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ABSTRACT / ABSZTRAKT

Steel corrugated webs are used in building and bridge engineering due to their numerous favorable properties. In building engineering they are applied with steel flanges, however, in bridge engineering they are applied together with concrete upper and steel or concrete lower flanges. The conservative international design practice follows the recommendations of the current international literature and standards that the full cross-section shear strength, namely the integrated shear strength, can be considered by the shear strength of the corrugated web. In the case of pure steel structures the appropriateness of this assumption is verified for normal strength steel. Combining the steel corrugated web with reinforced concrete flanges the number of researches in literature is limited and there is no existing design methods. This structural layout is called "hybrid" structure according to the terminology of the international literature.

The previous experimental and advanced FE model based investigations showed that the contribution of the concrete flange is significant to the initial shear stiffness, to the shear strength of the full cross-section, to the shear buckling mode of corrugated web and to the postcritical behavior. The aim of the study is to understand the shear behavior of hybrid girders. In this research analytical and numerical analyses are carried out and the results are compared with previous full-scale experimental test results performed at Budapest University of Technology and Economics Department of Structural Engineering. As a result, the contribution of the concrete flanges are presented to the initial stiffness, to the shear strength and to the postcritical behavior of the hybrid girder.

Az acél trapézlemezeket számos kedvező tulajdonságuk miatt használják magas- és hídépítésben. Az magasépítésben acél övekkel, a hídépítésben azonban beton felső és acél vagy beton alsó övekkel együtt alkalmazzák. A konzervatív nemzetközi tervezési gyakorlat a jelenlegi nemzetközi szakirodalom és szabványok ajánlásait követi, miszerint a teljes keresztmetszet nyírószilárdsága, a trapézlemez gerinc nyírószilárdságával vehető figyelembe. Tiszta acélszerkezetek esetében ennek a feltételezésnek a megfelelőségét normál szilárdságú acélra igazolták. A szakirodalomban kevés a kutatás a vasbeton övekkel kombinált acél trapézlemezzel kapcsolatban, és nincs elfogadott tervezési módszer. Ezt a szerkezeti elrendezést a nemzetközi szakirodalom terminológiája szerint „hibrid” szerkezetnek nevezik.

A korábbi kísérleti és fejlett FE-modell alapú vizsgálatok kimutatták, hogy a beton övek hozzájárulása jelentős a kezdeti nyírási merevséghez, a teljes keresztmetszet nyírási szilárdságához, a hullámlemez nyírási csavarodási módjához és a kritikus állapot utáni viselkedéshez. A tanulmány célja a hibrid gerendák nyírási viselkedésének megértése. A kutatás során analitikus és numerikus elemzéseket végzek, és az eredményeket

összehasonlítom a Budapesti Műszaki és Gazdaságtudományi Egyetemen végzett korábbi valós méretű kísérleti vizsgálati eredményekkel. Ennek eredményeként bemutatom a beton övek hozzájárulását a hibridgerenda kezdeti merevségéhez, nyírószilárdságához és a kritikus állapot utáni viselkedéséhez.

TABLE OF CONTENTS

Abstract / <i>Absztrakt</i>	- 1 -
1. Introduction	- 5 -
2. Literature review	- 5 -
2.1. Structural behaviour	- 5 -
2.1.1. Strength properties	- 5 -
2.1.2. Stability behaviour	- 7 -
2.2. Existing bridges	- 14 -
2.2.1. Pont de Cognac, France	- 15 -
2.2.2. Maupre Viaduct, France	- 15 -
2.2.3. Kurobegawa, Japan	- 16 -
2.2.4. Koinumarukawa híd, Japan	- 16 -
2.2.5. Aigawa bridge, Japan	- 17 -
3. problem statement	- 18 -
4. Experimental background	- 18 -
4.1. Loading and boundary conditions	- 19 -
4.2. Material properties	- 20 -
4.3. Geometrical properties	- 22 -
4.4. Stiffeners	- 25 -
4.5. failure modes	- 27 -
4.6. initial stiffness	- 27 -
4.7. structural behavior and strength	- 30 -
5. FE model development and validation	- 31 -
5.1. Validation background	- 31 -
5.2. Geometric model	- 32 -
5.3. Material model	- 32 -
5.4. Finite element mesh	- 33 -
5.5. Loads and Boundary conditions	- 34 -
5.6. Analysis type	- 35 -
5.7. Applied imperfections	- 35 -
5.8. Convergence study	- 37 -

6. Numerical Study.....	- 38 -
6.1. Load case #1 (B-1-1).....	- 38 -
6.2. Load case #2 (B-1-2).....	- 40 -
6.3. Load Case #3 (B-1-3).....	- 42 -
6.4. Load Case #7 (B-4)	- 44 -
6.5. Load Case #8 (B-5)	- 46 -
6.6. Conclusion.....	- 48 -
7. Bending-shear interaction	- 49 -
8. Conclusions.....	- 54 -
9. Acknowledgement.....	- 56 -
10. References.....	- 57 -

1. INTRODUCTION

Composite beams with corrugated web are used in bridge structures which tends because of its numerous advantageous properties. Due to the characteristics of corrugation, girders with corrugated steel webs leads to advantages compared to flat web beams. The web has a small stiffness in longitudinal direction therefore the prestressing force remains in the concrete flanges. The resistance against shear buckling (local and global) is greater compared to flat webs, so the number of stiffeners may be reduced. Due to a better shape against shear buckling, the stiffness is higher with reduced web thickness compared to flat web. These properties lead to a better use of the steel. With thinner web, designers can reach at least the same resistance. Therefore, the dead load of the web is reduced. The lighter the structure is, the easier the structure will be implemented. Hence in case of bridges, reducing the dead loads for incremental launching is none neglectable. Another advantage is that the concreting difficulty of thin reinforced concrete webs disappears, which also results in a faster and easier building process, which is the most widespread kind of webs in box girders for bridges.

2. LITERATURE REVIEW

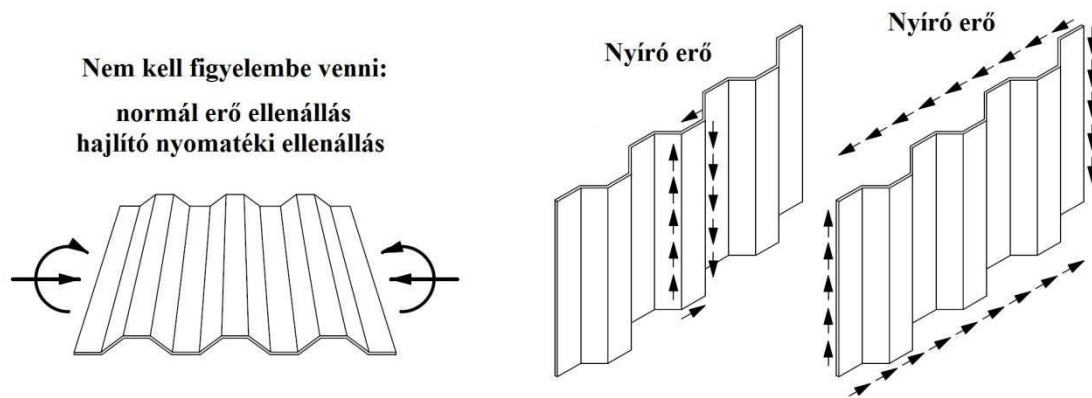
2.1. STRUCTURAL BEHAVIOUR

Research has been carried out in many universities and research institutes around the world to understand the special structural behaviour of corrugated-steel girders. In the following, I will present the strength properties, stability behaviour of these researches, as well as the structural behaviour of the trapezoidal web, the so-called "accordion effect".

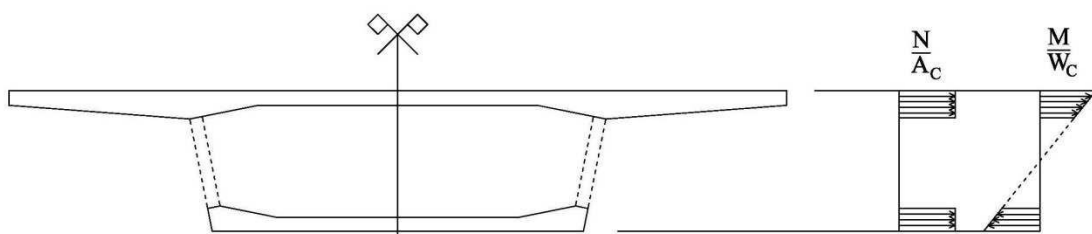
2.1.1. Strength properties

The CSW's (corrugated steel web) longitudinal stiffness is negligible in the design practice, especially in bridge engineering. Experiments have shown that 15-15% of the upper and lower height of the web plate can be taken into account when calculating the bending stiffness of the girder. [1] Once the effective cross-section has been determined, the strength analysis of the superstructure is carried out in a similar way to the strength analysis of prestressed reinforced concrete girders. [2]

Figure 2.1.1.1. illustrates the strength behaviour. The figure shows that the CSW cannot be considered for axial compression and bending. Thus, the cross section that can be considered for normal force and bending moment for hybrid CSW closed sections is shown in Figure 2.1.1.2.

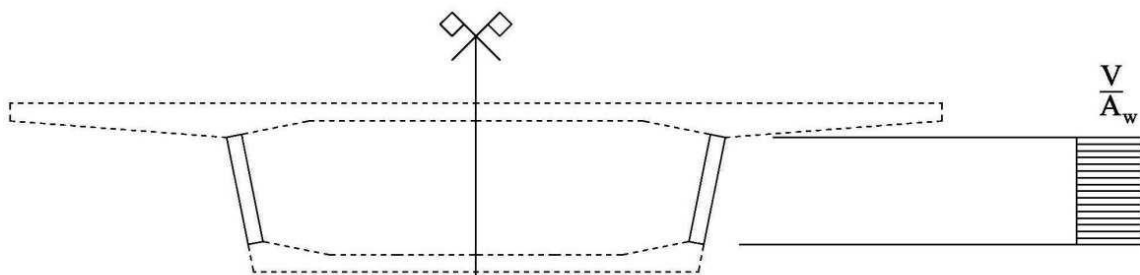


2.1.1.1. Figure: Strength properties of CSW [1]



2.1.1.2. Figure: Cross section to be considered for normal force and shear [1]

The shear flow distribution in the CSW girder is shown in Figure 2.1.1.3. Based on numerical methods and experiments with test specimens, the assumption that the cross-sections remain plane under normal directional loading (e.g., tension in one reinforced concrete flange) and that the shear force acting in the cross-section is almost entirely absorbed by the trapezoidal plate girder is verified [3]. Figure 2.1.1.3 shows the distribution of shear stress along the height of the CSW girder. This method can be applied to bridges in the engineering practice [2].

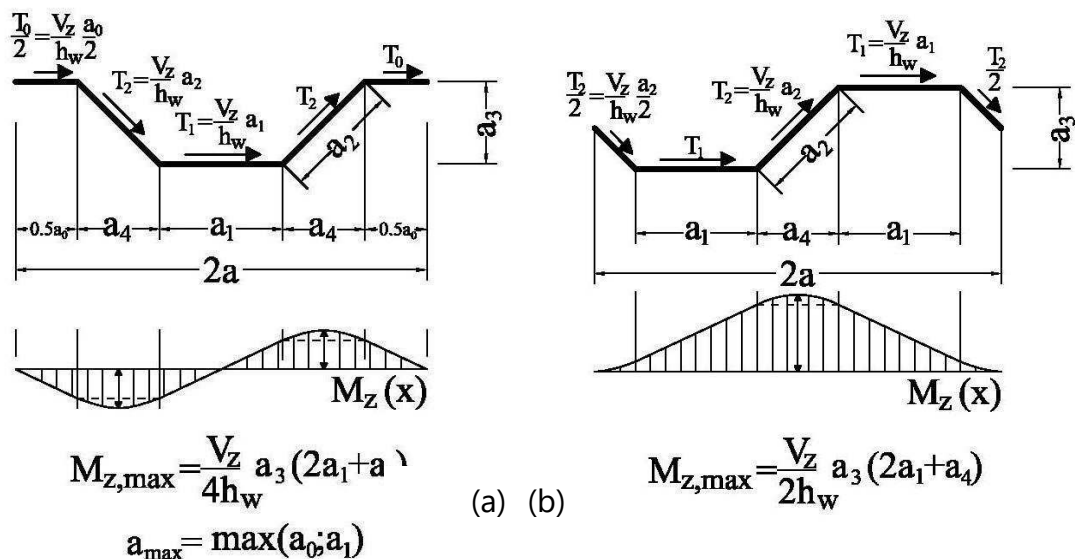


2.1.1.3. Figure Cross section to be considered for shear [1]

The strength of the section must also take into account the additional stresses due to the special structural behaviour of the CSW. Research also at the BME Department of Structural Engineering has investigated steel trapezoidal plate girder "I" beams. Following these investigations, it was possible to build a numerical model, validated by

experimental test results, which demonstrated that the shear flow in the steel trapezoidal plate induces transverse bending moments in the flanges. The transverse bending moments generate normal stresses in the longitudinal direction in the girder flanges in addition to the pure bending normal stresses [4][5][6][7].

