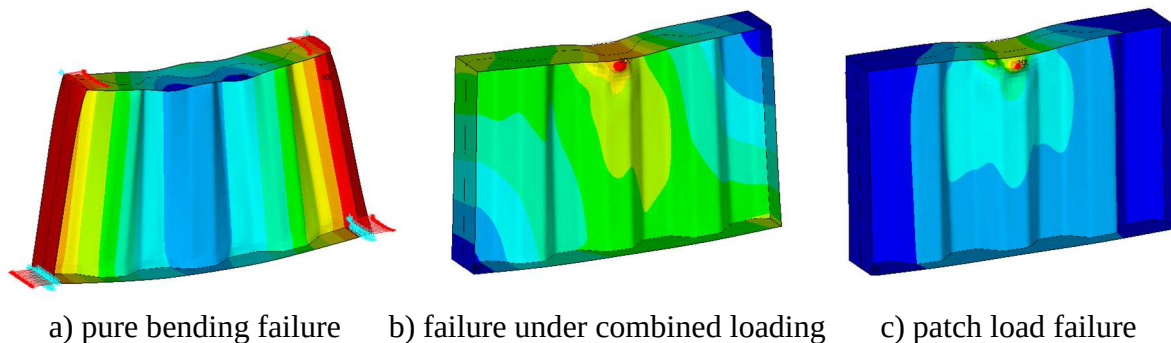


Fig. 39: Observed typical failure modes.

From the results of the 20 girder geometries the results of one typical bridge and one typical building structure are chosen for illustrating the structural behaviour under the combined loading situation. Figure 40 shows the observed interaction behaviours regarding to the three different patch loading lengths in comparison with the proposals of Elgaaly and Seshadri [17] and Kövesdi et al. [59]. For the sake of completeness it has to be emphasized again that Elgaaly and Seshadri proposed interaction curve for unstiffened flange model since Kövesdi et al. [59] proposed interaction curve for stiffened flange model. It was explained and illustrated previously in the Subsections 3.2, 4.5 and 4.9.

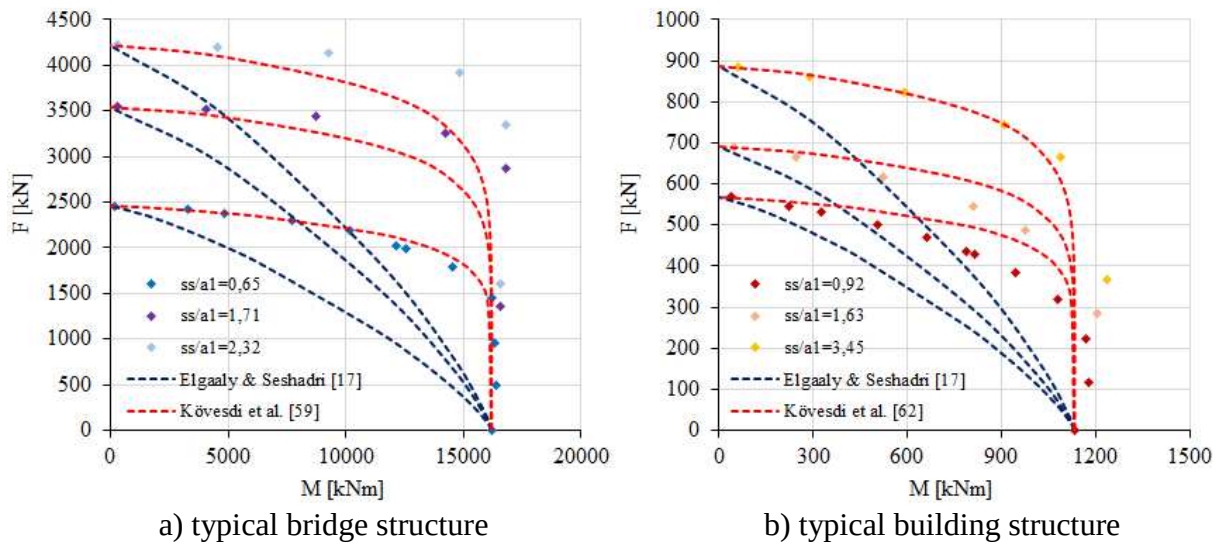


Fig. 40: Observed M-F interaction behaviours in case of different patch loading lengths.

In the current diploma thesis the proposal of Kövesdi et al. [59] is relevant. In Fig. 40 the horizontal axes represent the numerically computed bending capacities and the vertical axes represent the numerically computed patch loading capacities in order to illustrate the results of the combined loading situation. Both Fig. 40/a (typical bridge girder) and 40/b (typical building girder) show the results using different patch loading lengths. One can observe in Fig. 40/a that in case of a relatively short patch loading length ($ss/a_1=0,65$ plotted by blue dots) the interaction behaviour of the girder is very close to or a slightly under the lower limit proposal of Kövesdi et al. [59] illustrating by the red dashed line. By using longer patch loading lengths ($ss/a_1=1,71$ and $2,32$ plotted by purple and light blue dots) the observed interaction behaviours are above the proposal of Kövesdi et al. However in case of the building type girder (Fig. 40/b) the results are mostly under the proposal of Kövesdi et al. except that situation when very long patch loading lengths ($ss/a_1=3,45$ plotted by orange dots) were applied.

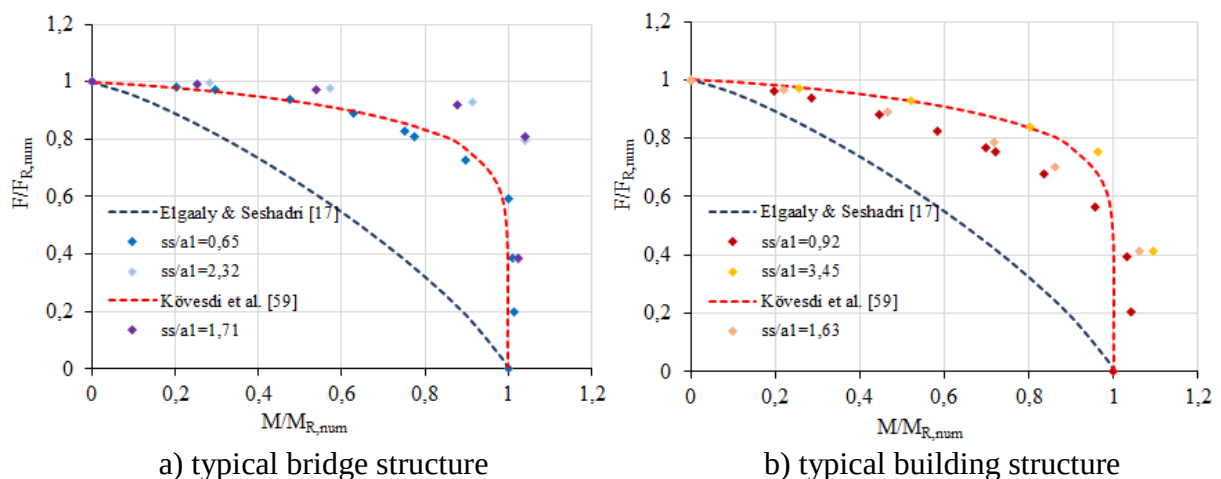


Fig. 41: Observed M-F interaction behaviours in case of different patch loading lengths.

The results of the same girders can be seen in Fig. 41 where the computed values are normalized with the numerically computed pure bending and patch loading resistances. A reasonable behaviour is drawn out in both cases, that by using longer patch loading lengths the interaction behaviour is reducing significantly. Another observation is drawn out during the investigation

that in case of bridge type girders the interaction behaviour is smaller than in case of building type girders which can be attributed to the fold length of the web panel.

The results of the total number of 400 numerical computations are plotted in Fig. 42. In the figure the horizontal and vertical axes represent the normalized results with respect to the numerically computed pure resistances, the blue and red dots represent the results regarding to bridge and building type girders, respectively. One can see that many points are under the proposal of Kövesdi et al. [59] represented by the red dashed line, therefore new proposals are given for bridge and building type girders separately. In Fig. 42 the red continuous line gives a lower limit proposal for building type girders (Eq. (40)) while the blue continuous line represents a lower limit proposal for bridge type girders (Eq. (41)).

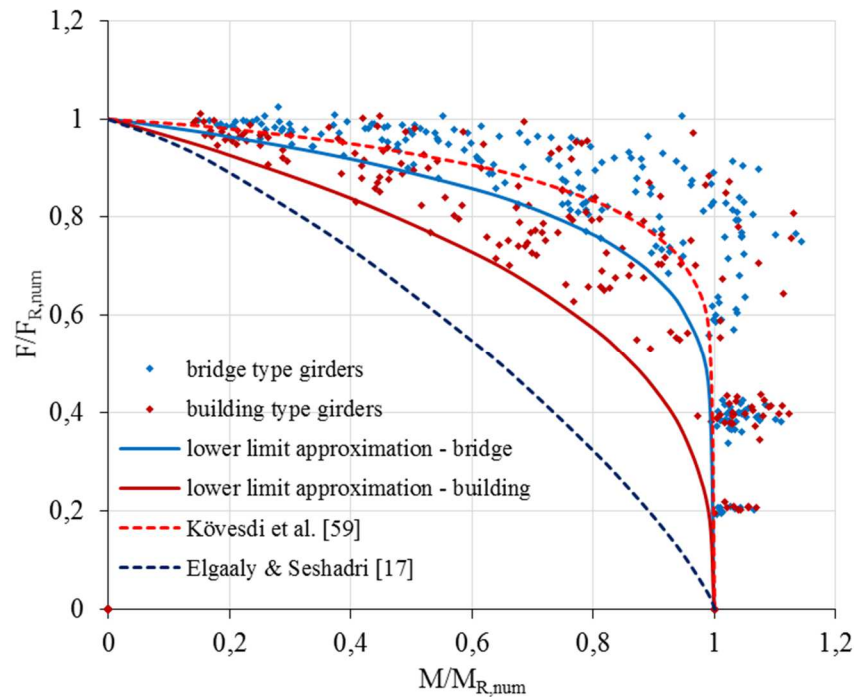


Fig. 42: Observed M-F interaction behaviour with respect to the computed resistances.

$$\left(\frac{M}{M_R}\right) + \left(\frac{F}{F_R}\right)^{2,9} \leq 1,0 \quad (40)$$

$$\left(\frac{M}{M_R}\right) + \left(\frac{F}{F_R}\right)^6 \leq 1,0 \quad (41)$$

where M_R numerically computed bending resistance,
 F_R numerically computed patch loading resistance.

The statistical evaluation regarding to the proposal of Kövesdi et al. [59] (Eq. (35) in the Subsection 3.2) and the current proposals (Eq. (40) and Eq. (41)) can be seen in Table 11. By using separately the Eq. (41) on bridge type girders (column 3) and the Eq. (40) on building type girders (column 4) both result a lower limit approximations for both samples. The received averages are 1,074 and 1,121 with standard deviations equal to 0,057 and 0,091, respectively.

If the Eq. (40) is used for the whole sample the average arises to 1,154 with standard deviation equal to 0,101. Figure 42 and the Table 11 show that the Eq. (40) is applicable as a lower limit proposal, and for bridge type girders another lower limit proposal presented by the Eq. (41) may be applicable by considering a smaller interaction behaviour.

Table 11: Statistical evaluation of the numerical results.

	Interaction curve of Kövesdi et al. [59]	Eq. (41) on bridge type girders	Eq. (40) on building type girders	Eq. (40)
Average	1,018	1,074	1,121	1,154
Standard deviation	0,064	0,057	0,091	0,101
Coeff. of variation	0,063	0,053	0,081	0,087
Minimum	0,837	0,997	0,998	0,998
Maximum	1,198	1,265	1,431	1,448

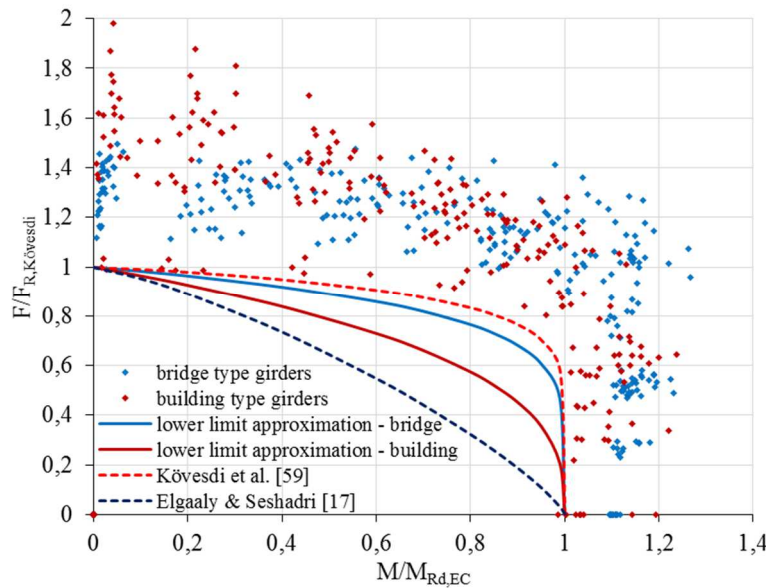


Fig. 43: Observed M-F interaction behaviour with respect to the calculated resistances.