Analytical models for calculating the transverse bending moments are already available, developed from tests on specimens and numerical models. The value of the transverse moment is strongly influenced by the location of the support and the period number of the CSW. Analytical procedures have been developed for the worst cases, which depend on the location of the girder supports. Figure 2.1.1.4/a shows the maximum transverse bending moment that can be analytically calculated for a girder supported on a parallel web section. Figure 2.1.1.4/b gives a recommendation for the manual calculation of the maximum transverse bending moment for a beam supported on a diagonal web section.



2.1.1.4. Figure: Analytical recommendations for transverse moment calculation [15]

The magnitude of the transverse torques also strongly influences the type of shear connection that can be used.

2.1.2. Stability behaviour

For steel structures, the problem of stability of the web plate arises for particularly tall, slender plates. The risk of this is greatly reduced by the use of trapezoidal CSW plates. Compared with conventional composite bridges, the shear load capacity of these webs is significantly higher than of flat plates of the same plate thickness. The ribs of the webs divide the large plate into smaller plates, which significantly reduce the tendency to buckling. As a result, the use of a large number of stiffening ribs can be avoided. In addition, the torsional stiffness of the trapezoidal CSW is higher than

that of a simple flat plate, which can have a positive effect on the number of cross braces required.

In the following subsections I will describe the typical stability phenomenons of trapezoidal CSWs.

2.1.2.1. Local shear buckling

Shear buckling failure can be elastic or inelastic for high or moderate web slenderness, respectively. In addition, based on elastic stability design proposals separately consider different buckling modes such as local and global buckling. Local buckling can be considered according to the conventional isotropic plate buckling theory [7] shown by Eq. (1).

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

In which k_L is the local shear buckling coefficient, it depends on the boundary conditions:

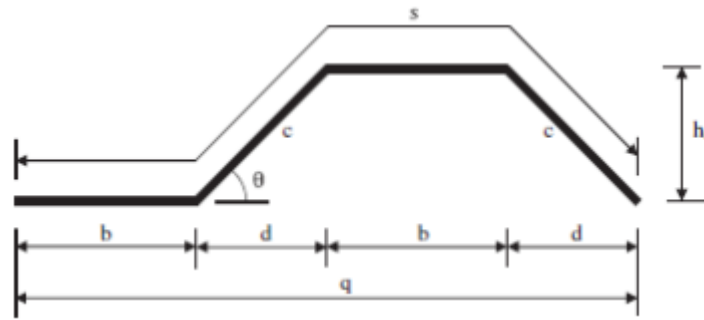
Boundary conditions	k_L
All edges simply supported [3]	$k_L = 5.34 + 4 \left(\frac{w}{h}\right)^2$
Vertical edges simply supported; horizontal edges clamped [3]	$k_L = 5.34 + 2.31 \left(\frac{w}{h}\right) - 3.44 \left(\frac{w}{h}\right)^2 + 8.39 \left(\frac{w}{h}\right)^3$
All edges clamped [3]	$k_L = 8.98 + 5.6 \left(\frac{w}{h}\right)^2$
The Eurocode 3 recommend using [6]	$k_L = 5.34$ (Trapezoidal web) $k_L = 5.34 + \left(\frac{h_{rs}}{h_w t_w}\right)$ (Sinusoidal web)
Estimated formula when 2 vertical stiffeners are spaced form a distance a [13]	$k_L = \left(5.34 + \frac{h_r^{0.5} s}{h_w t_w^{0.5}} + \frac{h_w}{a}\right)$

where k_L equals 5.34 (simply supported edges) to 8.98 (clamped edges), $\nu=0.3$ is the Poisson-ratio, Eq. (2) can be simplified for simply supported edges according to Eq. (2):

$$\tau_{cr,L}^{el} = 4.83 * E * \left(\frac{t_w}{a_{max}}\right)^2 \quad (2)$$

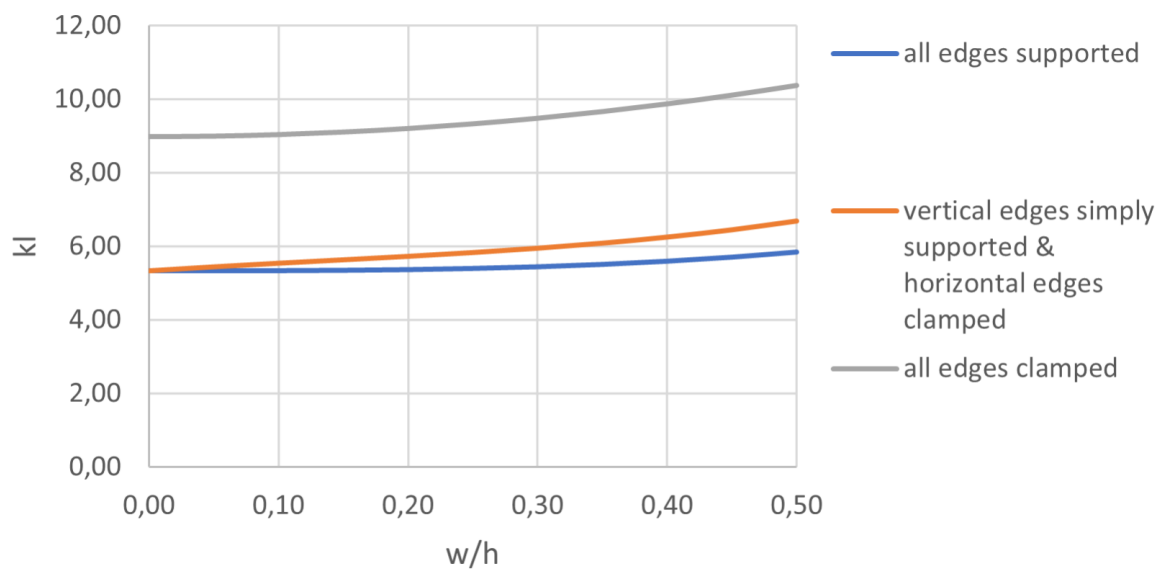
Where w is the length of a half wave of a sinusoidal web. For a corrugated web, the length is approximated by taking the maximum between the inclined panel width c and the horizontal panel width b .

$$w = \max(b; c) \quad (3)$$



2.1. Figure: Scheme of the geometry of a corrugated web

The boundary conditions mentioned above doesn't fit exactly the real boundary conditions of a corrugated web. This horizontal edges are usually welded to the upper and lower flanges while the vertical edges are supported by adjoining plates which are perpendicular to the web.



2.2. Figure: Evolution of the k_l coefficient

2.1.2.2. Global shear buckling

Considering global shear buckling when all folds are involved in the buckling Easley and McFarland [8] proposed a formula based on orthotropic plate buckling theory; shown by Eq. (4):

$$\tau_{cr,G}^{el} = k_G * \frac{D_y^{0.75} * D_x^{0.25}}{t_w * h_w^2} \quad (4)$$

In which:

- k_G is the global shear coefficient. It depends on the boundary conditions; it is resumed in [9]

Boundary conditions	k_G [38]	k_G [Easley]	k_G [EC3]
All edges simply supported	31.6	36	32.4
All edges clamped	59.2	68.4	---

- D_x is the bending stiffnesses per unit length of the corrugated web about the longitudinal axis

$$D_x = \frac{Et_w^3}{12(1-\nu^2)} \frac{q}{s} \quad (5)$$

- D_y is the bending stiffnesses per unit length of the corrugated web about the vertical axis

$$D_y = \frac{EI_y}{q} \quad (6)$$

- I_y , the second moment of area of one wavelength of the corrugated web about its own centroidal axis.

$$I_y = \frac{t_w h_r^2}{2} \left(\frac{c}{3} + b \right) \quad (7)$$

- q , s , t , h_r , c and b the geometrical parameter of the corrugated web show on the schema on Figure 2.3

In 2022 Mohammed Amin Alikhanifard [6] gives a new approach to determine the global elastic critical shear stress. A dimensionless coefficient can be computed depending on the corrugation angle, inclined and flat folds in the form of Eq. (8).

$$F(\alpha, \beta) = \sqrt{\frac{(1+\beta) \cdot \sin^3 \alpha}{\beta + \cos \alpha}} \cdot \left(\frac{3\beta+1}{\beta^2(\beta+1)} \right)^{0.75} \quad (8)$$

where $\beta = \frac{a_1}{a_2}$ is a geometrical coefficient, and α is the angle of web corrugation. After that, the global elastic critical shear stress can be computed according to Eq. (9).

$$\tau_{cr,G,Mohammed}^{el} = k_G * \frac{E * t_w^{0.5} * a_1^{1.5}}{12 * (1-\nu^2)^{0.25} * h_w^2} * F(\alpha, \beta) \quad (10)$$

The yielding strength is presented by Eq. (11) according to the von Mises yield criterion.

$$\tau_y = \frac{F_y}{\sqrt{3}} \quad (11).$$

where F_y is the yield strength. Some investigations give proposal for the inelastic critical shear stress as the square root of the product of the yield strength and elastic critical shear stress presented by Eq. (12). It has relevance in case of moderate web slendernesses.

$$\tau_{cr}^{inel} = \sqrt{0.8 * \tau_y * \tau_{cr}^{el}} \leq \tau_y. \quad (12)$$

2.1.2.3. Interactive shear buckling

In 1984 Leiva and Bergfelt et al. [10] proposed a formula for the calculation of the interactive elastic critical shear stress as the interaction of local and global elastic critical shear stresses in the form of Eq. (13). It suggests that local and global shear buckling occur together. They suggested a first order interactive formula with $n=1$ where the material inelasticity and yielding are not considered.

$$\frac{1}{(\tau_{cr,L}^{el})^n} = \frac{1}{(\tau_{cr,L}^{el})^n} + \frac{1}{(\tau_{cr,G}^{el})^n} \quad (13)$$

In 1988 Linder and Aschinger [11] 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. Sause [12] recommended exponent of $n=3$.

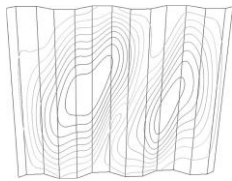


Fig. 2.4: Shape of the global shear buckling [13]

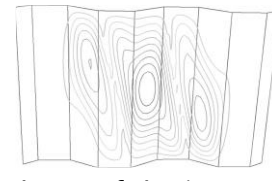


Fig. 2.5: Shape of the interactive shear buckling [13]

In 2008 Yi et al. [13] 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. (12)) provides a good estimation of the shear strength of corrugated steel webs. In addition, a new proposal for the relationship between ρ - λ s was provided. Moon et al. [14] 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, nevertheless a new proposal for the relationship between ρ - λ 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. In 2022 Mohammed Amin Alikhanifard [6] suggested Eq. (12) with $n=4$.

Elgaaly et al. [15] in 1996 suggested firstly to take the local and global inelastic critical stresses explicitly into account in the first order interactive formulae in the form of Eq. (13). This was confirmed by Luo and Edlund [16] in 1996, Johnson and Cafolla [17] in 1997 and Galambos [18] in 1998. In addition, Johnson and Cafolla proposed a design formula for the determination of the shear buckling stress of the corrugated profile.

$$\frac{1}{(\tau_{cr,L}^{inel})^n} = \frac{1}{(\tau_{cr,L}^{inel})^n} + \frac{1}{(\tau_{cr,G}^{inel})^n} \quad (13)$$

The same interaction equation considering the inelastic behaviour was proposed by Abbas et al. [19] in 2002 with $n=2$, Hiroshi et al. [20] in 2003 with $n=4$ and Driver et al. [21] 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. [21] 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 ($\rho-\lambda_s$).

There are two further methods for taking the interactive critical shear stress into account. The first one (*Eq. (14)*) is suggested by El-Metwally [22] 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. The new interaction equation was confirmed by Sayed-Ahmed [23] in 2003 with $n=3$ and $u=1$, Sause et al. [12] in 2011 with $n=3$ and $u=2$ and Hassanein et al. [24] 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.

$$\frac{1}{(\tau_{cr,l}^{el})^n} = \frac{1}{(\tau_{cr,L}^{el})^n} + \frac{1}{(\tau_{cr,G}^{el})^n} + u \frac{1}{(\tau_y)^n} \quad (14)$$

Eq. (15) was proposed by Eldib [25] 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.

$$\frac{1}{(\tau_{cr,l}^{el})^n} = \frac{1}{\min(\tau_{cr,L}^{el}; \tau_{cr,G}^{el})^n} + \frac{1}{(\tau_y)^n} \quad (15)$$

Finally the current European standard practically takes only the minimum of the local and global elastic critical shear stresses.

2.1.2.4. Shear buckling resistance

European standard gives design model for shear resistance calculation for corrugated web girders in the form of Eq. (16).

$$V_{bw,Rd} = \chi_c * \frac{F_y}{\gamma_{M1} * \sqrt{3}} * h_w * t_w. \quad (16)$$

where χ_c is the reduction factor considering local and global shear buckling, γ_{M1} is the partial safety factor for stability check. For local and global shear buckling the relative slendernesses can be determined separately according to Eqs. (17) and (18), respectively.

$$\lambda_{c,L} = \sqrt{\frac{\tau_y}{\tau_{cr,L}^{el}}} \quad (17)$$

$$\lambda_{c,G} = \sqrt{\frac{\tau_y}{\tau_{cr,G}^{el}}} \quad (18)$$

Having the relative slendernesses the reduction factors for local and global shear buckling can be separately computed according to Eqs. (19) and (20), 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.

$$\chi_{c,L} = \frac{1,15}{0,9 + \lambda_{c,L}} \leq 1,0 \quad (19)$$

$$\chi_{c,G} = \frac{1,5}{0,5 + \lambda_{c,G}^2} \leq 1,0 \quad (20)$$

The final reduction factor is the minimum of the above reduction factors regarding local and global shear buckling according to Eq. (21).

$$\chi_{c,EC} = \min(\chi_{c,L}; \chi_{c,G}) \quad (21)$$

In the case of the interactive (elastic/inelastic) critical shear stress the relative slenderness is presented by Eq. (22) where "n" represents the exponent of the interactive equations.

$$\lambda_{I,n} = \sqrt{\frac{\tau_y}{\tau_{cr,I}^n}} \quad (22)$$

For the shear buckling resistance different authors proposed the same formula of Eq. (16) by substituting different reduction factors. Some different reduction factors are presented below.

Yi [13] in 2008, n=1:

$$\chi_{c,Yi} = \begin{cases} 1 & , ha \lambda_{I,1} < 0.6 \\ 1 - 0.614 * (\lambda_{I,1} - 0.6) & , ha 0.6 < \lambda_{I,1} \leq \sqrt{2} \\ \frac{1}{\lambda_{I,1}^2} & , ha \lambda_{I,1} > \sqrt{2} \end{cases} \quad (23)$$

Sause and Braxtan [12] in 2011, $n=3$:

$$\chi_{C,Sause} = \left(\frac{1}{(\lambda_{I,3})^6 + 2} \right)^{\frac{1}{3}} \quad (24)$$

Leblouba et al. [26] in 2018, $n=4$:

$$\chi_{C,Leblouba,2018} = \frac{1}{\left(1 + \left(\frac{\lambda_{I,4}}{1.58} \right)^{1.6} \right)^{1.15}} \quad (25)$$

Leblouba et al. [27] in 2022, $n=3$:

$$\chi_{C,Leblouba,2022} = \frac{1}{1 + \left(\frac{\lambda_{I,3}}{1.4} \right)^{1.7}} \quad (26)$$

Mohammed Amin Alikhanifard [6] in 2022, $n=4$:

$$\chi_{c,Mohammed} = \begin{cases} 1, & \text{if } \lambda_{I,4} \leq 0.54 \text{ inelastic buckling} \\ \frac{1.15}{0.9 + \lambda_{I,4}}, & \text{if } 0.54 < \lambda_{I,4} \leq 1.7 \text{ elastic buckling} \\ \frac{1.5}{0.5 + \lambda_{I,4}^2}, & \text{if } \lambda_{I,4} > 1.7 \text{ elastic buckling} \end{cases} \quad (27)$$

El-Metwally [22] and Sayed-Ahmed recommended [23] the direct application of the interactive critical shear stress in the shear resistance formula *Eq. (28)* without any reduction factor.

$$V_{bw,R} = \tau_{cr,I} * h_w * t_w \quad (28)$$

2.2. EXISTING BRIDGES

Corrugated web beams aren't a very widespread worldwide construction's method. France was the first country to build bridges with corrugated web, indeed they build the two first bridges in history. Both of them were part of an experimental programme to investigate the behaviour of composite prestressed concrete bridges beams with corrugated beams. The idea was to lower the weight of the beams to reach higher spans.

However many beam bridges have been built, in China and Japan. There are now more than half a hundred hybrid girder bridges, with corrugated-steel-webs. Hybrid CSW (corrugated-steel-web) girder structures are used for extradosed and cable-stayed bridges as well as traditional girder bridges. There are four extradosed and one cable stayed bridges. Including two in Hungary, the Móra Ferenc extradosed bridge it leads the M43 motorway over the Tisza river near Szeged. The other hungarian example is the fourth extradosed hybrid bridge, spanning the Danube linking Paks and Kalocsa.

2.2.1. Pont de Cognac, France

The first hybrid girder bridge with CSW was put to service in France in 1986. Pont de Cognac was the first bridge with box-cross section, which webs were corrugated-steel plates. The Cognac bridge is a continuous four-span structure. Its longest span is 43m. The height of the structure is 2.285m bot at the supports and midspan. The CS plates used have a constant thickness also: 8mm. The angle of inclination of the trapezoidal plate section is 35 degrees.



2.6. Figure: Cognac bridge, France [28]

2.2.2. Maupre Viaduct, France

The second hybrid girder bridge, which was also built in France, was put to service in 1987. This bridge was the Maupre Viadukt, which has a triangular box cross-section. The lower flange is a 610mm diameter CHS section with concrete infill. The webs are CS, the deck is prestressed reinforced concrete. The structure is a continous seven-span girder. The main clearance is 53.6m. The height of the structure is constant 3.1m along the longitudinal axis. The thickness of the steel trapezoidal plate girders used is 8 mm both in span and above supports.



2.7. Figure: Maupre Viaduct, France [29]

2.2.3. Kurobegawa, Japan

The first railway bridge to be built as a hybrid girder bridge with corrugated steel webs (CSW) is the Kurobegawa Bridge in Japan. The bridge is a continuous fifteen-span girder. Of the bridge's 761 m length, the middle 344 m was built as a hybrid system. A stiffer connection was designed to work the reinforced concrete flanges and webs together, which was embedded, double perforated plate.



2.8. Figure: Kurobegawa bridge, Japan [3]

2.2.4. Koinumarukawa híd, Japan

The Koinumarukawa Bridge is a continuous six-span girder bridge in Japan. It has a maximum span of 81m and a total length of approximately 430m. The design was intended to avoid interference with construction work by post-tensioning tubes in the upper and lower concrete flanges. It was therefore decided to use only external tendons in the whole structure. For this reason, all cantilever tendons in each segment were anchored in concrete anchor blocks in the upper corners of the segments. This means that the shear forces from the anchorage are distributed between the remaining parts of the concrete slabs and the corrugated steel plates. In order to investigate the shear forces, it was necessary to clarify the proportion of shear forces distributed to each side. Therefore this bridge, real-size specimens were needed. [30]



2.9. Figure: Koinumarukawa bridge, Japan [31]

2.2.5. Aigawa bridge, Japan

For a long time, the record-holder bridge in this category was the German Altwipfergrund bridge, with a maximum span of 115 metres. But the CSW girder bridge structure with the largest span (179 m) is currently in Japan. The Aigawa bridge has two superstructures and as the span is curved, the inner and outer superstructure have different lengths: 636m (50.4+120.0+179.0+99.5+3x50.0+33.9), 545.5m (65.4+142.0+170.0+120.5+44.4). The bridges were built with balanced cantilever method. The maximum structural height is 11.5m, which has caused manufacturing difficulties. The design of this exceptionally large steel plate required special research and testing by the manufacturers. An interesting feature of the bridge is that the deck can be widened with cantilevers on both sides if the volume of the traffic requires it.



2.10. Figure: Aigawa bridge, Japan [32]

3. PROBLEM STATEMENT

As I described in section 2.1, the Eurocode design recommendation is that the designer should not take the reinforced concrete flange into account when determining the shear resistance. While experimental and FE results show that the contribution of the RC flange to the shear resistance is significant. In my paper, I would like to determine the extent of this contribution based on full-scale experimental results and numerical results obtained at the Budapest University of Technology and Economics.

Based on the experimental results a preliminary design proposal is developed in order to understand the behavior. In the future I would like to extend my work with numerical parametric study of CSW girders to develop a reliable design method for the determination of the integrated shear resistance.

4. EXPERIMENTAL BACKGROUND

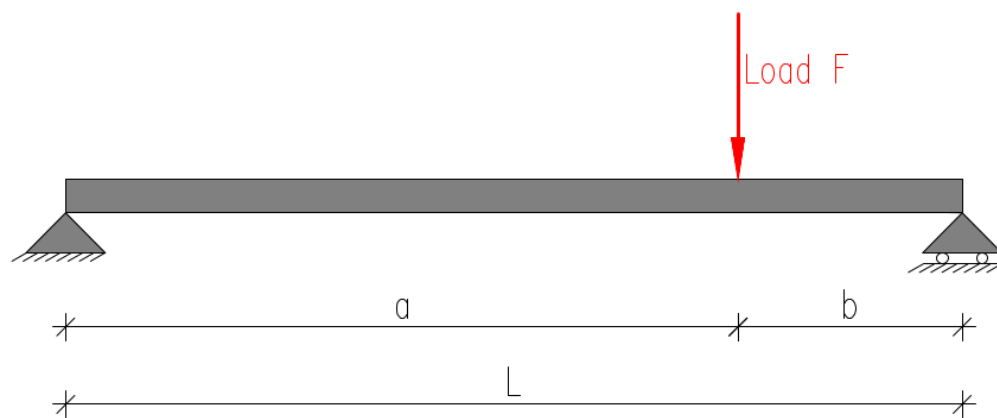
At the Department of Structural Engineering at the Budapest University of Technology and Economics in Hungary, an experimental and numerical research program is carried out on steel-concrete composite and hybrid girders having trapezoidally corrugated webs. The analysed girders are formed by reinforced concrete top flanges and steel (composite) or pre-stressed concrete (hybrid) bottom flanges

with steel corrugated web. The shear connectors between the steel-to-concrete junctions are embedded type connections using transverse reinforcement through the web openings. The research program includes study of the embedded shear connections by push-out tests; the bending-shear interaction behaviour with special focus on the contribution of the concrete flanges to the shear resistance. In this program, some large-scale tests were carried out in 2019 to study the load carrying capacity. In this part of this study, 5 large scale composite beams (8 m long) have been studied. This section discusses the experimental background.

In my work I focused on the composite girders, I plan to continue my research with the hybrid girders.

4.1. LOADING AND BOUNDARY CONDITIONS

According to the Eurocode standard and the geometry of the cross-section, we can expect certain type of failure. Table 4.1.2. presents the expected failure modes. It will be presented and discussed. The loading case are described below, the goal was to study the bending-shear interaction under various bending and shear utilizations. (Figure 4.1.2.)



4.1. Figure: Loading scheme

4.1.1. Table: Load position, web thickness, expected yielding and failure of the tested beams

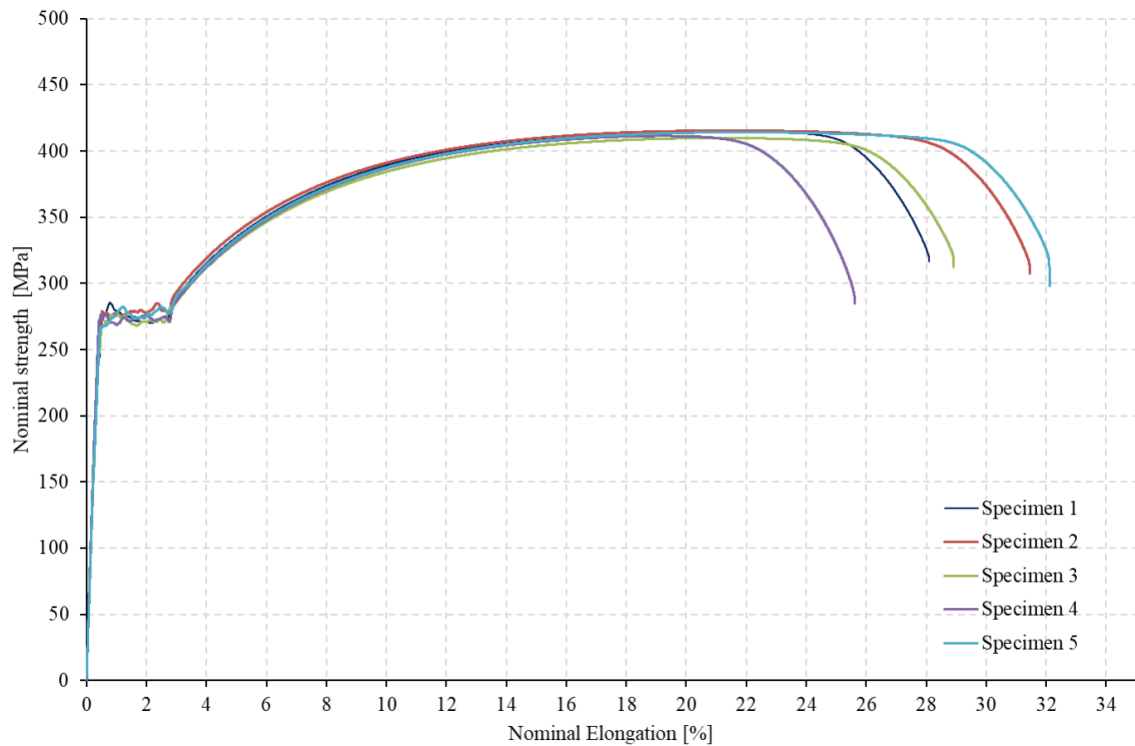
Girder No.	L [m]	a [m]	b [m]	t_w [mm]	Failure
B-1-1	8	7	1	4	shear buckling of the web

B-1-2	7	6	1	4	shear buckling of the web
B-1-3	5.67	4.67	1	4	shear buckling of the web
B-2-1	8	6.67	1.33	4	shear buckling of the web
B-2-2	6.67	5.17	1.5	4	shear buckling of the web
B-3	8	6	2	4	Column type failure of the concrete flange (bending failure)
B-4	8	5	3	4	Column type failure of the concrete flange (bending failure)
B-5	8	4	4	4	Column type failure of the concrete flange (bending failure)

4.2. MATERIAL PROPERTIES

The prescribed concrete quality is C40/50. For the corrugated steel web, a steel yield strength is 315 MPa, and for the flanges the steel grade is S275.