In Fig. 43 the normalized results with respect to the analytically calculated pure resistances according to the the proposal of the EN1993-1-5 [1] (Eq. (29)) and Kövesdi et al. [18] (Eq. (31)) can be seen. In the figure the red and blue dashed lines represents the proposal of Kövesdi et al. [59] and Elgaaly and Seshadri [17] as before.

$$\left(\frac{M}{M_{Rd,EC}} \right) + \left(\frac{F}{F_{R,Kövesdi}} \right)^{2,9} \leq 1,0 \quad (42)$$

where $M_{Rd,EC}$ calculated bending resistance according to the EN1993-1-5 [1],
 $F_{R,Kövesdi}$ calculated patch loading resistance according to Kövesdi et al. [59].

The statistical evaluation of the proposal of Kövesdi et al. [59] and the current proposal (Eq. (42)) can be found in Table 12. The averages are 1,299 and 1,440 and the standard deviations are 0,169 and 0,195. The minimum values (0,986) are equal to the minimum value found in Table 7 in the Subsection 5.2. Thus the minimum value belongs to the pure resistance and not

to the interaction behaviour, therefore the current proposal can be applicable for design purposes.

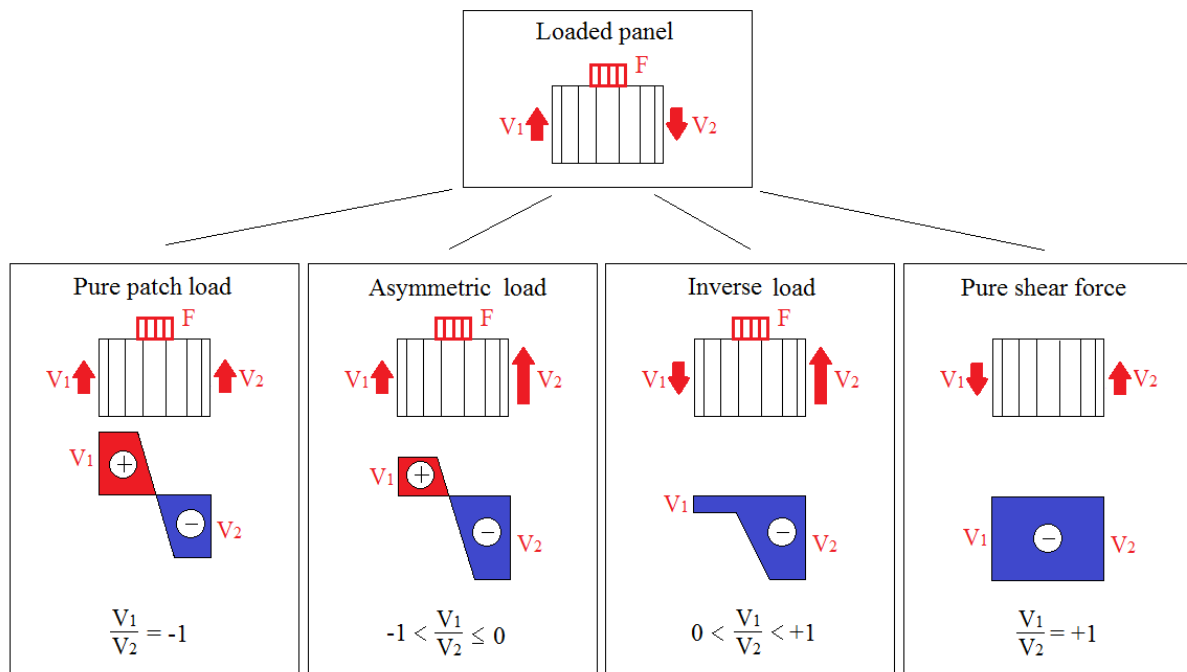
Table 12: Statistical evaluation of the numerical results.

	Interaction curve of Kövesdi et al. [59]	current proposal (Eq. (42))
Average	1,299	1,440
Standard deviation	0,169	0,195
Coefficient of variation	0,130	0,135
Minimum	0,986	0,986
Maximum	1,983	1,995

6.4. Investigation of the V-F interaction plane

Parametric study can be carried out due to the verified numerical model to investigate the combined shear force and transverse force interaction behaviour of corrugated web girders as a distinct interaction plane of the M-V-F interaction surface.

The ultimate load carrying capacities of the girders, namely the shear buckling and patch loading resistances have been determined numerically in Section 5 for each girder geometries found in Table 6. The aforementioned capacities are the intersection points of the interaction curves with the axes. Three different patch loading lengths have been applied on the structure for better understanding of the interaction behaviour.



a) pure patch load b) asymmetric loading c) patch and shear force d) pure shear force

Fig. 44: Phases of the combined shear and patch loading interaction.

In this part of the recent research program 20 intermediate points between the ultimate capacities are determined numerically applying different shear and transverse force ratios on

the structure. From the 20 computations regarding to one girder geometry, 10 points belong to the shortest applied patch loading lengths, and 5-5 belong to the longer ones. Thus a total number of 400 numerical computations have been performed on the 20 girder geometries in this subsection.

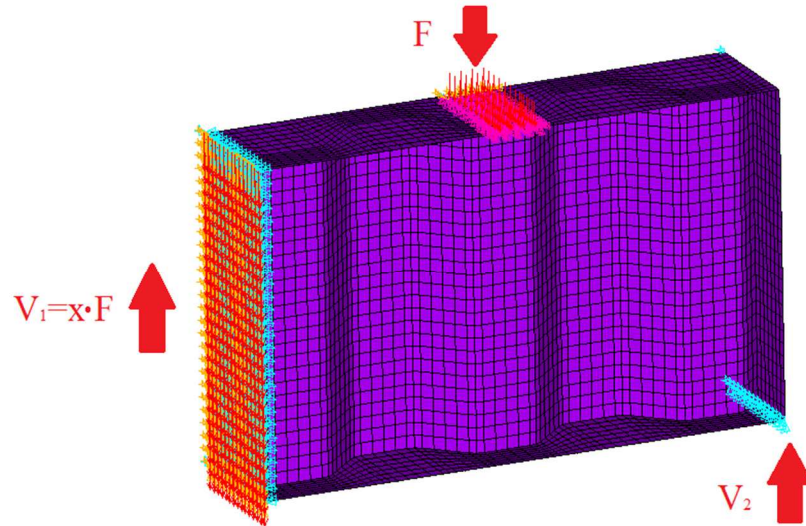


Fig. 45: Numerical model of the loaded panel.

The current investigation is carried out according to Figure 44. The determination of the load carrying capacities under pure patch loading (Fig. 44/a) and under pure shear force (Fig. 44/d) have been already executed in Section 5 while this subsection is dealing with the asymmetric (Fig. 44/b) and inverse loading (Fig. 44/c).

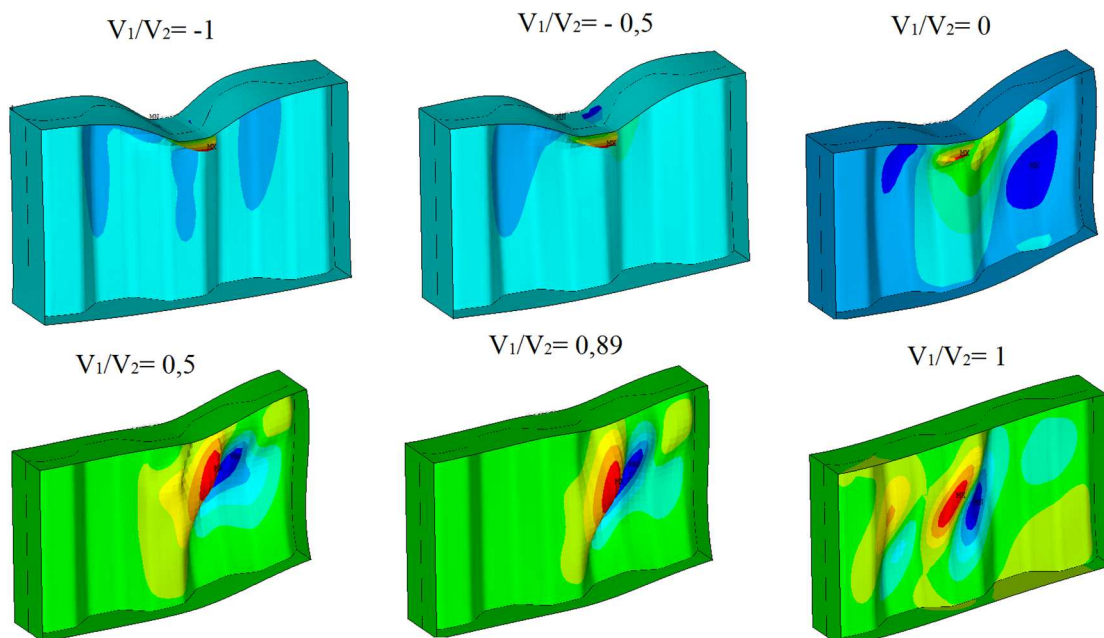


Fig. 46: Typical observed failure modes in case of different V_1/V_2 ratios.

To deal with such a loading situation the model presented by Fig. 45 is needed where “x” varies theoretically from 0,5 to $-\infty$. Figure 46 shows the typical collapse modes of the structure under different V_1/V_2 ratios. One can see that if V_1/V_2 is equal to -1 then pure patch loading failure

governs the collapse mode of the structure (Fig. 44/a), on the other hand if V_1/V_2 is equal to +1 then shear buckling failure occurs as Fig. 44/d also shows. The additionally presented intermediate collapse modes in Fig. 46 belong to the V_1/V_2 ratios equal to -0,5, 0, 0,5 and 0,89. In this cases the location of the failure is shifted into the direction of the influential shear force which is the right side of the structure.

From the results of the 20 girder geometries the results of one typical bridge and one typical building structure are chosen for illustrating the structural behaviour under the combined loading situation. Figure 47 shows the observed interaction behaviours regarding to the three different patch loading lengths in comparison with the proposals of Elgaaly and Seshadri [17] and Kövesdi et al. [62] (Eq. (36) and Eq. (37) in the Subsection 5.3).

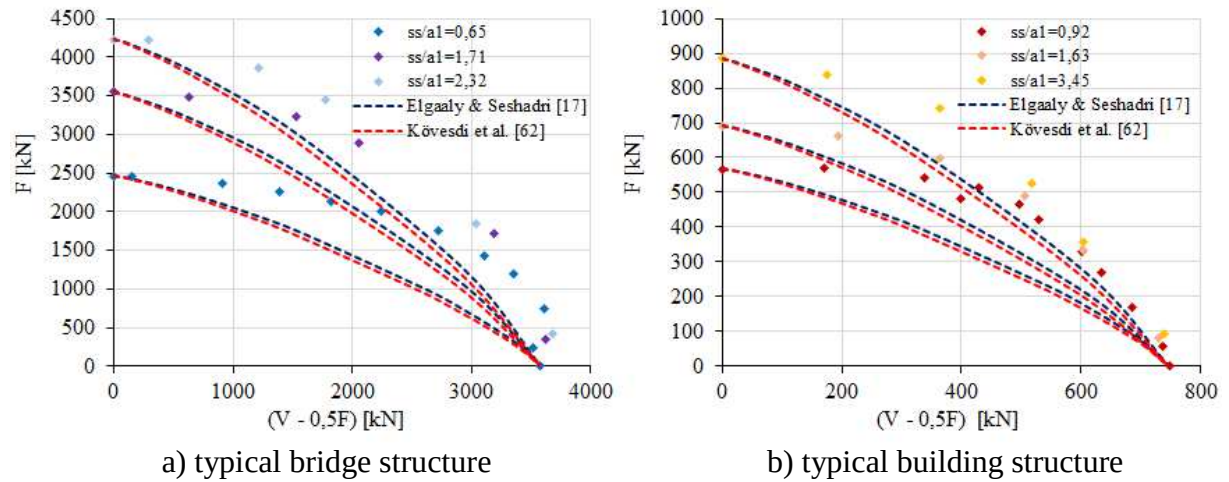


Fig. 47: Observed V-F interaction behaviours in case of different patch loading lengths.

In Fig. 47 the horizontal axes represent the numerically computed shear capacities and the vertical axes represent the numerically computed patch loading capacities in order to illustrate the results of the combined loading situation. Both Fig. 47/a and 47/b show the results using different patch loading lengths. One can observe in Fig. 47/a and 47/b that the results are above the proposals illustrating by the blue and red dashed lines.

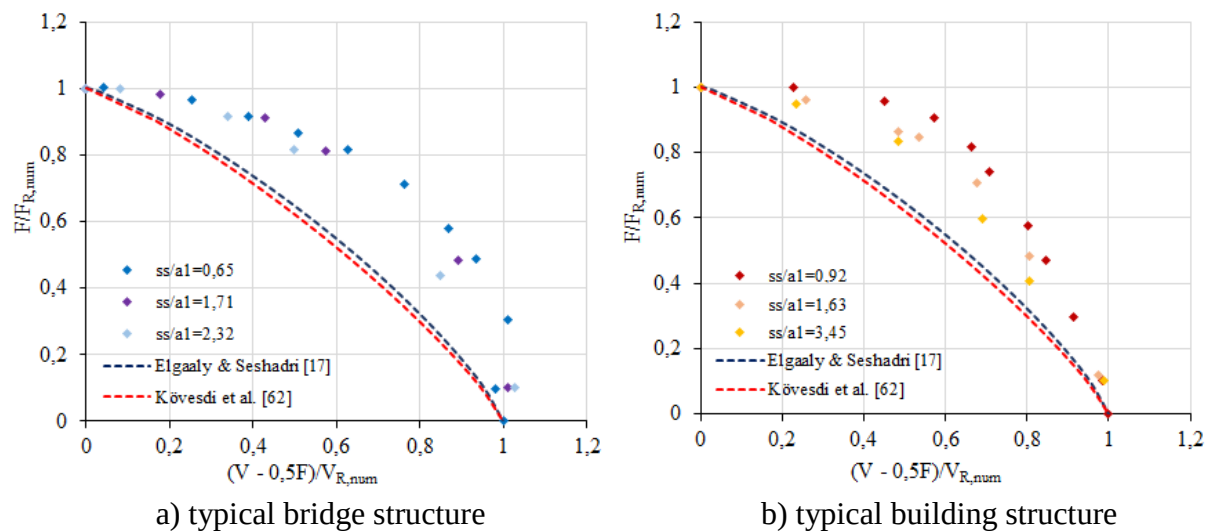


Fig. 48: Observed V-F interaction behaviours in case of different patch loading lengths.

The results of the same girders can be seen in Fig. 48 where the computed values are normalized with the numerically computed pure shear buckling and patch loading resistances. A reasonable behaviour is drawn out in both cases, that by using longer patch loading lengths the interaction behaviour is increasing.

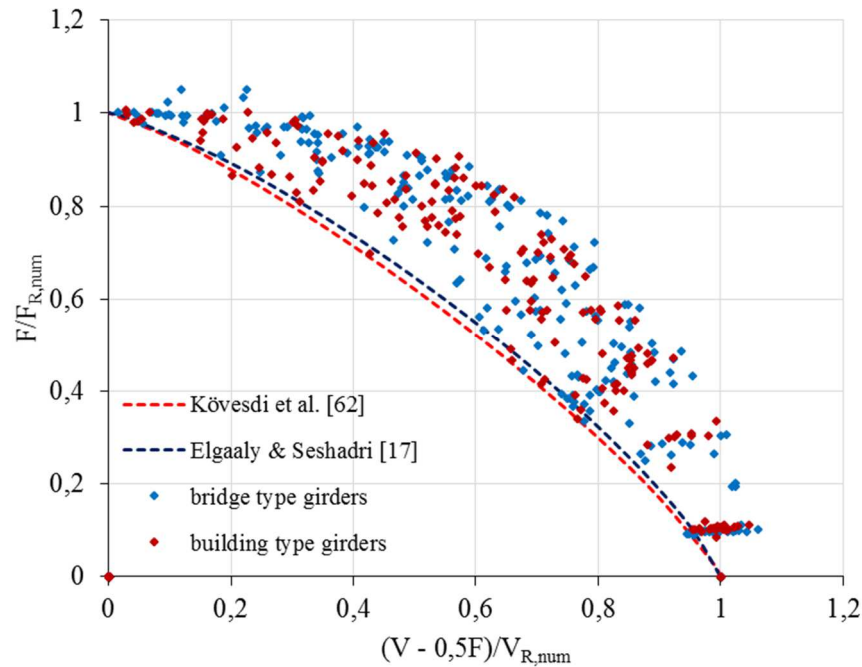


Fig. 49: Observed V-F interaction behaviour with respect to the computed resistances.

The results of the total number of 400 numerical computations are plotted in Fig. 49. In the figure the horizontal and vertical axes represent the normalized results with respect to the numerically computed pure resistances, the blue and red dots represent the results regarding to bridge and building type girders, respectively. One can observe that both proposals (Eq. (36) and (37)) give good lower estimate for the V-F interaction behaviour of corrugated web girders. Furthermore it is drawn out that there is no notable difference between bridge and building type girders in the V-F interaction behaviour. Moreover the observation of Kövesdi et al. [61] is confirmed by the results which says that smaller the h_w/t_w ratio, the greater is the interaction behaviour.

Table 13: Statistical evaluation of the numerical results.

	Interaction curve of Elgaaly & Seshadri [17]	Interaction curve of Kövesdi et al. [62]
Average	1,125	1,145
Standard deviation	0,085	0,091
Coefficient of variation	0,076	0,079
Minimum	0,977	0,991
Maximum	1,321	1,352

The statistical evaluation regarding to the proposal of Elgaaly and Seshadri [17] and Kövesdi et al. [62] (Eq. (36) and (37) in the Subsection 3.2) can be seen in Table 13. The received averages are 1,125 and 1,145 with standard deviations equal to 0,085 and 0,091, respectively.

The minimum values are 0,977 and 0,991 while the maximum values are 1,321 and 1,352, respectively.

In Fig. 50 the normalized results with respect to the analytically calculated pure resistances according to the proposal of the EN1993-1-5 [1] and Kövesdi et al. [18] (Eq. (30) and (31) in the Subsection 3.2) can be seen. In the figure the red and blue dashed lines represents the proposal of Kövesdi et al. [62] and Elgaaly and Seshadri [17] as previously. All of the intermediate points are above the proposals, however 5 points are under the curves lying on the horizontal axis. These points have been already discussed in the Subsection 5.3 where the pure shear buckling resistances were investigated.

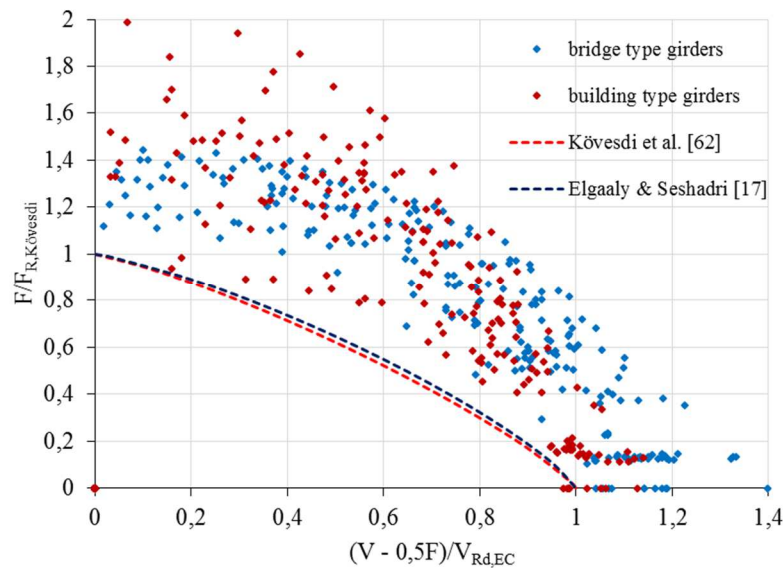


Fig. 50: Observed V-F interaction behaviour with respect to the calculated resistances.

The statistical evaluation of the proposals of Elgaaly and Seshadri [17] and Kövesdi et al. [62] can be found in Table 14. The averages are 1,387 and 1,407 and the standard deviations are 0,212 and 0,218. The minimum values (0,974) are equal to the minimum value found in Table 8 in the Subsection 5.3. Thus the minimum values belong to the pure resistance and not to the intermediate points, therefore the current proposal can be applicable for design purposes.

Table 14: Statistical evaluation of the numerical results.

	Interaction curve of Elgaaly & Seshadri [17]	Interaction curve of Kövesdi et al. [62]
Average	1,387	1,407
Standard deviation	0,212	0,218
Coefficient of variation	0,153	0,155
Minimum	0,974	0,974
Maximum	2,090	2,114

In spite of the fact that the proposal of Elgaaly and Seshadri is also found to be adequate, some intermediate points in Fig. 49 are under their interaction proposals, therefore in the followings the proposal of Kövesdi et al. is used.

6.5. Evaluation

In this section the investigation of the distinct interaction planes of the M-V-F interaction surface was performed. During the investigation a total number of 960 computations have been executed to understand the M-V, M-F and V-F interaction behaviours separately.

The EN1993-1-5 [1] recommends formula for the consideration of the M-V interaction of corrugated web girders. The results did not show any relationship with formula of the Eurocode. A very small, only 5% interaction behaviour was observed under the combined bending moment and shear force interaction. Based on this result new proposal was given for the consideration of the interaction behaviour. This proposal is represented by the Eq. (38) and the Eq. (39).

There is only one existing proposal for stiffened flange model how to take the combined bending moment and patch loading interaction into account. It is proposed by Kövesdi et al. [59]. The results showed that the proposed “moving” interaction formula is not appropriate for the analyzed parameter range, therefore a new proposal was provided as a lower limit proposal in Eq. (40) and the Eq. (42).

Two proposals have been provided for taking the V-F interaction behaviour into account. These proposals belong to Elgaaly and Seshadri [17] and Kövesdi et al. [62]. The results showed that the proposal of Kövesdi et al. (Eq. (37)) may be applicable as a lower limit estimate for the consideration of the V-F interaction behaviour of corrugated web girders.

7. Numerical investigation of the bending-shear-patch loading interaction

In the current section the 3D interaction surface (M-V-F) of corrugated web girders are investigated. There are no previous investigations found in the international literature dealing with this subject. The main aim of this section is to give and verify a lower limit proposal for the 3D interaction behaviour of the corrugated web girders.

7.1. Numerical research program, strategy and goals

In this section the results of the numerical research program are introduced in detail. The pure resistances, in other words the intersection points of the distinct interaction curves with the corresponding axes are investigated in Section 5 and the distinct interaction planes of the 3D interaction surface are investigated in Section 6.

As it was introduced in the previous sections all of the girders are loaded under three different patch loading lengths which are defined between $\sim 0,5$ to $\sim 3,5$ times the parallel fold length (a_1). In the current section a total number of 640 numerical simulations have been performed on 20 different girder geometries defined in Table 6. On each girder 32 simulations are carried out in average. Based on the results of Section 6 more computations belong to those girders where the results are more unfavourable from point of view of the M-V, M-F and/or V-F interaction behaviours.

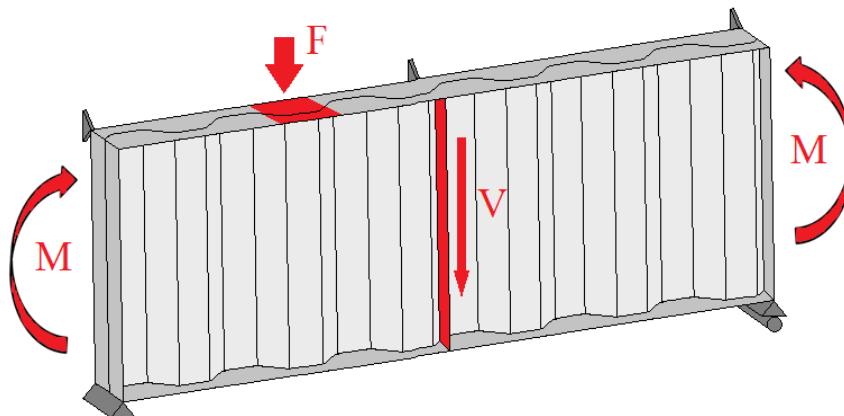


Fig. 51: Schematic view of the boundary and loading conditions.

The schematic view of the loaded girder can be seen in Fig. 51. The girder is simply supported, stiffeners are placed at the ends and at the midspan. The girder is laterally supported at the stiffeners to avoid lateral torsional buckling failure of the structure. The bending moments are acting at both ends while shear force is acting at the midspan and the transverse force is located close to the quarter point of the beam placed above the middle point of the parallel web fold. The internal forces are analyzed at the cross-section under the midline of the transverse force. The concerning numerical model is shown by Fig. 18 in the Section 4.5.

To execute the intended numerical research program the following strategy is applied. The corner points of the 3D interaction equations, namely the numerically computed pure resistances are already known from Section 5. At first the shear force is fixed to a constant value

and the bending moment and transverse forces are varied. This method results in points along the red curves in Fig. 52. After that the transverse force is fixed and the bending moment and shear forces are varied which results in points along the blue curves in Fig. 52.

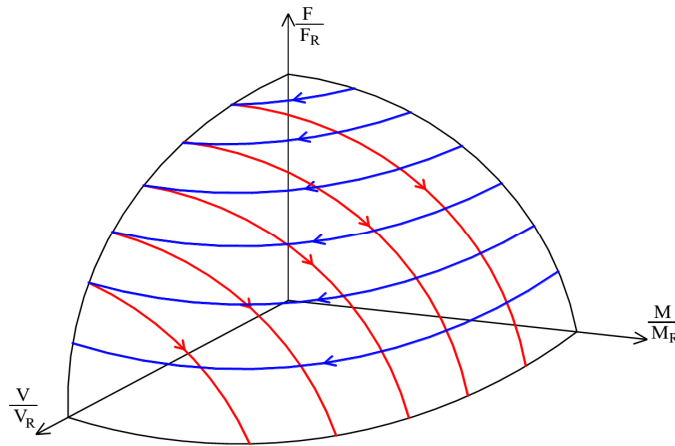


Fig. 52: Research strategy of the M–V–F interaction surface validation [100].