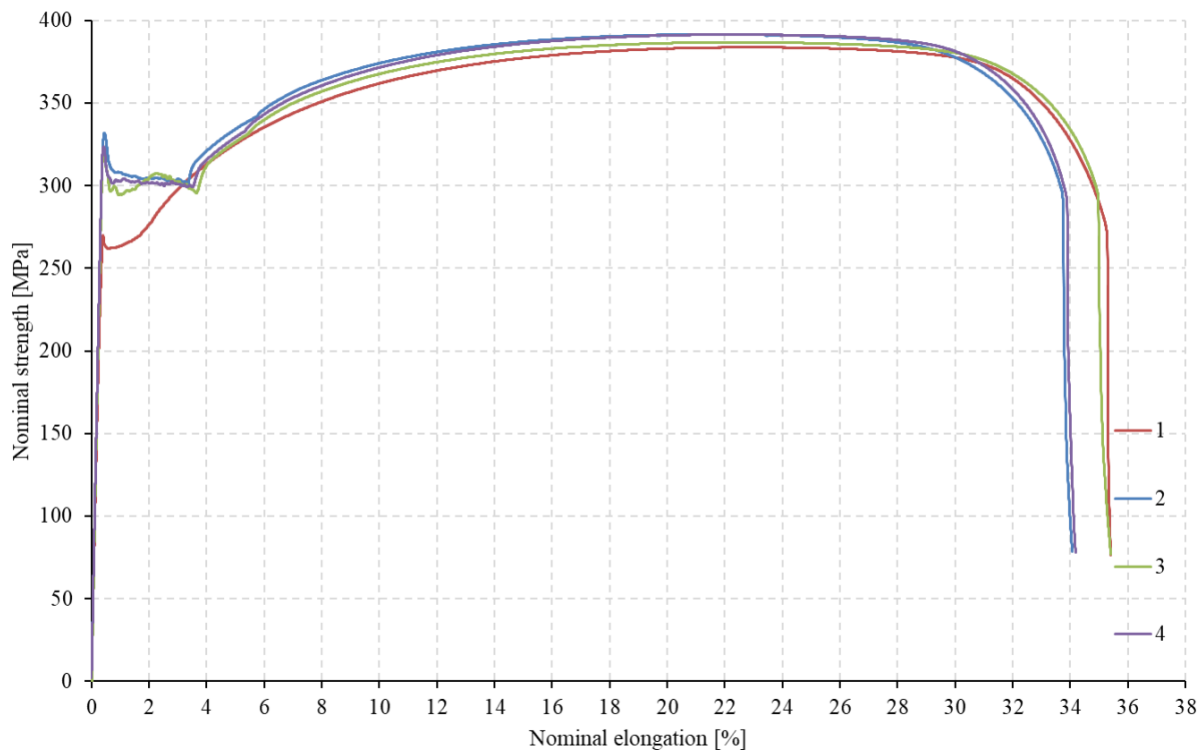
The tested specimens of steel are represented bellow in the Figure 4.2. The average yielding strength of the steel flange is 275 MPa and the average ultimate strength is 413 MPa (4.2.1. Table). The average yielding strength of the web panel is 311 MPa and the tensile strength is 390 MPa (4.2.2. Table) from coupon tests. Unfortunately, we could not evaluate the young modules of the coupon test, the reason is that the deflection sensor was not quite sensitive enough to have precise results. Therefore, the young modulus value of 210 GPa will be considered in this paper. As the material shows good approximation of the prescribed values, the steel strength of 275 MPa and 315 MPa will be considered for the web and the flanges, respectively. For the Girder 1 a value of 260 MPa will be taken.



4.2.1. Figure: Flange material properties

4.2.1. Table: Yield and tensile strength value for the flanges

	Yield strength [MPa]	Tensile strength [MPa]
Specimen 1	270,55	411,36
Specimen 2	274,65	414,14
Specimen 3	280,60	415,05
Specimen 4	277,31	415,47
Specimen 5	275,17	409,81
Mean	275,66	413,16



4.3. Figure: Web material properties

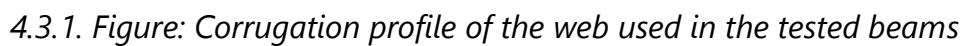
4.2.3. Table: Yield and ultimate strength value for the web

	Yield strength [MPa]	Tensile strength [MPa]
Specimen 1 (4 mm)	262,59	391,54
Specimen 2 (4 mm)	319,53	391,54
Specimen 3 (4mm)	307,81	386,75
Specimen 4 (4mm)	307,84	391,44
Mean	311,73	390,31

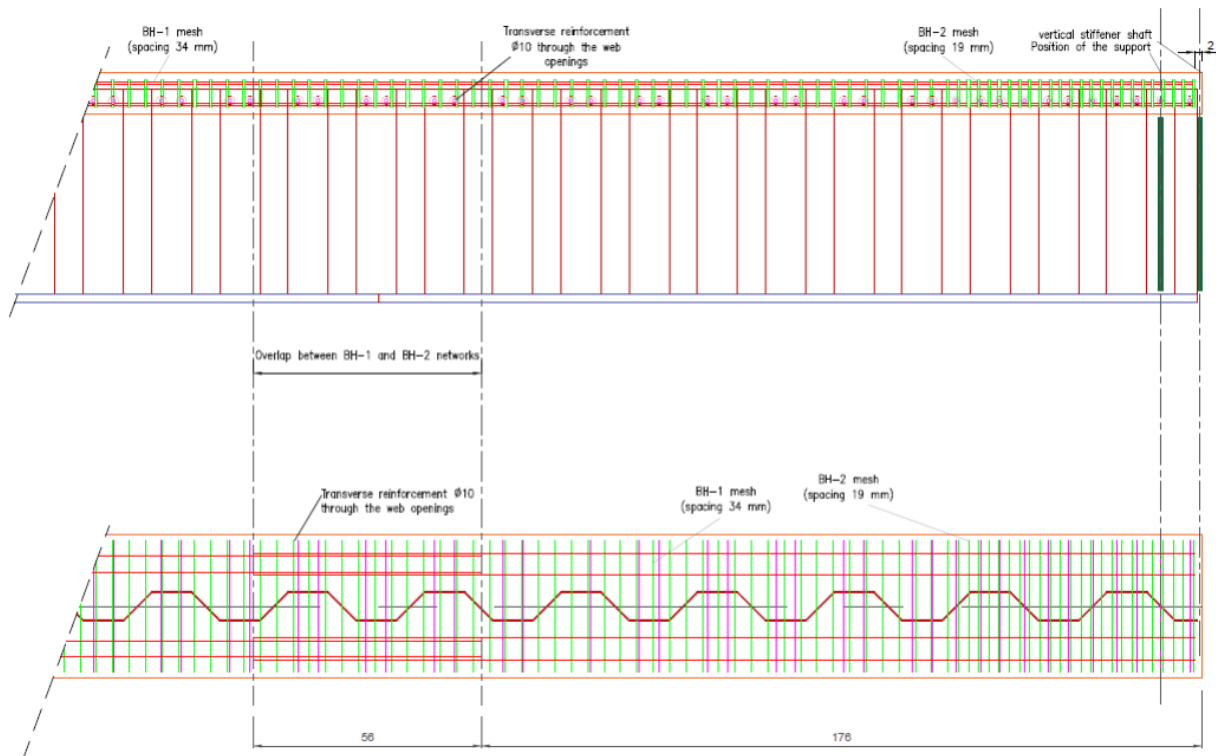
The stress of the concrete will be evaluated in the range of 40 to 50 MPa. The young modulus of the concrete will also be evaluated by numerical analysis. The concrete strength was not measured since sample cubes were not created.

4.3. GEOMETRICAL PROPERTIES

The web corrugation profile is shown by 4.4. Figure. The thickness of the web is 4 mm. This corrugated web has the same geometry for all the tested beams. The steel flange is welded to the corrugated web at the bottom and at the top the web is embedded in the concrete slab. The connection between both element is assured with transverse rebars with diameter of 10mm which pass through oval web openings of 24x20mm size.



- 23 -

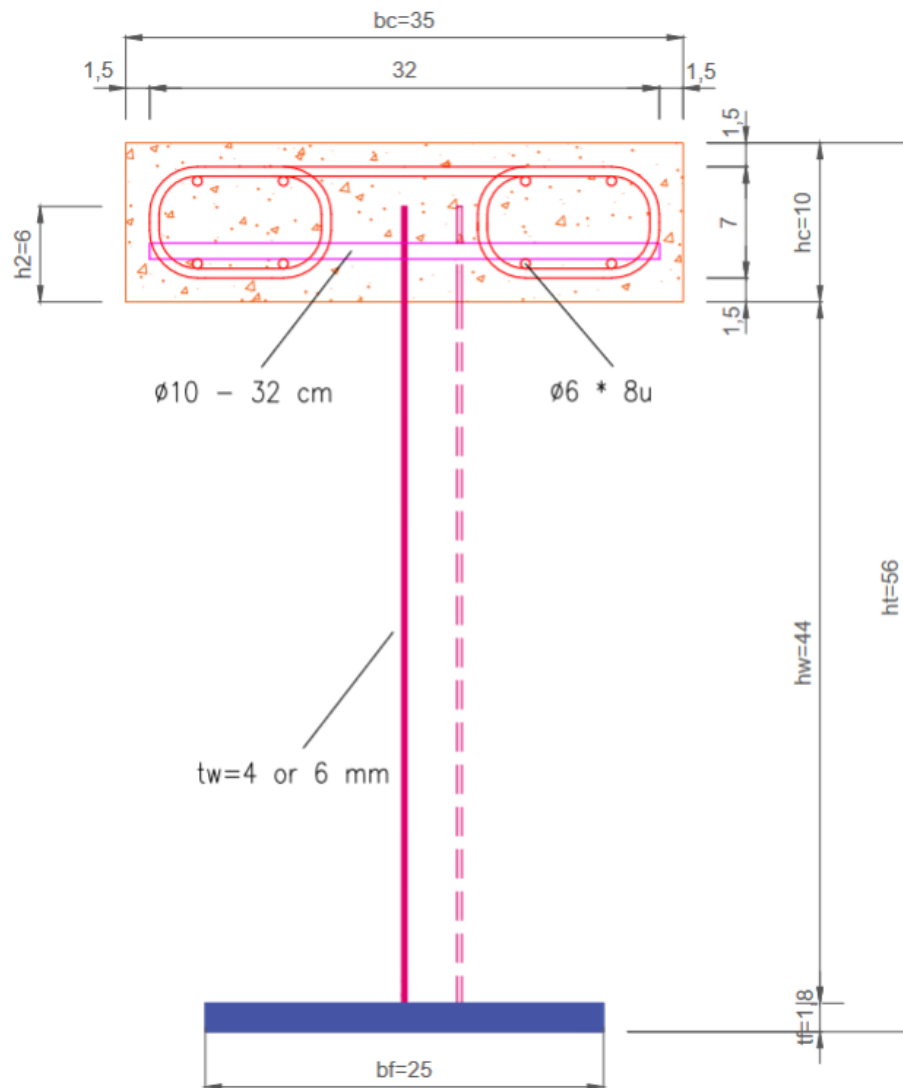


4.5. Figure: Reinforcement of the concrete flange

The stirrup spacing along the beams will have an impact on the shear behaviour of the beams. It is hard to evaluate the effect of the spacing on the final load, it will not be evaluated in this paper. However, the table below (Table 4.3.2.) summarizes the spacing near the load introduction in each girder.