After the numerical simulation statistical evaluations are executed. The statistical evaluation is based on the distance between the numerically computed results and the proposed interaction curve shown in Fig. 53. The values of R_e and R_t represent the numerically computed results and its projection to the interaction curve, respectively, since the r_e and r_t represent the vectors pointing to the R_e and R_t points. The ratio of the lengths of the vectors characterizes the conformity of the proposed interaction curve.

In most of the cases the exponents of the “moving” interaction curves differ which means that no analytical solution is existing for the problem. The only way is to solve the problem numerically by using a nonlinear equation solver.

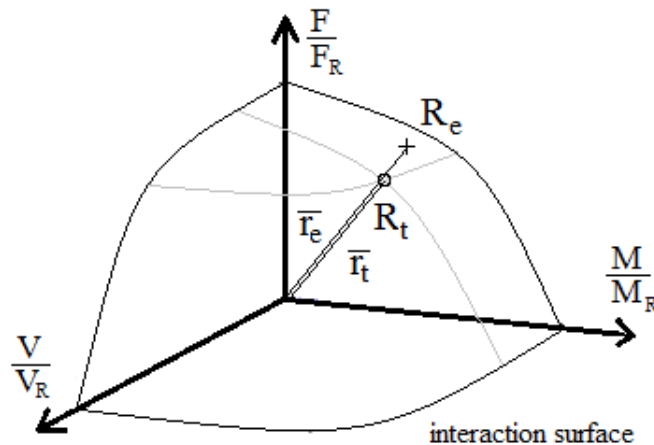


Fig. 53: Basis of the statistical evaluation.

According to the proposals of Kövesdi et al. [59, 62] for the M-F and V-F interaction and the current proposal for the M-V interaction (Section 6.2) the following M-V-F interaction equation can be proposed what is represented by Eq. (43).

$$\max \begin{cases} \left(\frac{M}{M_R} \right)^{1,2} + \left(\frac{F}{F_R} \right)^8 + \frac{1}{20} \frac{V}{V_R} \leq 1,0 \\ \left(\frac{V - 0,5F}{V_R} \right)^{1,2} + \left(\frac{F}{F_R} \right)^{1,2} + \frac{1}{20} \frac{M}{M_R} \leq 1,0 \end{cases} \quad (43)$$

where M_R pure bending moment resistance,
 V_R pure shear buckling resistance,
 F_R pure patch loading resistance.

The interaction surface defined by Eq. (43) can be seen in Fig. 54. It can be observed that the interaction surface differs from the ordinary interaction surfaces such as the M-V-F interaction surface (Fig. 65, 66 and 67 in Section 8) of flat web girders. It can be explained by the fact that in case of corrugated web girders the interaction of bending moment and shear force is almost negligible, only maximum 5% resistance reduction is observed.

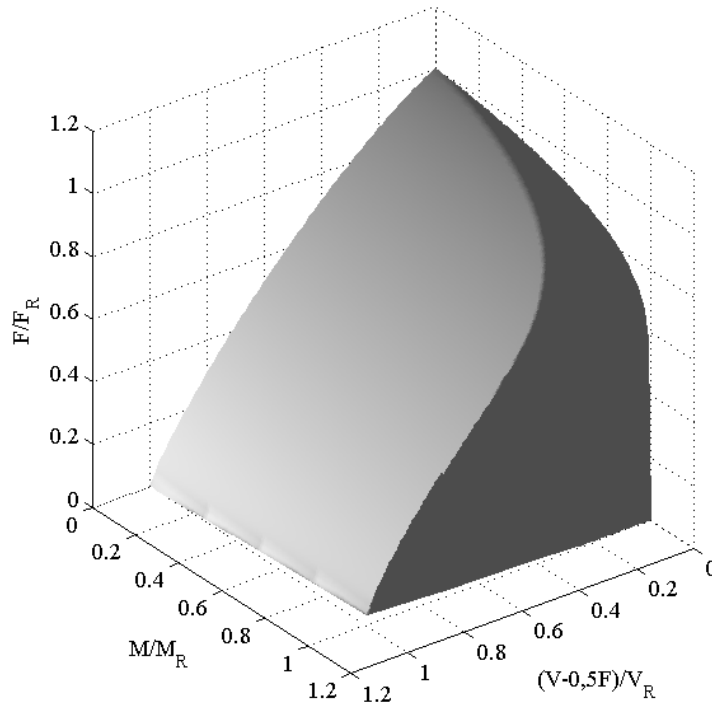


Fig. 54: Composed M-V-F interaction surface according to Eq. (43).

Due to the numerical results provided by the Section 6 other 3D interaction equations can be also proposed. Equation (44) is intended to represent the interaction equations regarding to bridge type girders while Eq. (45) represents a lower limit proposal for the analyzed parameter range. The difference between Eq. (43), Eq. (44) and Eq. (45) are the exponents in the first equations since they belong to the M-F interaction where new proposals are also needed (Section 6.3).

$$\max \left\{ \begin{array}{l} \left(\frac{M}{M_R} \right) + \left(\frac{F}{F_R} \right)^6 + \frac{1}{20} \frac{V}{V_R} \leq 1,0 \\ \left(\frac{V - 0,5F}{V_R} \right)^{1,2} + \left(\frac{F}{F_R} \right)^{1,2} + \frac{1}{20} \frac{M}{M_R} \leq 1,0 \end{array} \right. \quad (44)$$

$$\max \left\{ \begin{array}{l} \left(\frac{M}{M_R} \right) + \left(\frac{F}{F_R} \right)^{2,9} + \frac{1}{20} \frac{V}{V_R} \leq 1,0 \\ \left(\frac{V - 0,5F}{V_R} \right)^{1,2} + \left(\frac{F}{F_R} \right)^{1,2} + \frac{1}{20} \frac{M}{M_R} \leq 1,0 \end{array} \right. \quad (45)$$

where M_R pure bending moment resistance,
 V_R pure shear buckling resistance,
 F_R pure patch loading resistance.

The aim of this section is to investigate the M-V-F interaction behaviour of corrugated web girders and verify the above equations with the advanced numerical simulations.

7.2. Investigation of the M-V-F interaction surface

In this section the results of the numerical simulations are presented. In Fig. 55 the typical observed failure modes can be seen under the combined loading situation. On the interaction surface located at the top of the Fig. 55 nine different points are marked which are related to the different applied bending moment, shear force and transverse force ratios. For example marker 1 represents a loading situation when the applied patch load almost reaches the patch loading resistance of the girder while the applied bending moment and shear force are relatively small. The three failure modes in case of marker 1 can be seen just below the marker 1 under the interaction surface. It can be seen that three different failure modes belong to each marker due to the three different applied patch loading lengths. In case of the presented girder the applied patch loading lengths are 0,75, 1,705 and 2,24 times the parallel web fold (a_1). In Fig. A1 in the Appendix A the von-Mises stresses under the combined loading situation can be seen.

The failure of the structure occurs under or very close to the transverse force introduction place in every cases. In case of marker 1, 7 and 9 the dominant failure modes are clearly patch loading, shear buckling and bending failures, respectively. The further markers 2, 3, 4, 5, 6 and 8 represent the real combined failure modes. In case of marker 2, 3 and 6 the combined bending moment and patch loading failures are dominant while in case of marker 4 the combined shear buckling and patch loading failure is dominant. In case of marker 5 all the internal forces in the cross-section under the midline of the patch load have nearly the same ratios with respect to the pure capacities, nevertheless the dominant failure mode is the patch loading failure of the structure, while in case of marker 8 the failure is the combined M-V-F failure of the structure. It can be said that the most dominant failure mode under M-V-F loading is the patch loading failure of the structure.

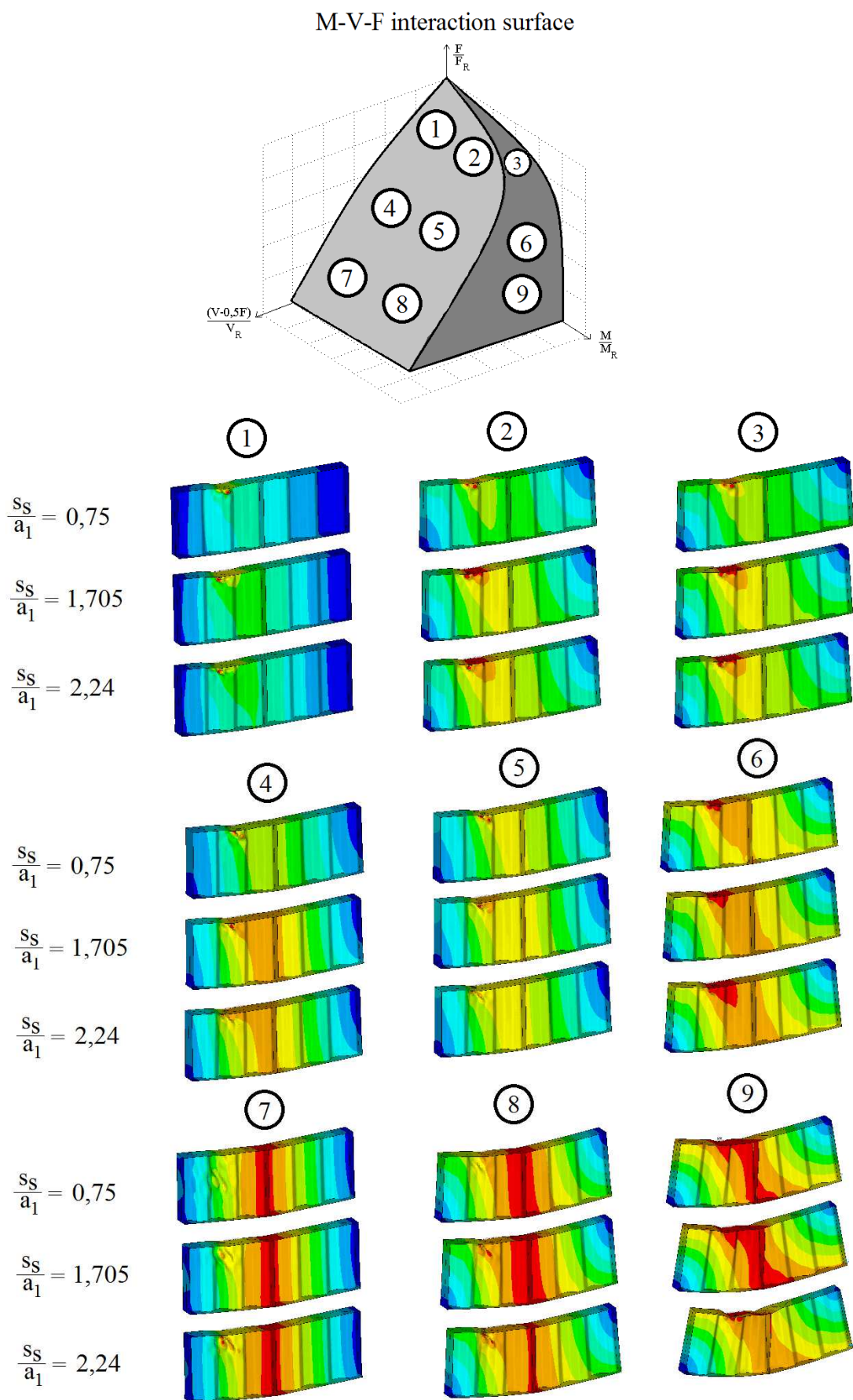


Fig. 55: Typical observed failure modes under the combinted loading situation.

In the followings the numerical results are presented and compared to the interaction equations and the correctness of the proposed equations are investigated with statistical evaluation. In this section a total number of 1700 numerical results are plotted on the diagrams. From the 1700 numerical results 100 belong to the pure resistances (Section 5), 960 belong to the interaction planes (Section 6) and finally 640 belong to the 3D interaction surface. In the following figures the green dots represent the results related to the interaction planes presented in Section 6, the magenta dots represent the results regarding to the smallest applied patch loading lengths while the black dots demonstrate the results obtained from the simulations using longer loading lengths.

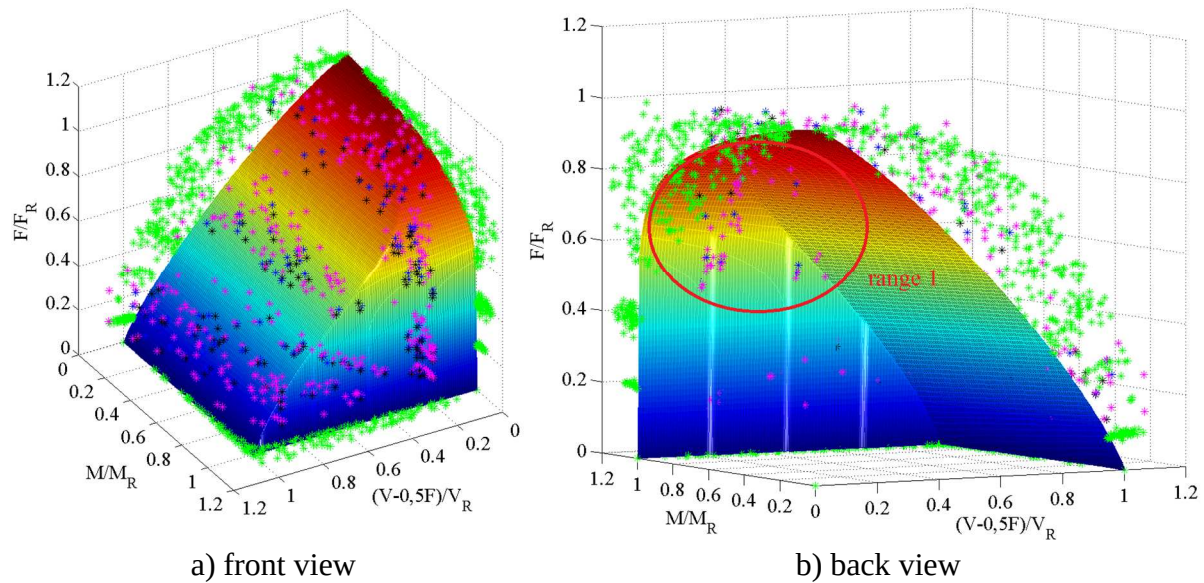


Fig. 56: Interaction surface regarding to Eq. (43) and the numerical results.

At first the numerical results are plotted coupled with the Eq. (43) in Fig. 56. All the results are normalized with respect to the numerically computed pure resistances. It can be observed that several points are under the Eq. (43) shown by the range 1, therefore correction is needed on the interaction equation.

Table 15: Statistical evaluation of the numerical results.

	Interaction surface Eq. (43)
Average	1,079
Standard deviation	0,096
Coefficient of variation	0,089
Minimum	0,831
Maximum	1,406

The statistical evaluation of the Eq. (43) with the normalized results can be seen in Table 15. The received average value is 1,079 with a standard deviation equal to 0,096. The obtained minimum value is 0,831 located on the unsafe side. Obviously this proposal is not adequate to characterise the lower bound interaction behaviour of corrugated web girders.

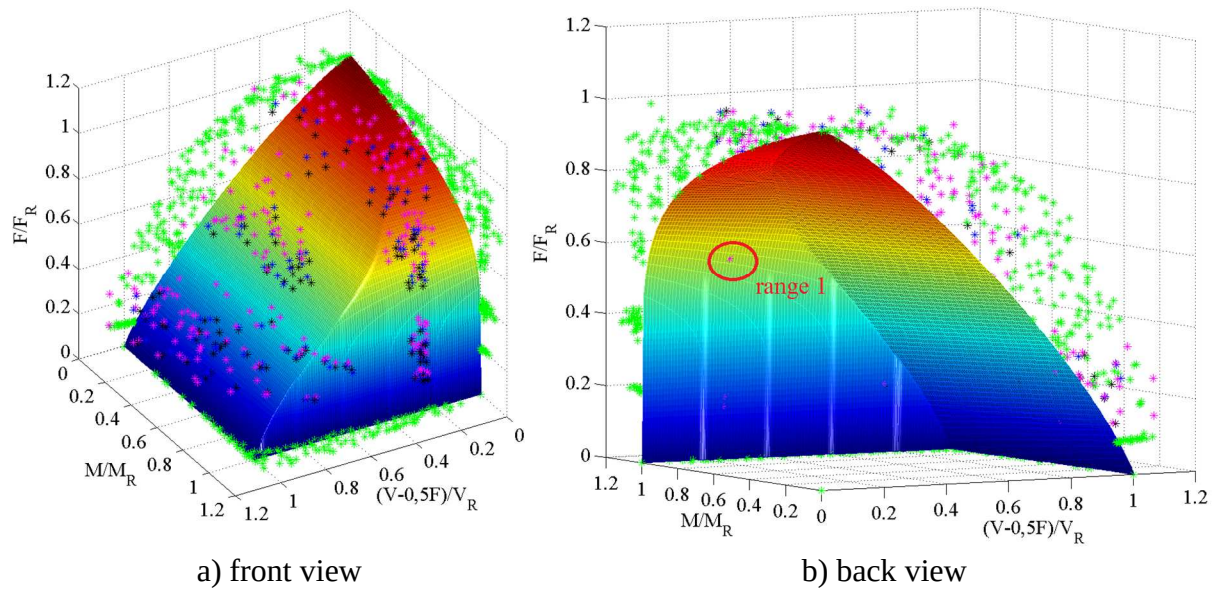


Fig. 57: Interaction surface regarding to Eq. (44) and the numerical results of bridge girders.

In Fig. 57 the numerical results of the bridge type girders are plotted with the Eq. (44). The numerical results are also normalized to the pure numerically computed resistances. It can be seen that only one intermediate point is located under the interaction surface shown by the range 1 which is practically above the surface.

Table 16: Statistical evaluation of the numerical results.

	Interaction surface Eq. (44)
Average	1,106
Standard deviation	0,088
Coefficient of variation	0,079
Minimum	0,991
Maximum	1,406

The statistical evaluation is shown by the Table 16 regarding to the Eq. (44) and the results of the bridge type girders. The received average value is 1,106 with a standard deviation equal to 0,088. The obtained minimum value is 0,991 which is equal to the minimum value found in the column 3 of Table 13 in Section 6.4 which is dedicated to evaluate the V-F interaction plane of corrugated web girders. This proposal is judged to be adequate enough for bridge type girders as a lower limit approximation.

All the numerical results and the interaction surface according to Eq. (45) are plotted in Fig. 58. The numerical results are also normalized to the pure numerically computed resistances. It can be seen in Fig. 58 that no intermediate points are found under the interaction equation.

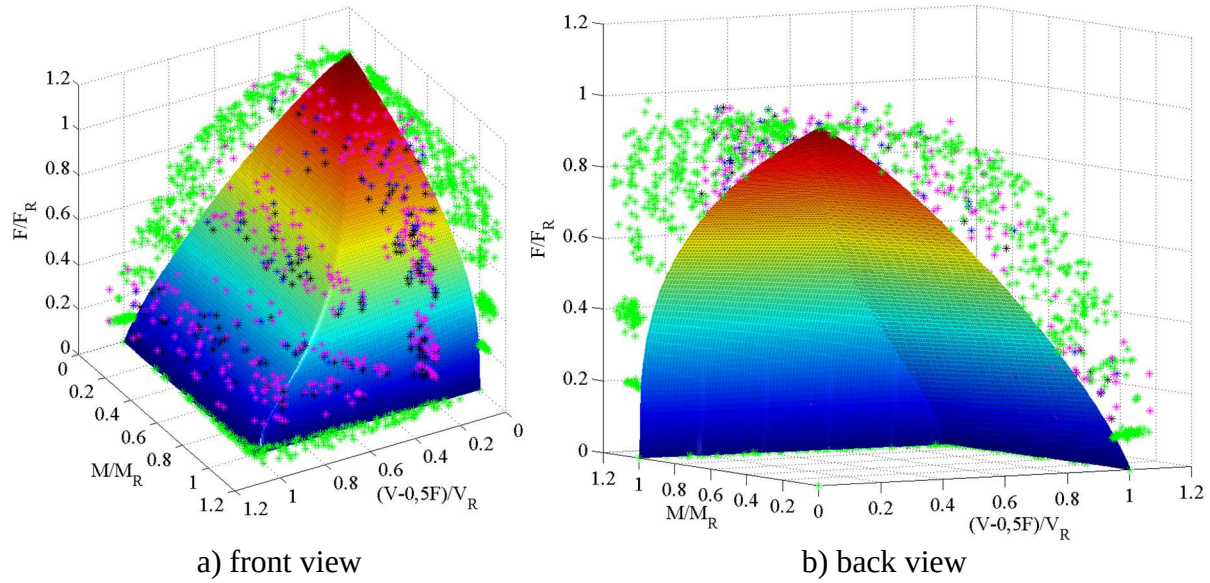


Fig. 58: Interaction surface regarding to Eq. (45) and the numerical results.

The statistical evaluation is shown in Table 17 regarding to the Eq. (45). The received average value is 1,132 with standard deviation equal to 0,098. The obtained minimum value is 0,991 which also relates to the V-F interaction plane as in case of the Table 16. This proposal is judged to be adequate enough for the whole analyzed parameter range as a lower limit approximation.

Table 17: Statistical evaluation of the numerical results.

	Interaction surface Eq. (45)
Average	1,132
Standard deviation	0,098
Coefficient of variation	0,087
Minimum	0,991
Maximum	1,448

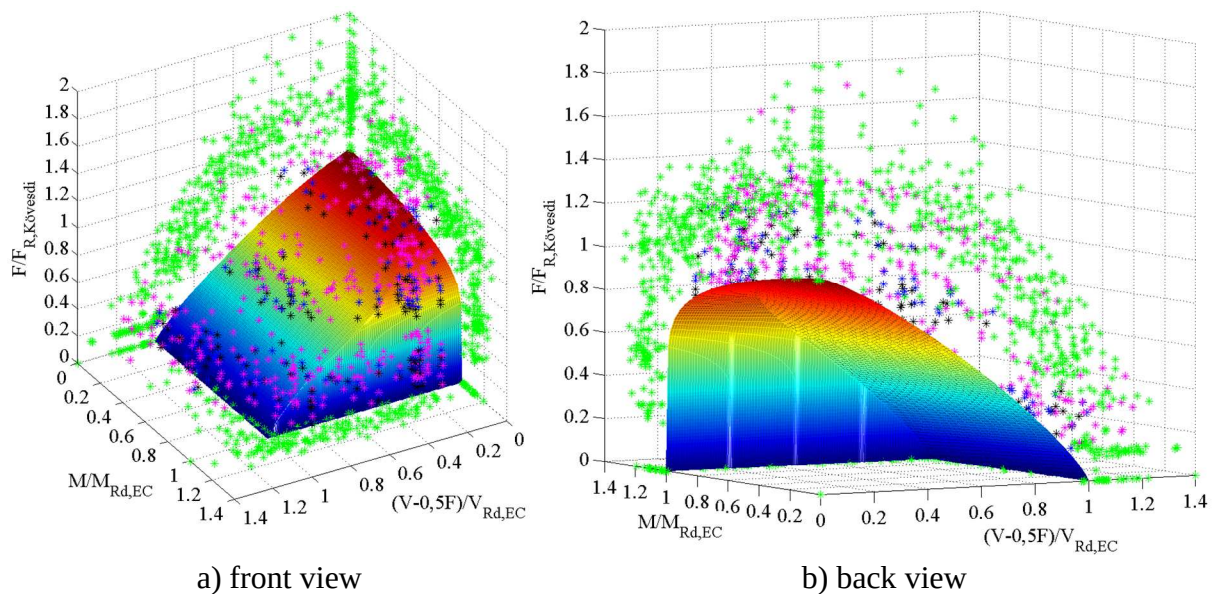


Fig. 59: Interaction surface regarding to Eq. (43) and the numerical results.

In Fig. 59 the numerical results divided by the analytically calculated pure resistances with the Eq. (43) are presented. The analytically resistance formulas are found in Eqs. (29) - (31) in Section 3.2. It can be seen that all of the intermediate points are above the interaction surface.

Table 18: Statistical evaluation of the numerical results.

	Interaction surface Eq. (43)
Average	1,297
Standard deviation	0,196
Coefficient of variation	0,151
Minimum	0,974
Maximum	2,114

The statistical evaluation of the results plotted in Fig. 59 is demonstrated in Table 18 regarding to the Eq. (43). The received average value is 1,297 with a standard deviation equal to 0,196. The obtained minimum value is 0,974 which is equal to the minimum value found in Table 8 in Section 5.3 which is dedicated to evaluate the pure shear buckling resistances of corrugated web girders. As it is written in Section 5.3 that some numerically computed pure shear buckling resistances are under the proposal of the EN1993-1-5 [1]. This observation can be attributed to the large imperfection magnitude, because it depends on the web depth ($h_w/200$).