4.3.1. Table: Spacing of the stirrups for each girder at the load introduction plate

Girder No.	Spacing [mm]
B-1	BH-2 (19 mm)
B-1-2	BH-1 (34 mm)
B-1-3	BH-1 (34 mm)
B-2-1	BH-2 (19 mm)
B-2-2	BH-1 (34 mm)
B-3	BH-1 (34 mm)
B-4	BH-1 (34 mm)
B-5	BH-1 (34 mm)

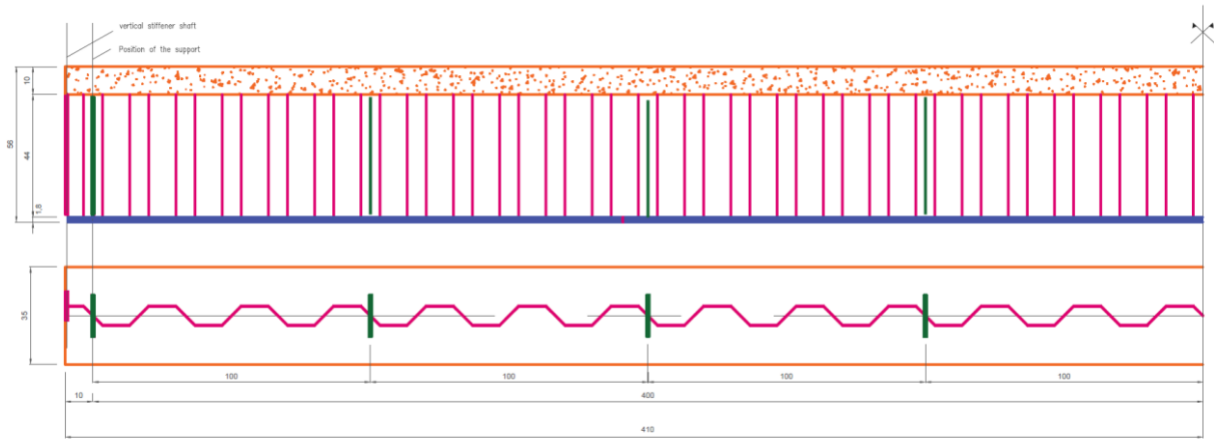


4.6. Figure: Cross section of the tested beams [cm]

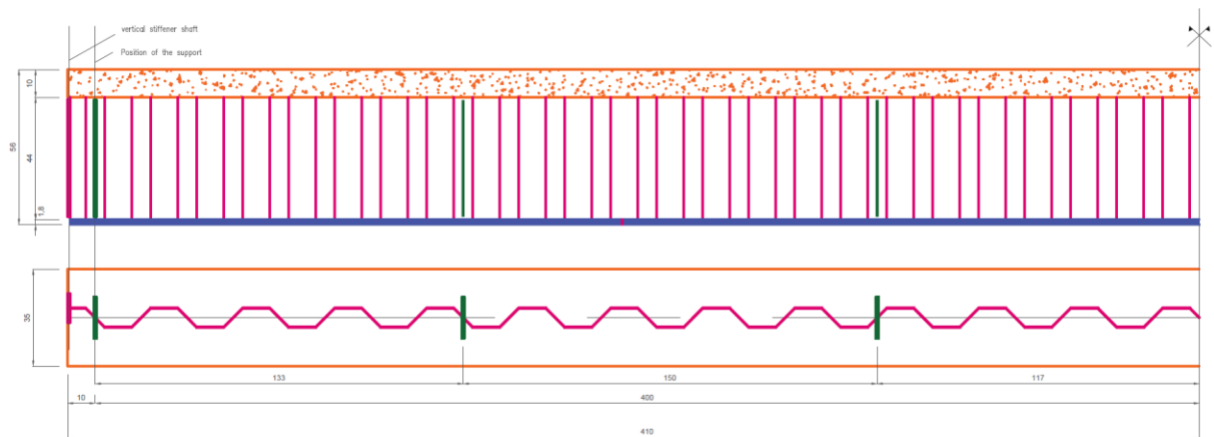
The concrete slab is 350 mm wide and 100 mm high. The concrete cover is 15 mm in each direction. The embedded height of the web in the concrete slab is 60 mm. The vertical distance between the centre of gravity of the longitudinal rebars is 40 mm. The corrugated web is 500 mm high, with 440 mm free depth. The steel flange is 250 mm wide and 18 mm thick. The cross-section is 560mm high. (Figure 4.3.6)

4.4. STIFFENERS

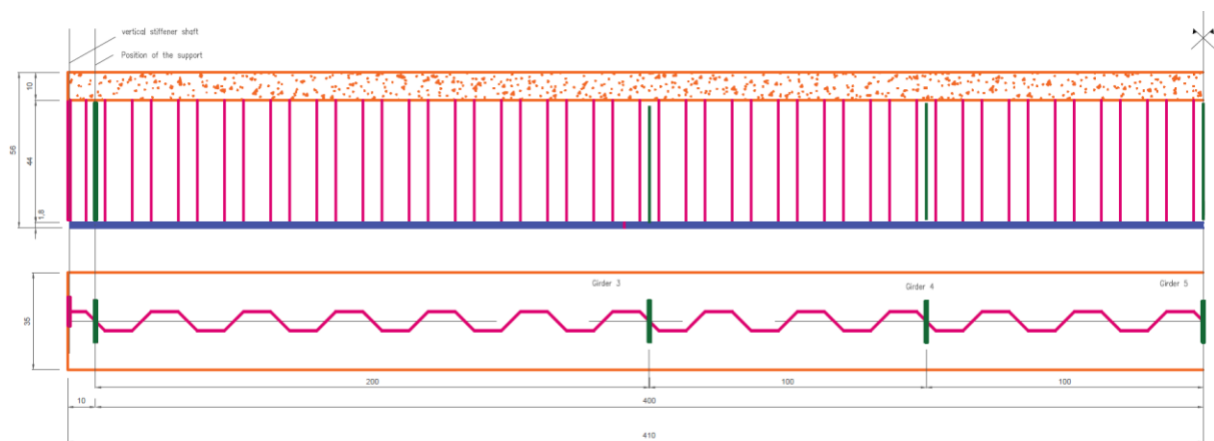
On the different tested beams, several stiffeners were positioned, on all the support and underneath each load introduction place. It creates panel between the stiffeners which act like a truss by creating a rigid frame all around the web. The stiffeners are 10 mm thick and are 125 mm width. Their positions are represented in Figure 4.7-4.8. The role of stiffeners is to prevent local type failure in the web at the load introduction places (supports and load positions).



4.4.1. Figure: Position of the stiffeners of the girder 1 (1-1, 1-2, 1-3)



4.4.2. Figure: Position of the stiffeners of the girder 2 (2-1, 2-2)



4.9. Figure: Position of the stiffeners of girders 3, 4 and 5

4.5. FAILURE MODES

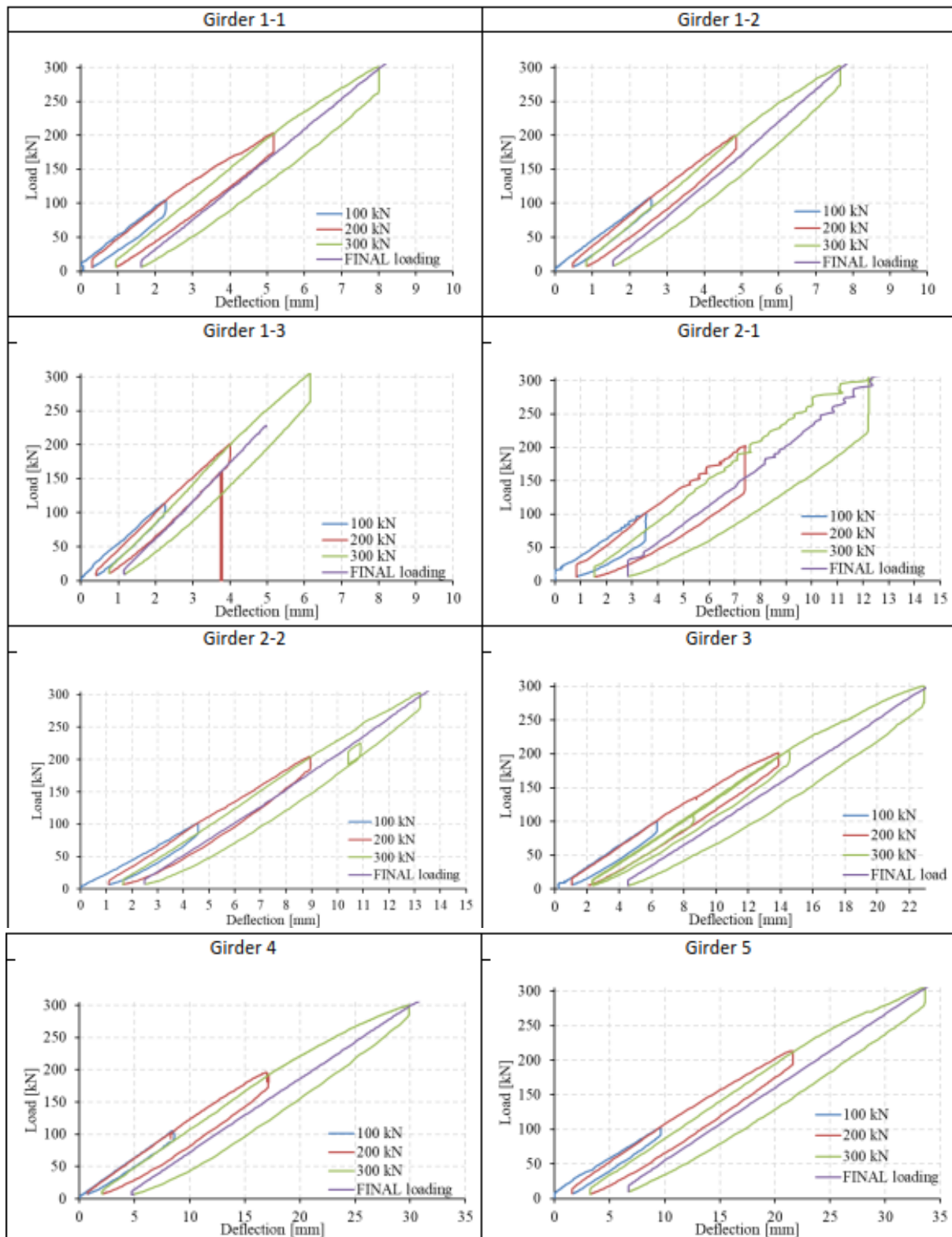
In the tested beams three different failure modes could be observed and identified:

1. **Shear type failure of the web** which results in **out-of-plane buckling of the web**. It can happen before reaching the full plasticity of the web which is often the case when the thickness is too small.
2. **Yielding of the steel flange** leading to a **column type failure of the concrete slab** in the end of the second phase. It is also observed on the force-strain diagram thanks to the strain gauges placed on the top of the steel flanges.
3. Patch loading failure near the support which results in **out-of-plane buckling of the web locally**. This failure was not supposed to happen. The bottom of the stiffeners was not properly welded to the steel flange.

4.6. INITIAL STIFFNESS

All the beams were tested according to the following procedure:

An incremental load is applied to the beam up to the value of 100 kN, then the beam is unloaded. This procedure is repeated two times with the value of 200 and 300 kN. After that the final loading is applied, the load is incremented until the failure of the beam happened.



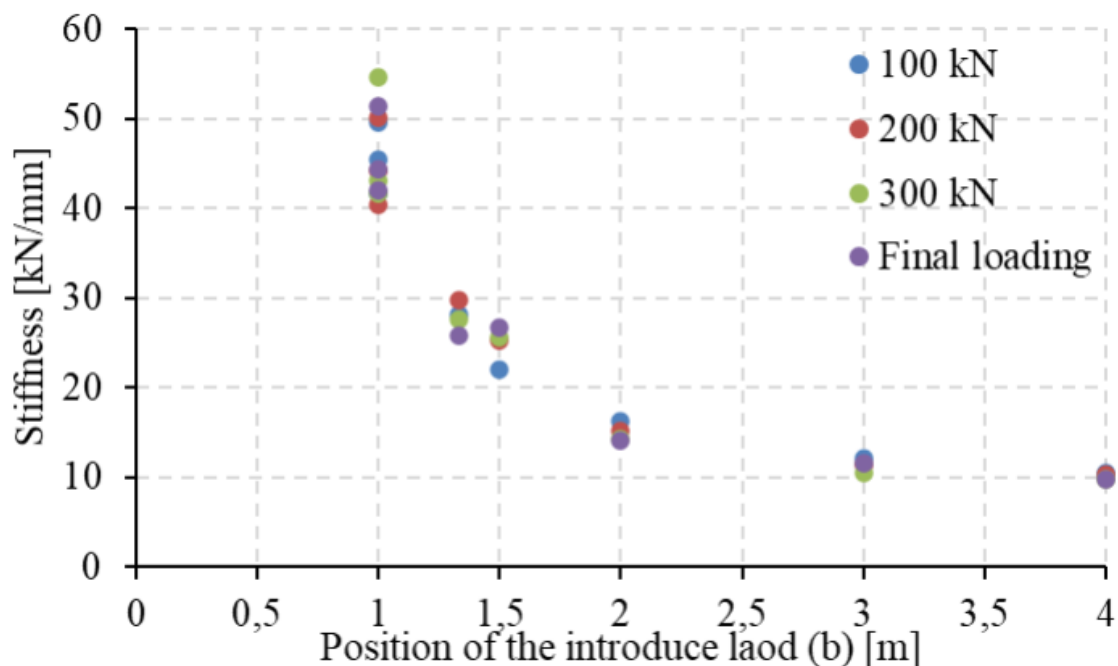
The stiffness of the cross-section can be assimilated to be proportional to the slope of force-displacement diagrams if the cross-section is in an elastic linear stage. The value of the slope (stiffness) during each stage of the loading procedure can be found in Table 4.6.1. After 3 loading phases, there is no significant decrease in the value. We could even say that the stiffness of each girder before and after the loading is pretty

much the same. This slope is also dependent on the position of the load introduction place. As the result showed, the deflection is smaller when the load introduction place is near the support which is logical as the bending moment value is smaller for the same load value. (Figure 4.10.).

4.6.2. Table: Stiffness of the cross-section in [kN/mm] during the loading cycle

Girder No.	100 kN	200 kN	300 kN	Final loading	Mean
B-1	45,53	40,42	41,64	42,06	42,41
B-1-2	41,66	44,10	43,19	44,37	43,33
B-1-3	49,59	50,09	54,59	51,46	51,43
B-2-1	28,14	29,81	27,65	25,81	27,85
B-2-2	21,96	25,20	25,56	26,63	24,84
B-3	16,24	15,24	14,35	14,09	14,98
B-4	12,14	11,47	10,53	11,49	11,41
B-5	10,46	10,29	9,83	9,84	10,10

In Figure 4.11., the stiffness of the cross-section follows a parable, it is a logic result because the displacement is a quadratic function which depends on the load introduction place.



4.6.2. Figure: Stiffness of the cross section depending on the position of the introduced load b [m]

After each loading cycle, a residual value for the load and the deflection is observed. To ensure that theses residuals values didn't affect the analysis of the behaviour of the

beams, they will be deducted from the test results. The values are available in Table 4.6.4.

4.6.3. Table: Residual deflection and load deduced of the ultimate load/displacement after the loading of 300 kN

Girder No.	Residual deflection [mm]	Residual load [kN]
B-1-1	1,62	6,25
B-1-2	1,58	7,96
B-1-3	1,17	9,68
B-2-1	2,86	6,55
B-2-2	2,49	7,31
B-3	4,56	6,29
B-4	4,93	7,03
B-5	6,75	10,15

4.7. STRUCTURAL BEHAVIOR AND STRENGTH

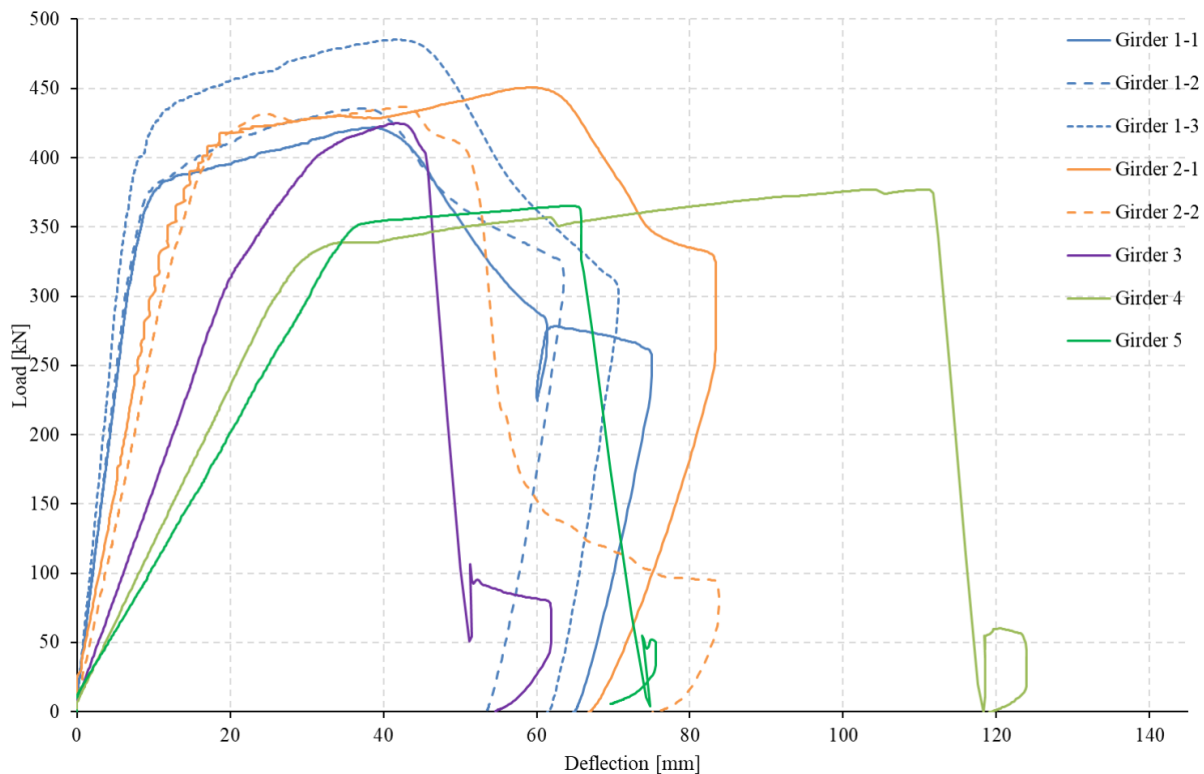
After the deduction of the table (Table 4.7.1.), the yield strength **F_y** and the ultimate load **F_u** can be extrapolate from the corrected force-displacement diagrams. We observed 3 different phases in the diagrams. (Figure 4.12.). The first one is elastic up to the point (**e**, **F_y**), the second one is plastic up to the point (**e_u**, **F_u**) and the last one is either post buckling phase or an unloading phase. The next two equations defined the relation between the first and the second phases.

$$F_y + \delta F = F_u \quad (29)$$

$$e + \delta e = e_u \quad (30)$$

4.7.1. Table: Experimental data of the tested beams

Girder No.	F _y [kN]	δF [kN]	F _u [kN]	e [mm]	δe [mm]	e _u [mm]	F _y / e	δF / δe
B-1	352.91	75.20	428.11	8.5	30.16	38.66	41.52	2.29
B-1-2	363.2	80.27	443.47	8.5	28.28	36.78	42.73	2.56
B-1-3	400.94	93.82	494.76	8.5	32.81	41.31	47.17	2.56
B-2-1	391.45	65.55	457.00	16	43.07	59.07	24.47	1.37
B-2-2	386.75	57.25	444.00	16	26.94	42.94	24.17	1.86
B-3	329.81	100.15	429.96	29	81.37	110.37	11.37	0.58
B-4	348.19	35.71	383.90	36	28.11	64.11	9.67	0.61
B-5	352.91	19.09	372.00	8.5	30.16	38.66	41.52	2.29



4.7.1. Figure: Force displacement diagrams of the tested beams (modified)

The mechanical background and the difference between the graphs will be described in Section 6.

5. FE MODEL DEVELOPMENT AND VALIDATION

The developed finite element model (geometric, mesh, load and boundary conditions) is introduced in this section. The finite element model is developed in ANSYS 24.0 environment. The experimental background, the analysis type, convergence study, applied imperfections and the validation of the numerical models are also presented in this Section.

The aim of the advanced FE model development is to give a preliminary estimate for the ultimate load carrying capacity and to investigate the structural behavior of the experimental test specimens being under production, therefore in this Section the previously designed steel-concrete composite girders with trapezoidally corrugated web are modelled.

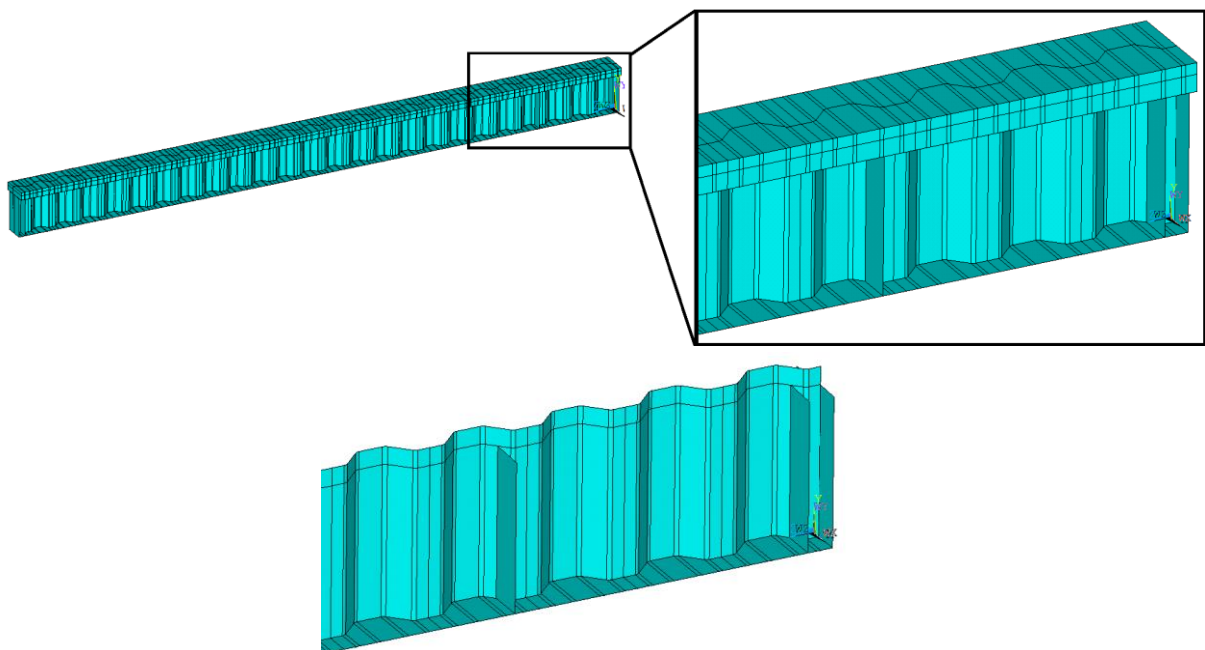
5.1. VALIDATION BACKGROUND

The behavior of steel-concrete composite girders with corrugated web under the combined loading situation is in the scope of investigation. The preliminary validation

of the FE numerical model with respect to the pure ultimate load carrying capacities, namely the bending moment and shear capacities have high importance in this research program. The validation is performed by comparing the FE results with the hand calculated ultimate load capacities. For the sake of simplicity the interaction behavior is ignored in the hand calculations. The evaluation of the comparison can be seen in Section 7.

5.2. GEOMETRIC MODEL

The geometrical model with the defined surfaces can be seen in Figure 5.1. It is shown that the full geometrical model is divided into areas bounded by edges. Figure 5.2. shows the areas of the geometrical model without the concrete slab as well.



5.2.1. Figure: Geometric model of steel-concrete composite girder with corrugated web

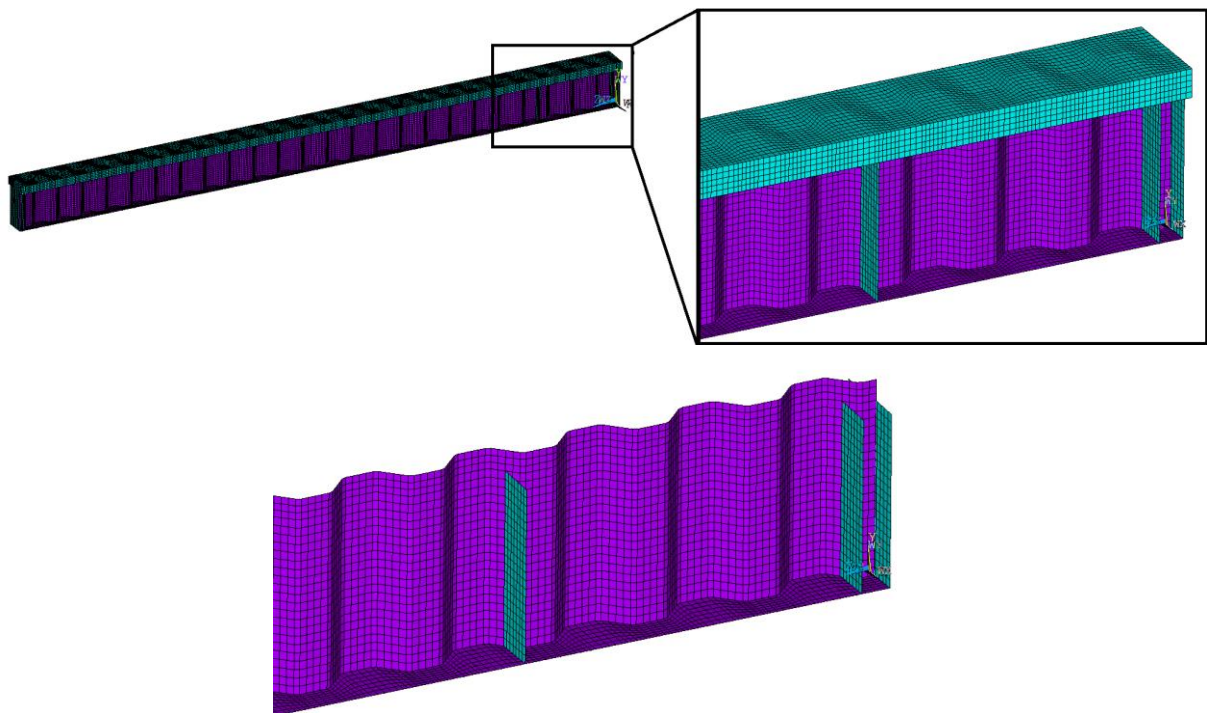
5.3. MATERIAL MODEL

In the FE model two material models are used; one for the steel and one for the concrete material. For the steel material the character of the applied material model is 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's modulus equal to 210000 MPa up to the assumed mean yield stress ($f_y=280\text{MPa}$). The yield plateau is modelled up to 1% strains. Thereafter and until it reaches the assumed mean ultimate stress ($f_u=460\text{MPa}$), 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.