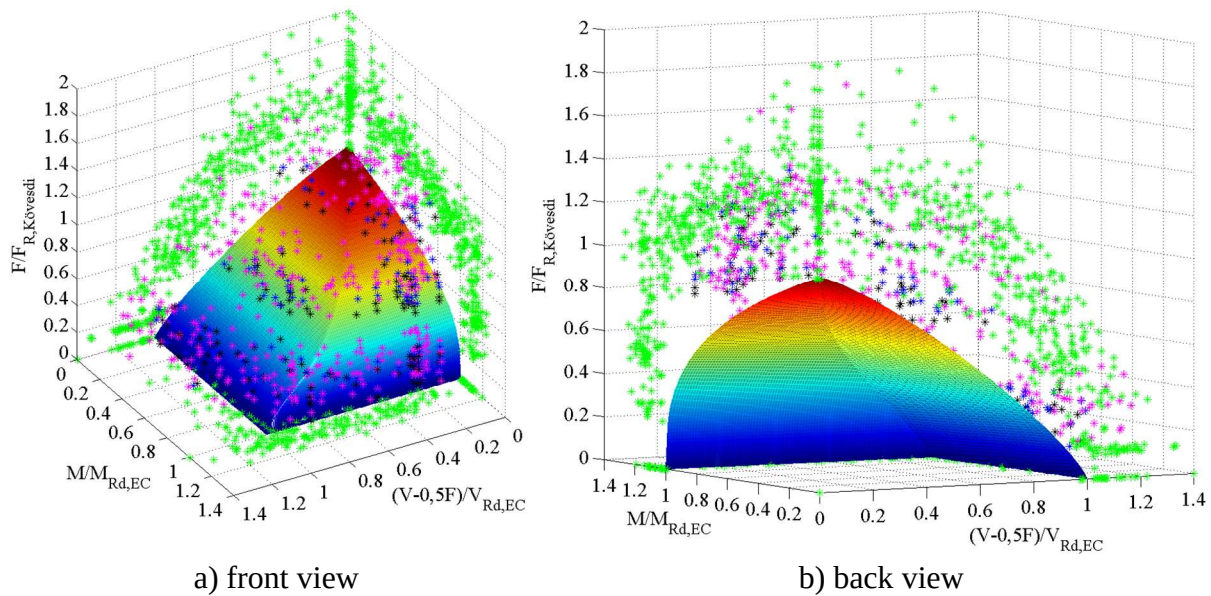


Fig. 60: Interaction surface regarding to Eq. (45) and the numerical results.

In Fig. 60 the numerical results divided by the analytically calculated pure resistances are presented. The analytical resistance formulas can be found in Eqs. (29) - (31) in Section 3.2. It can be seen that all of the intermediate points are above the interaction surface.

The statistical evaluation of the results plotted in Fig. 60 are shown in Table 19 regarding to the Eq. (45). The average value is 1,359 with standard deviation equal to 0,213. The obtained minimum value is 0,974 which is also equal to the minimum value found in Table 8 in Section 5.3. For sake of completeness it can be said that this was attributed to the applied large

imperfection amplitude. But anyway the maximum difference is 2,6% on the unsafe side which is judged to be acceptable.

Table 19: Statistical evaluation of the numerical results.

	Interaction surface Eq. (45)
Average	1,359
Standard deviation	0,213
Coefficient of variation	0,157
Minimum	0,974
Maximum	2,114

7.3. Evaluation

In this section new M-V-F interaction surfaces are investigated in order to characterize the interaction behaviour of the analyzed parameter range of corrugated web girders with a lower limit approximation.

The results show that the Eq. (45) provides the best prediction for the 3D interaction behaviour of corrugated web girders as a lower limit interaction surface. The obtained minimum value is 0,991, if the numerically computed resistances are in the denominator. In addition the minimum value is located in the V-F interaction plane.

8. Comparison with flat web girders

8.1. Concept of the comparison

This section discusses the comparison of trapezoidal corrugated web girders with ordinary flat web girders from design aspects which is important for a designer in the practice. The comparison is the so called “in-weight” comparison which means that the weight of compared girders are quasi equal by not considering the weight of the welds. Therefore the corrugated webs are changed to a flat one with an equivalent thickness calculated using the Eq. (46).

$$t_{w,eq,flat} = t_{w,trap} \cdot \frac{a_1 + a_2}{a_1 + a_4} \quad (46)$$

where $t_{w,trap}$ web thickness of the corrugated web girders (column 5 in Table 6),
 a_i characteristic lengths of the corrugation profile (Fig. 2 and Table 6).

The obtained equivalent web thicknesses of the 20 flat web girders can be seen in the column 5 in Table 20. Obviously the flange parameters, the web depth and the yield and ultimate strengths are not changed during the calculations.

Table 20: Parameters of ordinary flat web girders.

	b_f [mm]	t_f [mm]	h_w [mm]	$t_{w,eq}$ [mm]	f_y [MPa]	f_u [MPa]
Typical bridge geometries						
1	300	30	2000	4,16	296	460
2	300	20	1000	4,69	355	510
3	400	40	1500	7,03	355	510
4	400	40	1000	11,72	355	510
5	450	40	2500	9,74	355	510
6	450	40	2500	10,54	355	510
7	450	40	2500	12,00	355	510
8	300	20	2500	9,74	355	510
9	300	30	2500	9,74	355	510
10	350	30	2000	8,66	355	510
11	350	30	1200	10,00	355	510
Typical building geometries						
12	200	15	500	2,97	270	430
13	300	20	1000	4,69	355	510
14	250	16	750	5,86	355	510
15	250	16	750	6,67	355	510
16	250	16	750	5,36	355	510
17	250	16	750	5,86	355	510
18	250	16	750	5,86	355	510
19	250	16	750	5,86	355	510
20	220	12	1000	1,66	270	360

In the followings at first the corrugated web girders found in Table 6 and the unstiffened flat web girders found in Table 20 are compared with respect to their pure capacities found in the interaction equations. Then the 3D interaction behaviours are compared in case of each girder geometry.

8.2. Comparison results

The proposed M-V-F interaction equation for corrugated web girders can be found in Eq. (45) while the M-V-F interaction equation for flat web girders can be found in Eq. (C.4) in the Appendix C.

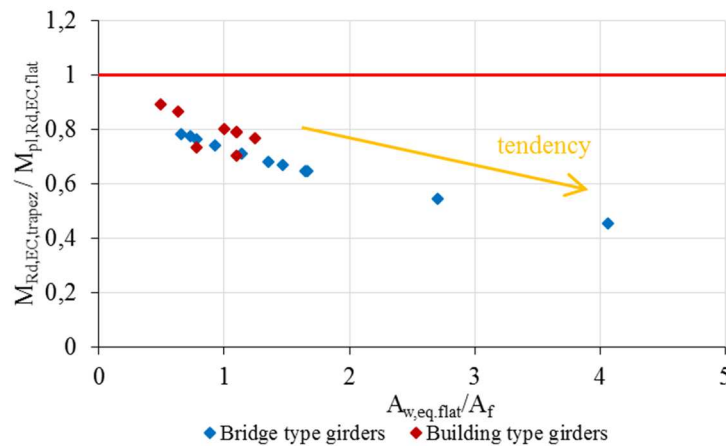


Fig. 61: Comparison of the applied bending moment resistance formulas.

It can be observed that in case of flat web girders the plastic bending moment resistance (Eq. (B.1) in the Appendix B) is in the denominator in the bending term while in case of corrugated web girders the “effective” bending moment resistance (Eq. (29) in Section 3.2) is in the denominator. The ratios of the two bending resistances for each girder are demonstrated in Fig. 61. It is observed that greater the $A_{w,eq,flat} / A_f$ ratio (area of the web and flange), the smaller is the $M_{Rd,EC,trapéz} / M_{pl,Rd,EC,flat}$ ratio. This observation is reasonable because in case of corrugated web girders the web does not contribute in the bending moment capacity because of the so called “accordion effect”.

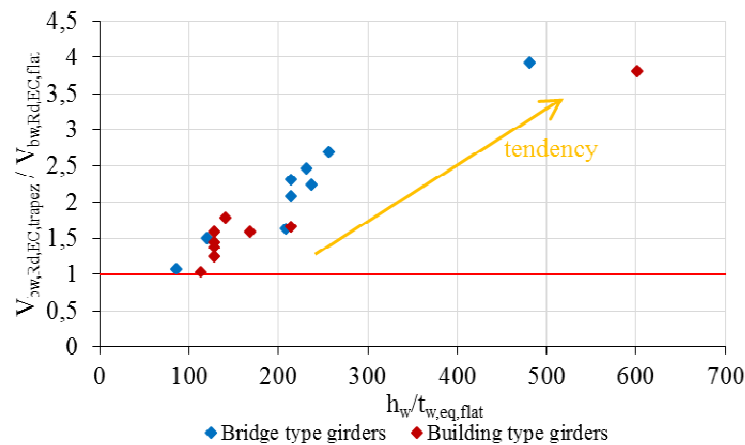


Fig. 62: Comparison of the applied shear buckling resistance formulas.

In case of both interaction equations (Eq. (45) and Eq. (C.4)) the shear buckling resistances according to the EN1993-1-5 [1] (Eq. (30) and Eq. (B.2) in the Appendix B) are in the denominators in the shear terms. The comparison of the shear resistances of corrugated and flat web girders can be seen in Fig. 62. The figure shows that using greater $h_w/t_{w,eq,flat}$ ratios the $V_{bw,Rd,EC,trapez}/V_{bw,Rd,EC,flat}$ ratios are increasing significantly. It can be observed that design shear buckling capacity of the corrugation profile can be four times greater than for flat web girders.

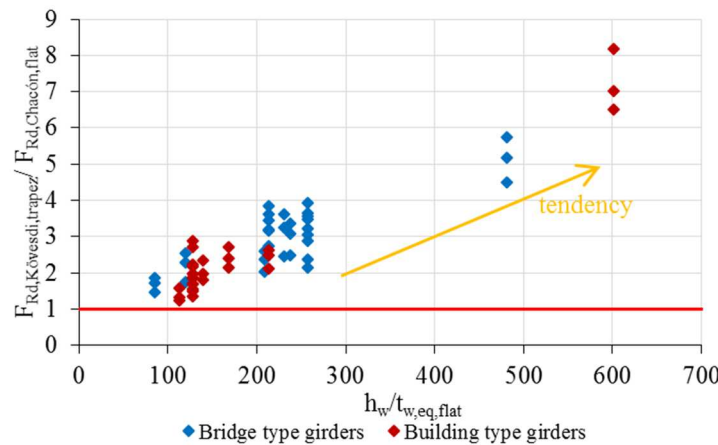


Fig. 63: Comparison of the applied patch loading resistance formulas.

In the transverse force terms the proposal of Kövesdi et al. [18] (Eq. (31)) and the proposal of Chacón et al. [103] (Eq. (B.3) in the Appendix B) are used. The comparison of the patch loading resistances of corrugated and flat web girders can be seen in Fig. 63. The figure shows that using greater $h_w/t_{w,eq,flat}$ ratios the $F_{Rd,Kövesdi,trapez}/F_{Rd,Chacón,flat}$ ratios are increasing significantly. It can be observed in the figure that the design patch loading capacity of corrugated web girders might be occasionally three or eight times greater than for flat web girders.

In the followings the M-V-F interacting stability capacities of flat and corrugated web girders are compared. The strategy is the followings:

- 1) The volumes under the interaction surfaces are determined and compared with the enclosing $1 \times 1 \times 1$ unit cube (M_R , V_R and F_R are equal to 1,0).
- 2) Comparison is taken place between flat (Table 20) and corrugated web girders (Table 6) regarding to the M-V-F interacting stability capacities.

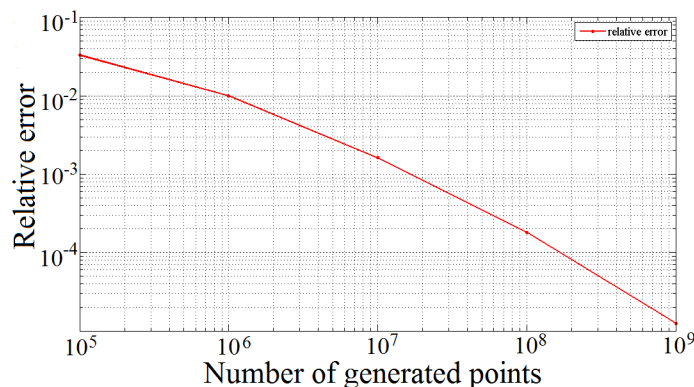


Fig. 64: Convergence of the relative error in log-log coordinate system.

During the comparison the volumes under the interaction equations are determined with the help of the Monte Carlo method in which the points are generated randomly. Figure 64 shows the convergence study of the necessarily applied generated points. Obviously the shape of the curve can change but the tendency remains the same after a new run. One can see in the figure that the relative error is decreasing when more points are generated. If 10^8 points are applied the relative error reduces to about 10^{-4} which is judged to be an acceptable precision. By using single-precision floating-point format the allocated memory is almost 400 MB while in case of 10^9 generated points it would be almost 4 GB therefore using 10^8 points is reasonable.

Table 21: Volumes under the interaction equations from the enclosing 1x1x1 cube.

	Average volume
Interaction equation of flat web girders – Eq. (C.4)	0,4413
Interaction equation of corrugated web girders – Eq. (45)	0,4906
Interaction equation of bridge type corrugated web girders – Eq. (44)	0,5332

Table 21 shows the volumes of the normalized interaction equations (M_R , V_R and F_R are equal to 1,0) from the enclosing 1x1x1 cube which has unit volume. One can observe that 44,13% of the generated points were under the interaction equations of flat web girders while in case of corrugated web girders this percentage is more advantageous with 49,06%. In addition in the paper of Kövesdi et al. [100] the analyzed patch loading lengths which is one of the most influential parameters are between 200 and 600 mm while in this diploma thesis the applied patch loading lengths are between 23 and 865 mm depending on the length of the parallel web fold (a_1).