For the concrete material the character of the applied material model is a linear elastic – softening plastic material model. For the sake of simplicity the cracking and crushing are not considered in the analysis. This multilinear material model has in the plastic domain a kinematic hardening behaviour. The material is assumed to behave linearly elastic and to obey Hook's law with a Young' modulus equal to 20000 MPa up to the assumed mean compressive stress ($f_c=50\text{MPa}$). The yield plateau is also modelled. Thereafter the material is assumed to behave linear softening.

5.4. FINITE ELEMENT MESH

Figure 2 presents the finite element mesh. In the finite element model the SHELL181 and SOLID65 elements are applied for the steel and concrete parts, respectively, which can be seen in Figure 5.3. In the model mapped mesh is applied for each part. The connection between the steel corrugated web and the concrete upper slab is rigid by having the same nodes for both parts, the embedded steel web and concrete slab at the connection interface.



5.4.1. Figure: Finite element mesh

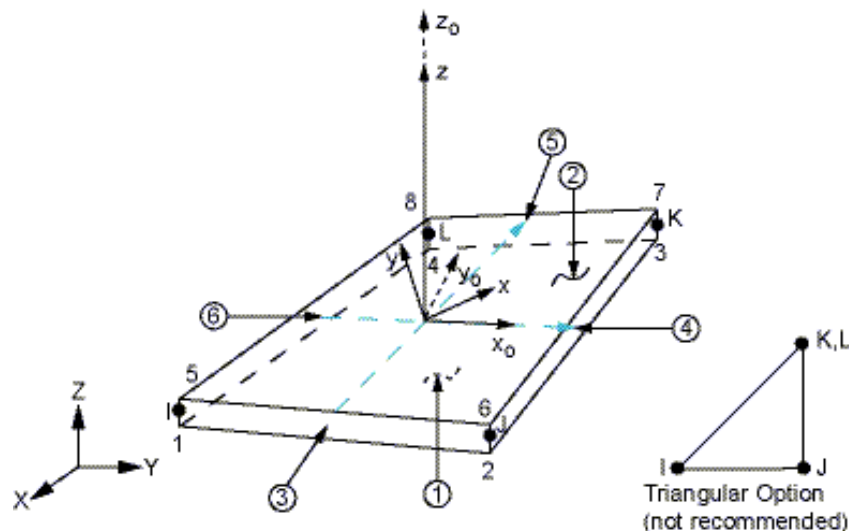
The attributes of the SHELL181 finite element are the followings:

- 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,

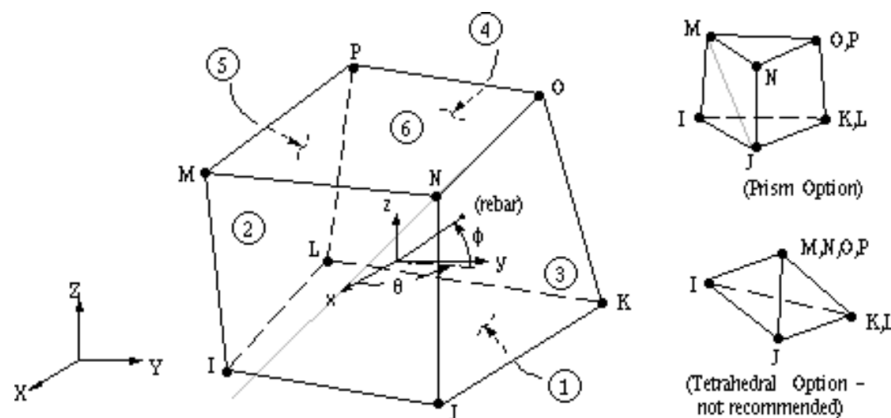
- it can be applicable for stability analysis (critical stress level determination),
- applicable for linear or nonlinear analyses.

The attributes of the SOLID65 finite element are the followings:

- eight-node solid element,
- three degrees of freedom at each node, translations in the nodal x, y and z direction,
- applicable for linear or nonlinear analyses.



5.4. Figure: Shell 181 element

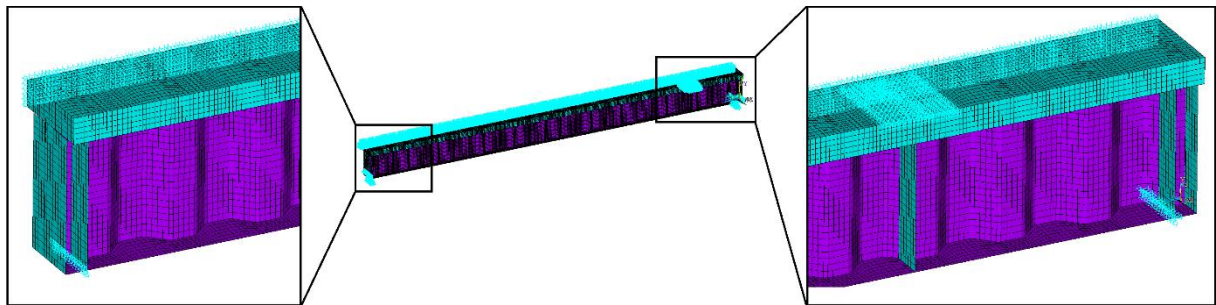


5.5. Figure: SOLID65 finite element

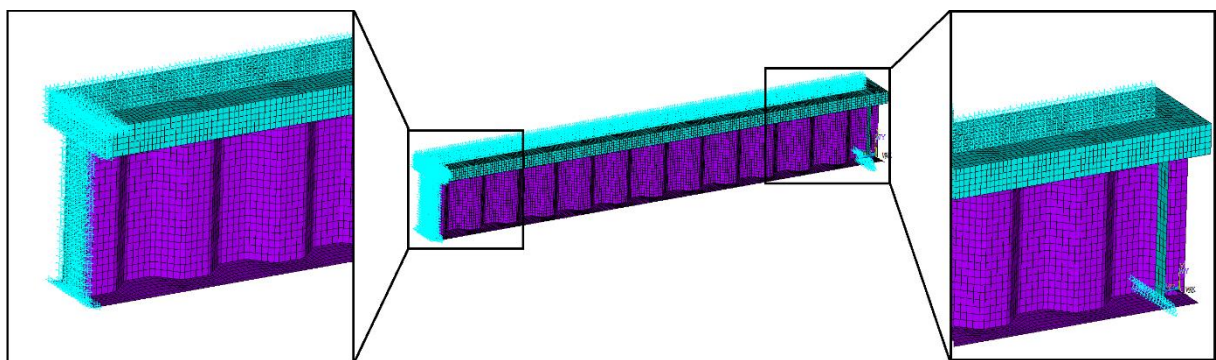
5.5. LOADS AND BOUNDARY CONDITIONS

The different boundary and loading conditions of the finite element model specialized for the investigation of each load case are presented. Fig. 5.6. shows the case when the whole girder is modelled; the girder is simply supported at the two ends and supported laterally along the whole girder to ignore lateral torsional buckling failure. For the load, displacement magnitude is prescribed at the load introduction

places. Fig. 5.7. shows the case when the half of the girder is modelled; the right side of the girder is simply supported while on the left side symmetric condition is prescribed.



5.5.1. Figure: Boundary conditions if the full girder is modelled



5.5.2. Figure: Boundary conditions if the half of the girder is modelled

5.6. ANALYSIS TYPE

The analyzed numerical FE model is developed using the finite element software ANSYS 24.0. The numerical modelling is based on the combination of a shell and solid 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. In the analysis displacement control is used.

5.7. APPLIED IMPERFECTIONS

The aim of the current research is to determine the resistances of the pure and combined loading situations under bending and shear force of trapezoidal corrugated web girders. The shear failure is expectedly 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) 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.

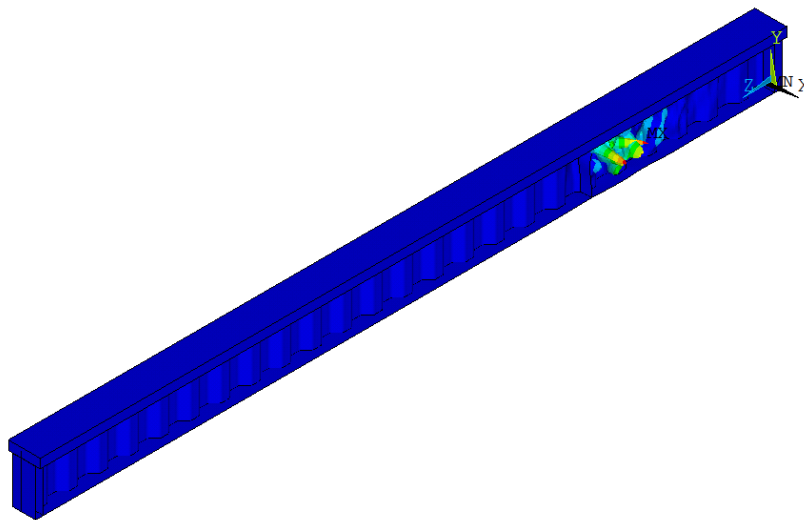
In most of the cases the initial imperfections of the specimens are not known for the validation of the developed numerical model. The measuring procedure is very time-consuming and does not contain the residual stresses. Using periodic functions as equivalent geometric imperfections can be a possible 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 geometric 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 Figure 5.8.

There is no recommendation in the EN1993-1-5 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. [19] 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. [24]. Another magnitude for the imperfection amplitude was proposed by Yi et al. [14], Nie et al. [50] and Jäger et al. [33]. 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.



5.7.1. Figure: Shear buckling eigenmode shape

5.8. CONVERGENCE STUDY

Mesh convergency study is executed to ensure the model reliability. According to Moon et al. [14] one web fold should be modelled by minimum of 4-6 elements along the fold length to determine the capacity of the girder with adequate accuracy. Own investigation confirmed this statement. During my research the most efficient average mesh size proved to be 50 mm. So my following results are calculated with this mesh size. As the lower picture shows

The best fit curved used of a young modulus of concrete of $E_{cm}=20$ GPa and the young modulus of steel of $E_s=200$ GPa.

6. NUMERICAL STUDY

This section presents the FE analysis results regarding each test specimens. And the comparison of these results with the full-scale experiments.

Within the scope of this thesis, there was not enough time to build a sufficiently accurate numerical model of all the experiments. Their refinement is planned for the calculation of the necessary parametric numerical tests. My primary objective in the model constructed within the limits of the thesis is to trace the initial stiffness and to match the load bearing levels. A more realistic tracking of the ductile section could have been achieved by calibrating the concrete model used, which was also not possible within the boundaries of the thesis.

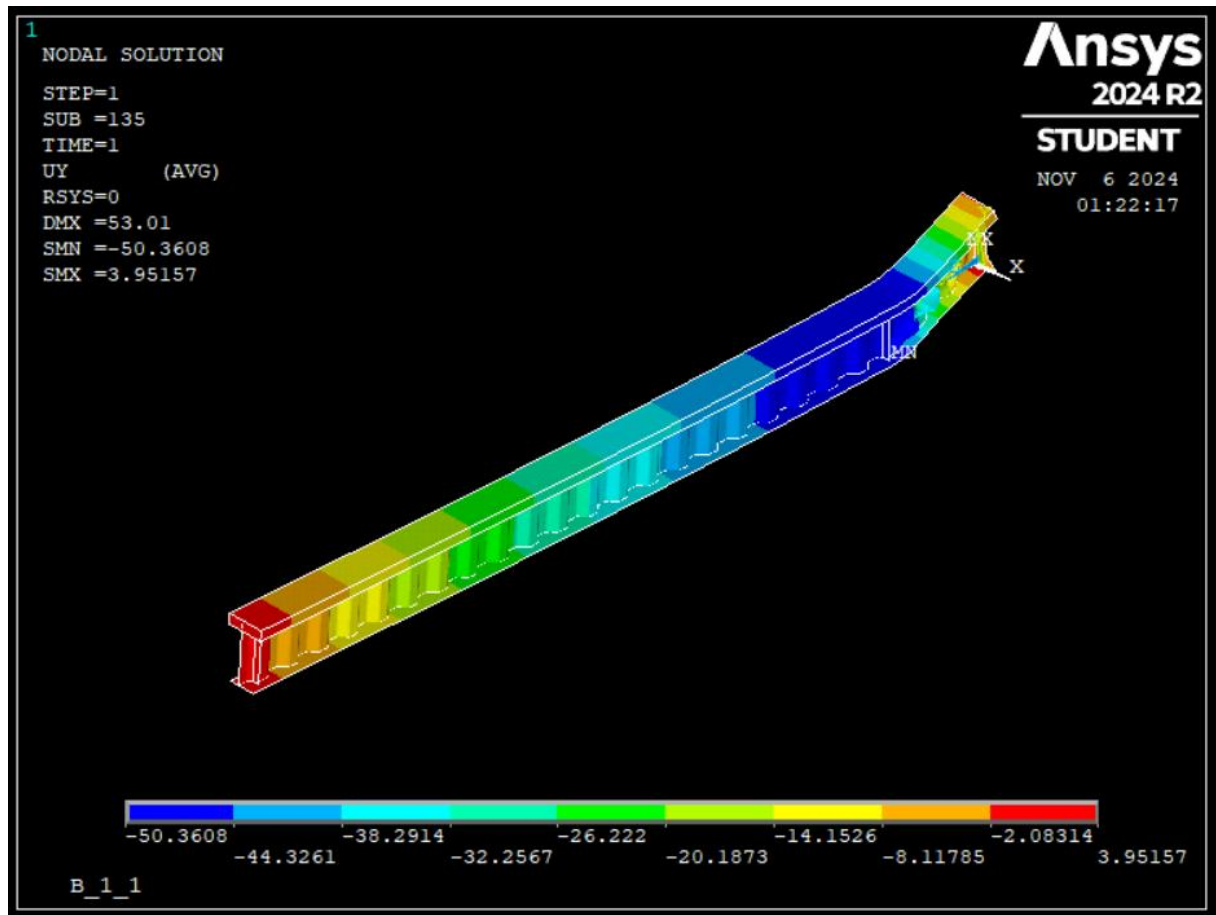
Therefore, the numerical model results are presented only for B-1-1,2,3, B-4, B-5. Most interestingly, I would like to point out that the precise modelling was not performed for the beams B-2-1,2, B-3.