In the followings the M-V-F interacting capacities of the 20 girders are compared using flat and corrugated web girders. This comparison is performed as follows:

- 1) For each of the 20 girders the smallest enclosing cuboids are determined at first. For instance in case of girder 18 (applied patch loading length $s_s = 318$ mm) the lengths of the edges of the enclosing cuboid are

$$\max(M_{Rd,EC,trapex}; M_{pl,Rd,EC,flat}) = 1380,1556 \text{ kNm},$$

$$\max(V_{bw,Rd,EC,trapex}; V_{bw,Rd,EC,flat}) = 601,855 \text{ kN},$$

$$\max(F_{Rd,Kövesdi,trapex}; F_{Rd,Chacón,flat}) = 697,3294 \text{ kN}$$

as Fig. 65/a and Fig. 65/b also show. The example presented by Fig. 65 is one of the average from the 20 girders. The figure clearly shows that the “maximum” bending resistance is attributed to the flat web variant while the “maximum” shear and patch loading resistances are attributed to the corrugated one.

- 2) 10^8 points are randomly generated into the 1380,1556x601,855x697,3294 cuboid.
- 3) The volumes regarding to the flat and corrugated web variants are determined thus the M-V-F interacting capacities are known.

In case of girder 18 the 38,73% of the points are under the interaction equation (Eq. 45) related to the corrugated web while it is 14,36% for the equivalent flat web girder (Eq. (C.4)). The difference between the two obtained percentages is 24,37% for the benefit of the corrugated

one which means that using an equivalent “in-weight” corrugated web instead of a flat web the M-V-F interacting capacity of the girder is increasing about 24% in average regarding to the smallest enclosing cuboid. In other words, the M-V-F interacting capacity of the corrugated one is 2,7 times greater than that of with flat web.

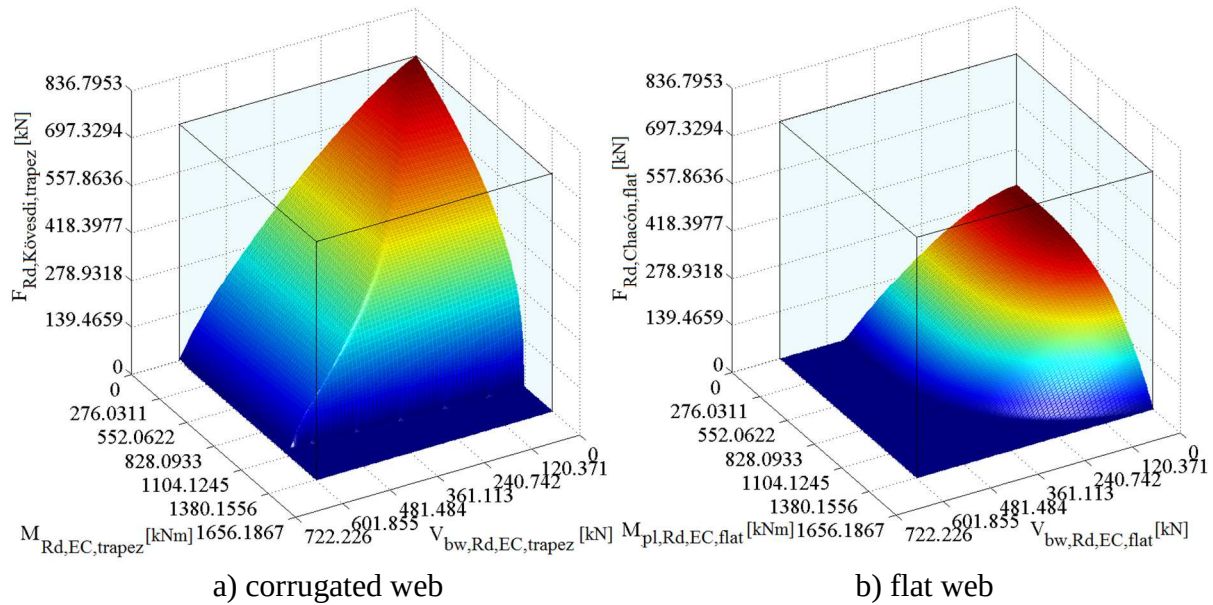


Fig. 65: Illustration of the smallest enclosing cuboid and the interaction surfaces in case of an average situation (girder 18 using $s_s = 318$ mm).

The extreme cases are also presented in this Section. In Fig. 66 the most favorable case can be seen which belongs to the girder 15 with applied patch loading lengths $s_s = 59$ mm. The obtained percentages are 37,58% and 34,50% using the same order as before, respectively. Thus it can be said that using equivalent “in-weight” corrugated web instead of a flat web girder, the M-V-F interacting capacity is increasing at least with 3,08% regarding to the smallest enclosing cuboid.

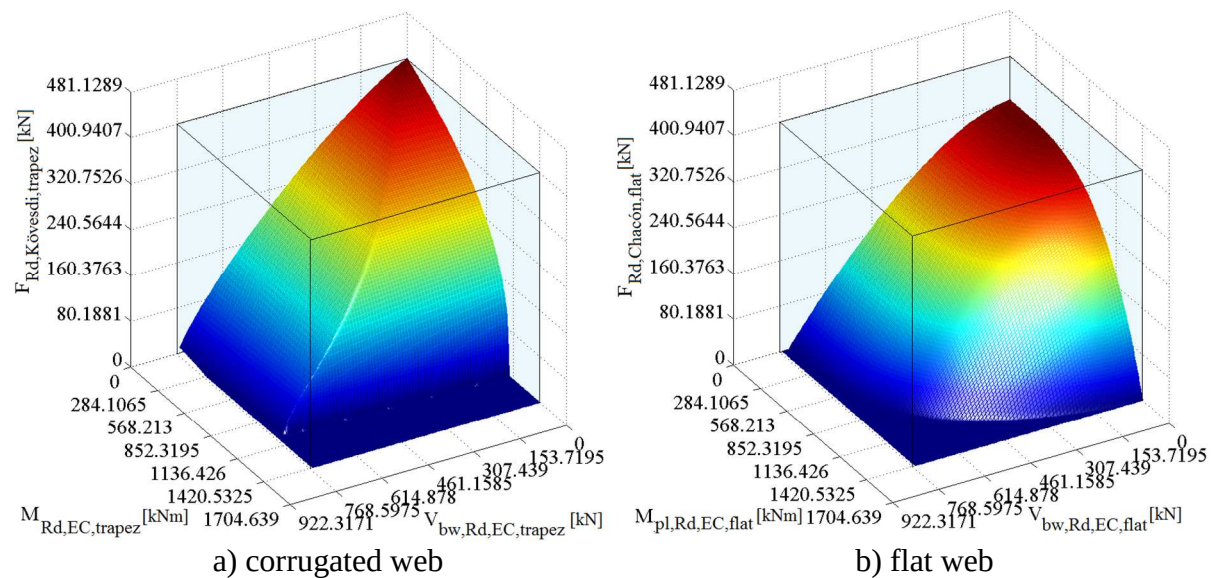


Fig. 66: Illustration of the smallest enclosing cuboid and the interaction surfaces in case of the most favorable situation (girder 15 using $s_s = 59$ mm).

Finally, the most unfavorable case is also presented in Fig. 67. The regarding results are 42,49% and 1,41% which means that using equivalent “in-weight” corrugated web instead of a flat one, the M-V-F interacting capacity is increasing even with 41,08% regarding to the smallest enclosing cuboid. By using the other definition the M-V-F interacting capacity of the corrugated one is 30,13 times greater than that of with flat web.

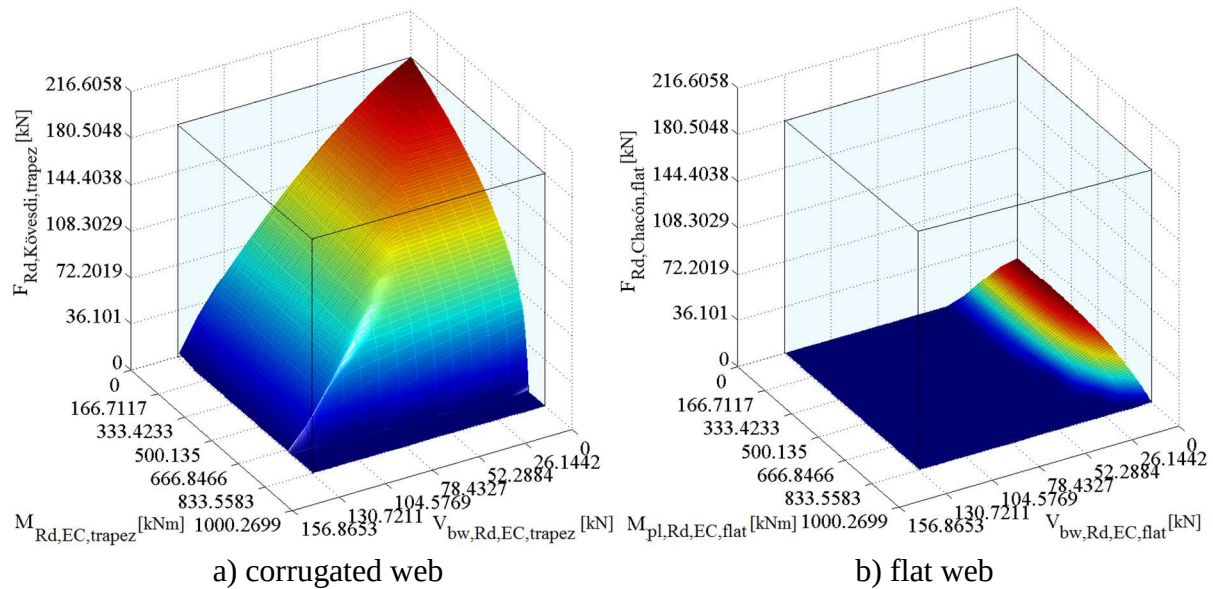


Fig. 67: Illustration of the smallest enclosing cuboid and the interaction surfaces in case of the most unfavorable situation (girder 20 using $s_s = 354$ mm).

During the comparison it was observed that the effect of the patch loading length can be significant. In case of girder 18 the obtained differences regarding to the smallest enclosing cuboid are 21,54%, 24,37% and 27,52% by using longer patch loading lengths. In case of girder 15 these are 3,08%, 5,57% and 10,49% while in case of girder 20 these are 40,71%, 40,83% and 41,08%, respectively. One can observe that in the most favorable case the relative increment is the greatest while in the most unfavorable case the relative increment is the smallest.

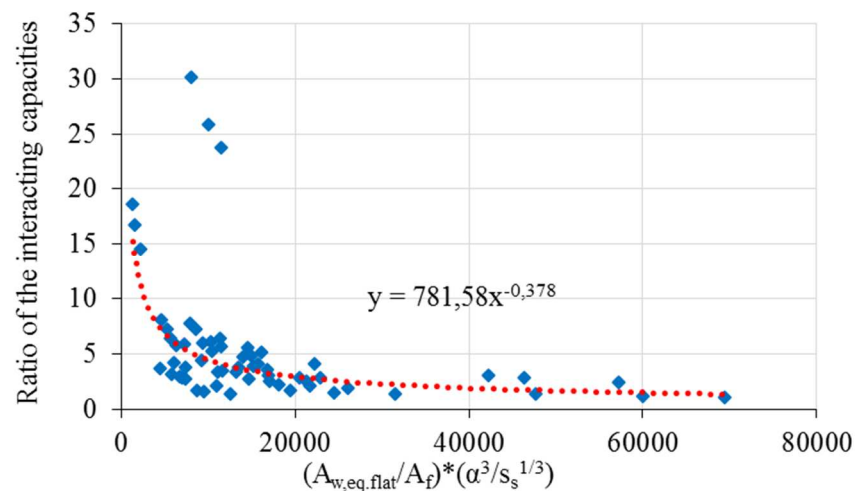


Fig. 68: Approximation of the ratio of the interacting capacity due to the corrugated web.

Figure 68 is intended to show that how many times greater is the interaction volume of a corrugated web girder than that of with flat web. In the figure the red dashed line represents the approximation of the ratio of the interacting capacities by changing the flat web to an equivalent “in-weight” corrugated web. The average ratio is 5,52.

$$ratio = 781,58 \cdot \left(\frac{A_{w,eq,flat}}{A_f} \frac{\alpha^3}{\sqrt[3]{s_s}} \right)^{-0,378} \quad (47)$$

where $A_{w,eq,flat}$ area of the web in mm² using web thickness according to the Eq. (46),
 A_f area of the flange in mm²,
 α corrugation angle substituted in degree (Fig. 2),
 s_s applied patch loading length in mm.

The ratio of the interacting capacities can be approximately calculated by the Eq. (47). The terms in the equation must be substituted as dimensionless variables. This formula can be useful if the designer wants to estimate the extra M-V-F capacity provided by the corrugated web girders.

8.3. Evaluation

In this section M-V-F interacting capacities of flat web and corrugated web girders are investigated in the frame of a so called “in-weight” comparison.

The results show that the average interacting capacity of corrugated web girders is 5,52 times greater due to the high shear buckling and patch loading resistances than that of with flat web. This average ratio is very large therefore the consideration of usage of corrugated web girders instead of unstiffened ordinary flat web girders under M-V-F loading is reasonable.

9. Conclusions and future aims

Previous studies have been already executed dealing with the combined bending moment and shear force (M-V), bending moment and transverse force (M-F) and shear and transverse force (V-F) interaction of corrugated web girders. These interaction curves may be the interaction planes of the M-V-F interaction surface. The primary purpose of the current investigations is to give a lower limit approximation for the M-V-F interaction surface regarding to corrugated web girders. The secondary purpose of this MSc thesis is to compare the so called interacting capacities of corrugated and flat web girders in the frame of an “in-weight” comparison. During the investigations the following results are achieved:

- 1) Detailed literature overview on the behavioural aspects of corrugated web girders was performed in Section 2. Numerous papers are studied dealing with stress distributions, strength, stability and also the fatigue problems of corrugated web girders.
- 2) Papers dealing with the interacting stability behaviours of corrugated web girders are also studied and the existing proposals are discussed in Section 3.2.
- 3) The interaction planes of the M-V-F interaction surface is investigated separately.
 - a. The proposal of the EN1993-1-5 [1] was found to be too conservative and a new proposal is developed for the consideration of the combined M-V loading represented by the Eq. (39) for design purposes.
 - b. The results show that the proposal of Kövesdi et al. [59] is not appropriate for the whole parameter domain, therefore a new lower limit approximation is developed and proposed for the combined M-F interaction of corrugated web girders shown by the Eq. (42) for design purposes.
 - c. The validity of the V-F interaction equation proposed by Kövesdi et al. [62] was found to be adequate enough as a lower limit approximation in the whole investigated parameter range.
- 4) According to the results a foremost proposal for the consideration of the combined M-V-F interaction of corrugated web girders is composed in form of the Eq. (45). The verification of the proposed interaction equation is performed with numerical simulations. Another suggestion is given for bridge type corrugated web girders presented by Eq. (44).
- 5) The comparison between flat and corrugated web girders shows that due to the high shear buckling and patch loading resistances of corrugated web girders the interacting capacities of corrugated web girders is more than 5 times greater in average than those of with equivalent “in-weight” unstiffened flat web.

Several researchers have proved that the use of corrugated web girders instead of flat web girders is reasonable. In this MSc thesis it is also proved that the interacting stability behaviour of corrugated web girders is also advantages.

In the future I would like to follow the research in the field of the fatigue and global stability (LTB) behaviours of corrugated web girders.

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I offer my master's thesis to the memory of my late grandfather. Especially I would say thank to him for his advices and his teachings, I will never forget them.

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Appendix A

Von Mises stresses under the M-V-F combined loading situation

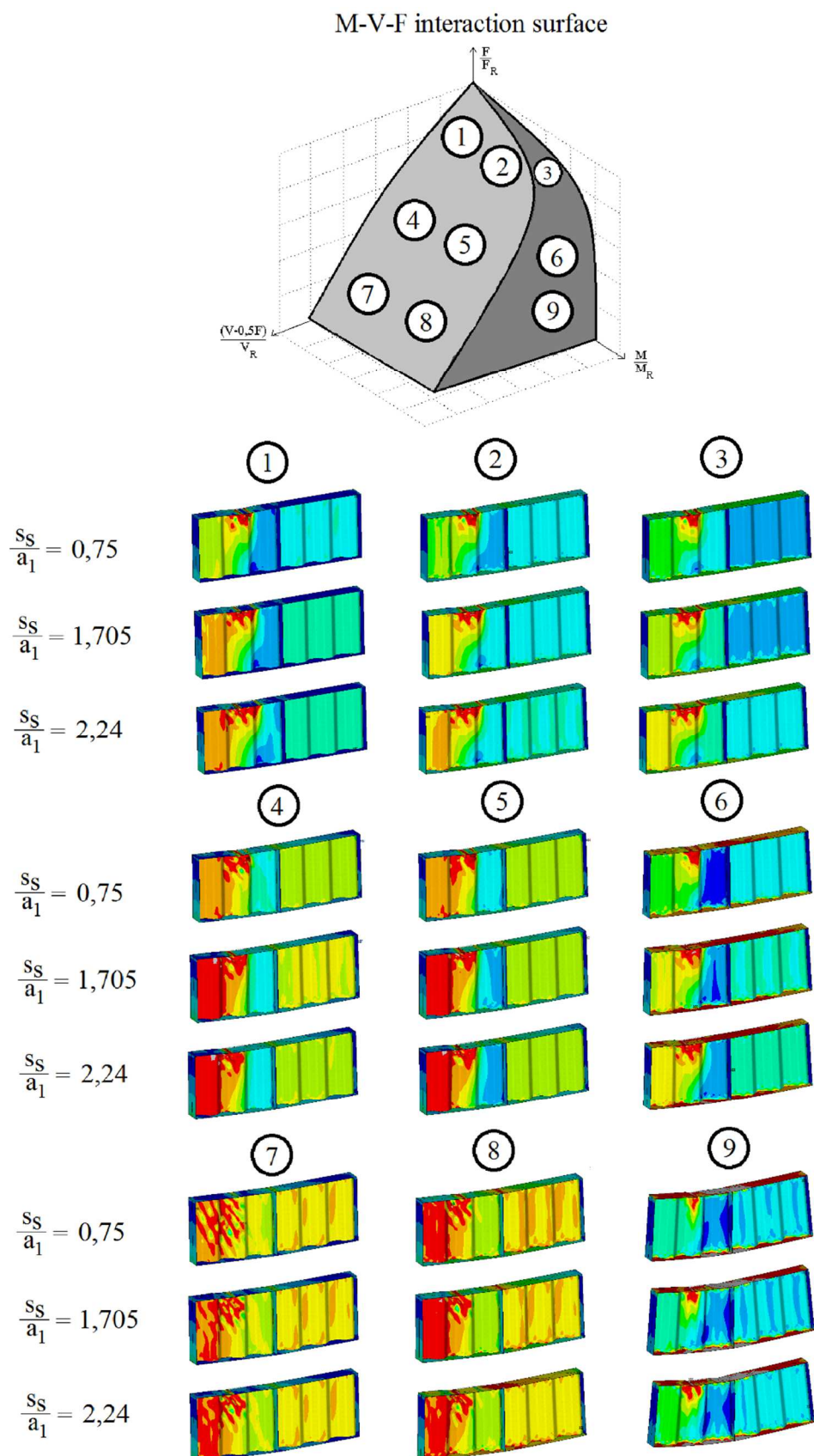


Fig. A.1: Von Mises stresses under the combined loading situation.

Appendix B

Calculation formulas for the pure resistances of ordinary flat web girders

Formulas for the pure bending moment, shear buckling and patch loading resistances have been already existing in the concerning Eurocode standard. Notwithstanding that there is recommendation in the standard for the calculation of the pure patch loading resistance of ordinary flat web girders, in the current diploma thesis the determination of the pure patch loading resistance is based on the proposal of Chacón et al. [103] which is the updated version of the Eurocode's recommendation.

The plastic bending resistance can be determined by the Eq. (B.1) below.

$$M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} \quad (B.1)$$

where W_{pl} plastic section modulus,
 f_y yield strength,
 γ_{M0} partial factor.

Procedure for the determination of the shear buckling resistance of corrugated web girders is implemented in the EN1993-1-5 [1]. The formula for the determination of the design shear capacity can be seen in Eq. (B.2) without introducing of the calculation of the reduction factor.

$$V_{bw,Rd} = \frac{\chi_w \cdot f_{yw} \cdot h_w \cdot t_w}{\gamma_{M1} \sqrt{3}} \quad (B.2)$$

where χ_w the lesser reduction factor regarding to local and global buckling,
 f_{yw} yield strength of the web,
 h_w web depth,
 t_w web thickness,
 γ_{M1} partial factor.

The patch loading resistance is determined in the current diploma thesis according to the proposal of Chacón et al. [103] which can be calculated according to the Eq. (B.3). The whole procedure is not introduced since it can be found in the concerning paper [103].

$$F_{Rd} = \frac{\chi_F \cdot f_{yw} \cdot l_y \cdot t_w}{\gamma_{M1}} \quad (B.3)$$

where χ_F reduction factor,
 f_{yw} yield strength of the web,
 l_y effective loaded length,
 t_w web thickness,
 γ_{M1} partial factor.

Appendix C

Interacting stability behaviour of ordinary flat web girders

To compare the load bearing capacity of flat web girders with corrugated web girders the literature overview is done in the field of flat web girders as well. Thus this section is dealing with the interacting stability behaviour of flat web girders. In this section not the whole literature but only one paper is referenced which is the latest one in this topic and includes all the relevant papers and discusses all of the proposals. This paper is written by Kövesdi et al. [100] in 2014.

The pure resistances of flat web girders, namely the patch loading, bending and shear resistances have been discussed for a long time and elaborated till now. These design proposals of the pure resistances are discussed in the Appendix B. There are existing interaction equations available for M-V, M-F, V-F and M-V-F interactions, respectively. All of these interaction formulas are discussed in the followings.

Interaction between bending and shear

An in-depth literature overview can be found in the paper of Kövesdi et al. [100] thus it is not discussed in the frame of this subsection. The EN1993-1-5 [1] gives recommendation for the calculation of the M-V interaction. The theoretical background of the standard is that if the bending moment exceeds the flanges' bending resistance then the interaction of bending and shear is come into action or if the shear force does not reach the 50% of the shear resistance then the reduction on the plastic bending moment capacity is not necessary.

Other types of interaction equations have been developed which is called “moving” interaction equations by taking each terms, namely bending (M), shear (V) and transverse force (F) terms with different indices (as exponents) into account. Using this approach breakpoints can be avoided in the interaction curves thus the formulas become simple. According to this approach different equations have been proposed using different indices by different authors. Based on the previously investigated M-F and V-F interaction formulas a new interaction equation for M-V is verified numerically by Kövesdi et al. [100]. In Eq. (C.1) the proposed lower limit M-V interaction equation can be seen.

$$\left(\frac{V}{V_{num,R}} \right)^{1,6} + \left(\frac{M}{M_{pl,R}} \right)^{3,0} \leq 1,0 \quad (C.1)$$

where $V_{num,R}$ numerically calculated shear buckling resistance,

$M_{pl,R}$ plastic bending resistance.

If the shear buckling resistance is calculated according to the formula of the EN1993-1-5 [1] the index of 3,0 on the bending moment term can be changed to 3,6.

Interaction between bending and patch loading

An in-depth literature overview can be found in the paper of Kövesdi et al. [100] thus it is not discussed in the frame of this subsection. The EN1993-1-5 [1] gives recommendation for the calculation of the M-F interaction considering axial forces as well. The proposal of the Eurocode contains two breakpoints in the interaction curve.

“Moving” type of interaction equations have been also developed. By applying this approach different equations have been proposed using different indices on the terms by different authors. The latest investigation was carried out by Kövesdi et al. [100]. Based on the numerical investigation they proposed the following lower limit interaction equation (Eq. (C.2)) for M-F interaction.

$$\left(\frac{M}{M_{pl,R}} \right)^{3,0} + \left(\frac{F}{F_{num,R}} \right) \leq 1,0 \quad (C.2)$$

where $F_{num,R}$ numerically calculated patch loading resistance of the web panel,

$M_{pl,R}$ plastic bending resistance.

If the analytically calculated patch loading resistance is applied according to Chacón et al. [103] instead of the numerically calculated patch loading resistance then index of 3,6 can be used on the bending moment term.

Interaction between shear and patch loading

An in-depth literature overview can be found in the paper of Kövesdi et al. [100] thus it is not discussed in the frame of this subsection. There is no recommended interaction equation found in the Eurocode regarding to the V-F interaction behaviour of flat web girders.

In this case “moving” type of interaction equations have been also developed. Different equations have been proposed using different indices on the terms by different authors. The latest investigation was carried out by Kövesdi et al. [100]. Based on the numerical investigation they proposed the following interaction equation (Eq. (C.3)) for V-F interaction.

$$\left(\frac{V - 0,5F}{V_{num,R}} \right)^{1,6} + \left(\frac{F}{F_{num,R}} \right) \leq 1,0 \quad (C.3)$$

where $V_{num,R}$ numerically calculated shear buckling resistance,

$F_{num,R}$ numerically calculated shear buckling resistance.

Interaction between bending moment, shear and patch loading

There is no recommended interaction equation can be found in the Eurocode regarding to M-V-F interaction of flat web girders. In addition, only one proposal had been existed in this topic in the literature without any verification before the paper of Kövesdi et al. [100].

Based on the previously investigated M-V, M-F and V-F interaction formulas a new interaction equation for M-V-F is composed and verified numerically by Kövesdi et al. [100]. In Eq. (C.4) the proposed lower limit M-V-F interaction equation can be seen.

$$\left(\frac{M}{M_{pl,R}} \right)^{3,0} + \left(\frac{V - 0,5F}{V_R} \right)^{1,6} + \left(\frac{F}{F_R} \right) \leq 1,0 \quad (C.4)$$

where $M_{pl,R}$ plastic bending resistance,

V_R shear buckling resistance considering the flange contribution,

F_R patch loading resistance considering the flange contribution.

If the flange contribution to the shear buckling and patch loading resistance models are neglected what is the case in the Eurocode design methods then the interaction equation can be calculated with the index of 3,6 on the bending moment term which results in a lower limit surface.