6.1. LOAD CASE #1 (B-1-1)

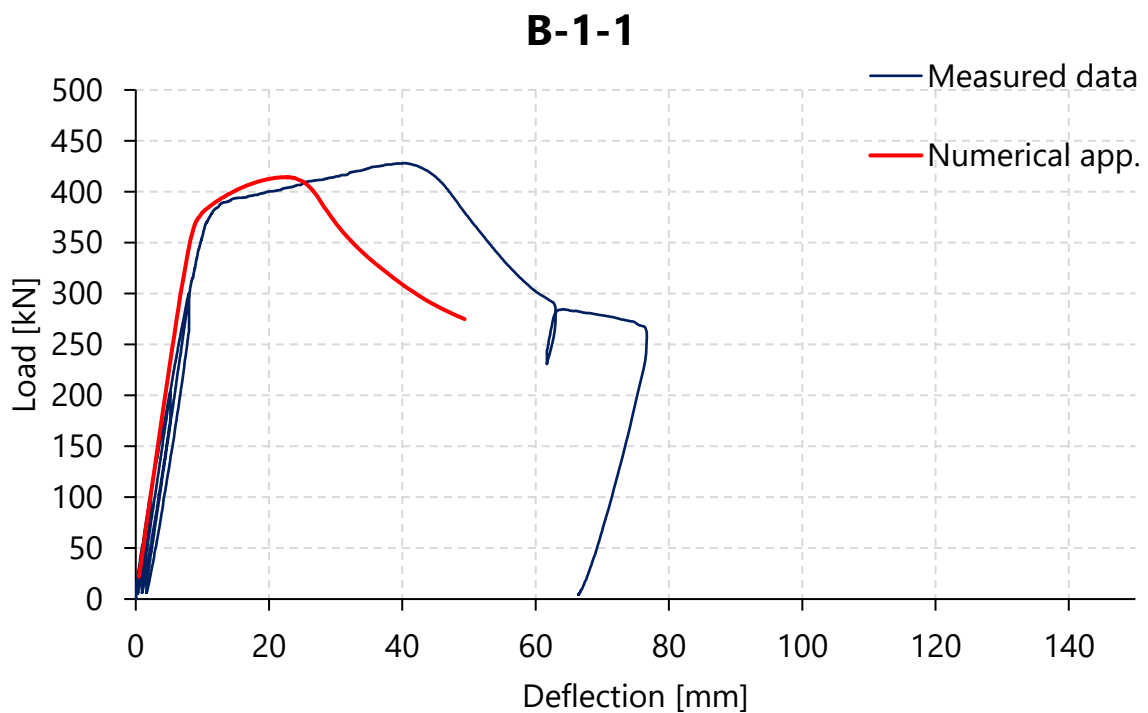
B-1-1 is an 8 meters beam, tested to evaluate the shear resistance since the introduce load is 1 meter from the support. Its shows a shear failure type buckling of the steel web. There are few degradations of the concrete slab near the introduce load. At the nearest support, the concrete slab peels off from the steel corrugated web. Small cracks are observed on the concrete slab. The web thickness is 4 mm. The strain gauges placed on the steel flange show that the flange remain moslty in an elatic stage. Small residual deflection remains after the beam was unloaded.



6.1. Figure: B-1-1 Initial setup and the deformed CSW



6.2. Figure: Vertical displacement of B-1-1 according to my FE approximation



6.3. Figure: Load-Deflection graph below the load of B-1-1

The beam used in the experiment behaves elastically up to 375 kN, followed by a plastic hardening zone, which gives a total load bearing capacity of 440 kN. The former results were calculated by FE to be 370 kN and 414 kN. This is a difference of 1% and 6% respectively, which I found acceptable.

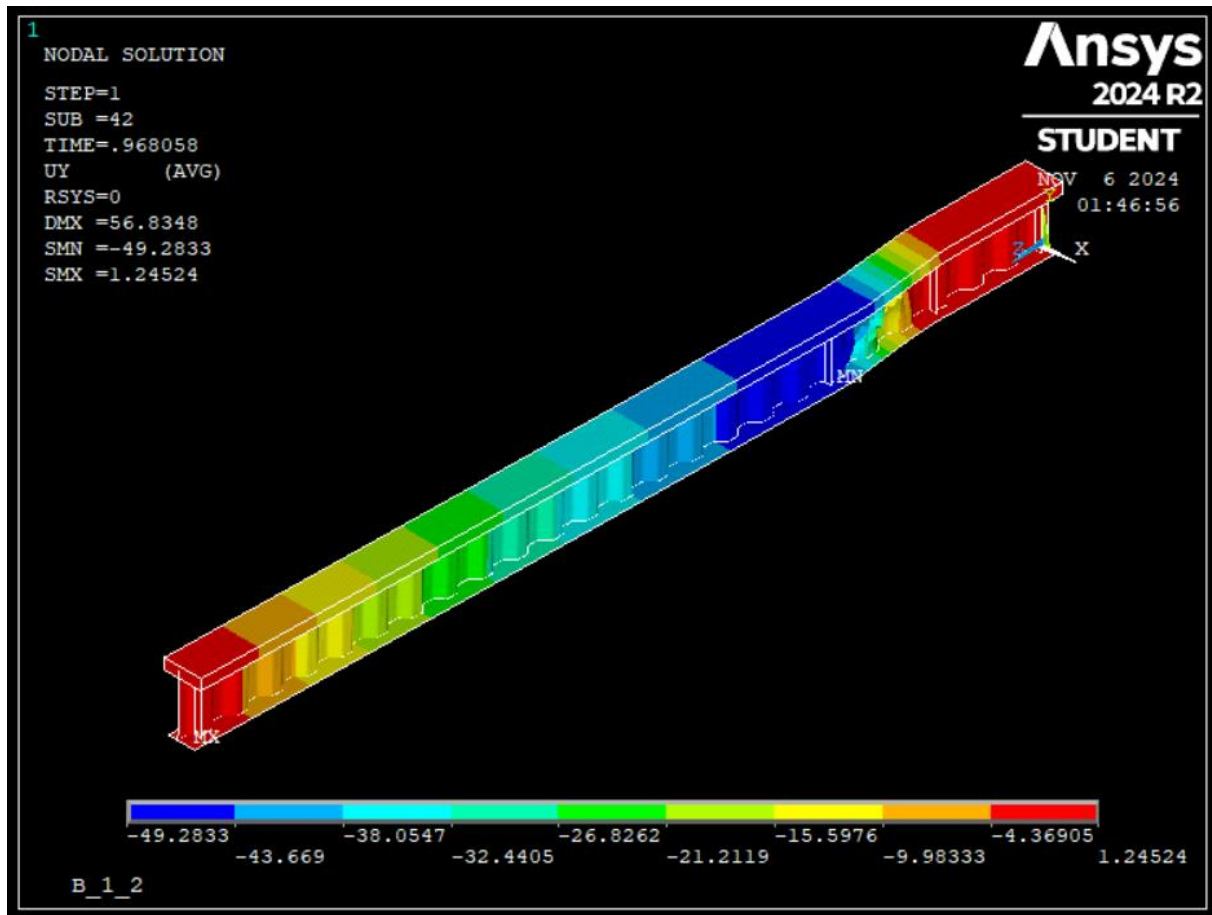
Shear failure of the beam occurred with buckling of the web after ~10mm of deflection and this was still maintained by the reinforced concrete top flange for a further ~60mm. Until the longitudinal rebars buckled, crushing the concrete core and reaching the maximum load capacity of the beam.

6.2. LOAD CASE #2 (B-1-2)

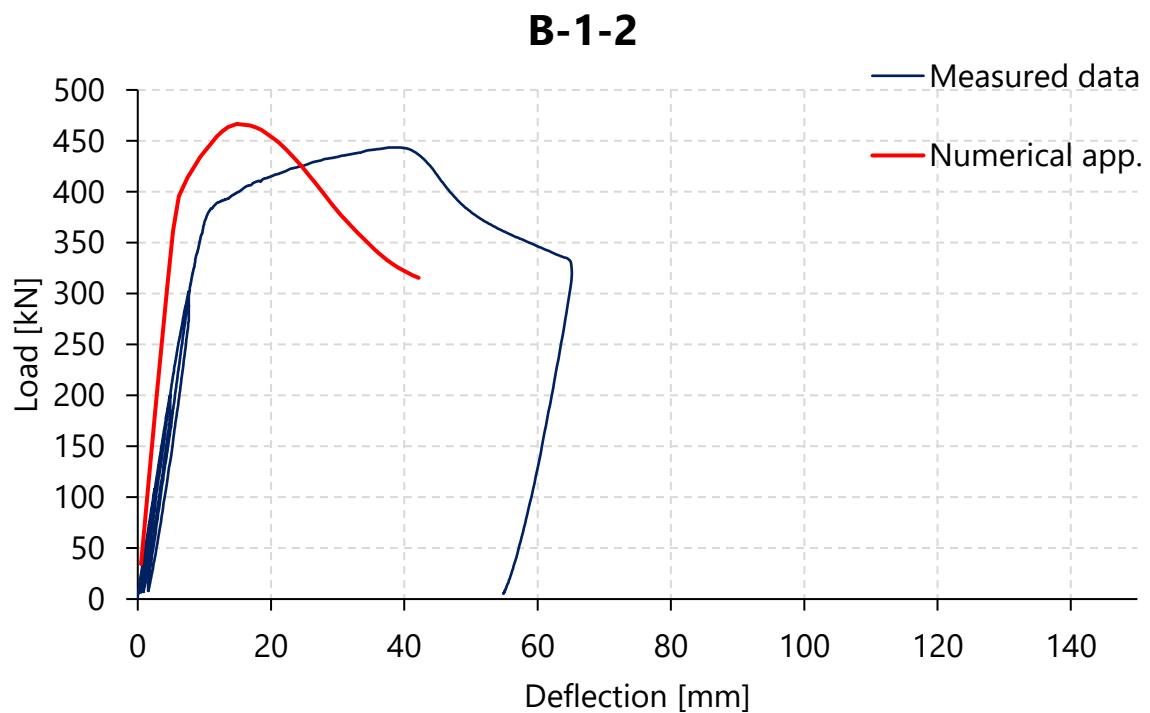
B-1-2 is the deformed girder 1-1, but studied on a 7 meters range, tested on the none-yielded part of the beam. The introduce load is 1 meter from the support. The goal of the test was also to evaluate the shear behaviour of this beam. Its shows a shear failure, on the support. It's called a path failure, meaning that the load isn't well transmitted to the support. As we saw on picture, the rigid stiffener isn't welded on the flange. It's means that the corrugated web yield, which wouldn't never happen if the stiffener was welded to the flange. In that case, the panel (the web between the two stiffeners) would have act like a truss and we will have shear type failure. Therefore, the initial stiffness will be considered, and the ultimate load will be detached from the other shear type failure. The web thickness is 4 mm.



6.4. Figure: B-1-2 Patch loading failure at the support



6.5. Figure: Vertical displacement of B-1-2 according to my FE approximation



6.6. Figure: Load-Deflection graph below the load of B-1-2

The beam used in the experiment behaves elastically up to 387 kN, followed by a plastic hardening zone, which gives a total load bearing capacity of 455 kN. The former results were calculated by FE to be 395 kN and 467 kN. This is a difference of 2% and 2% respectively, which I found acceptable.

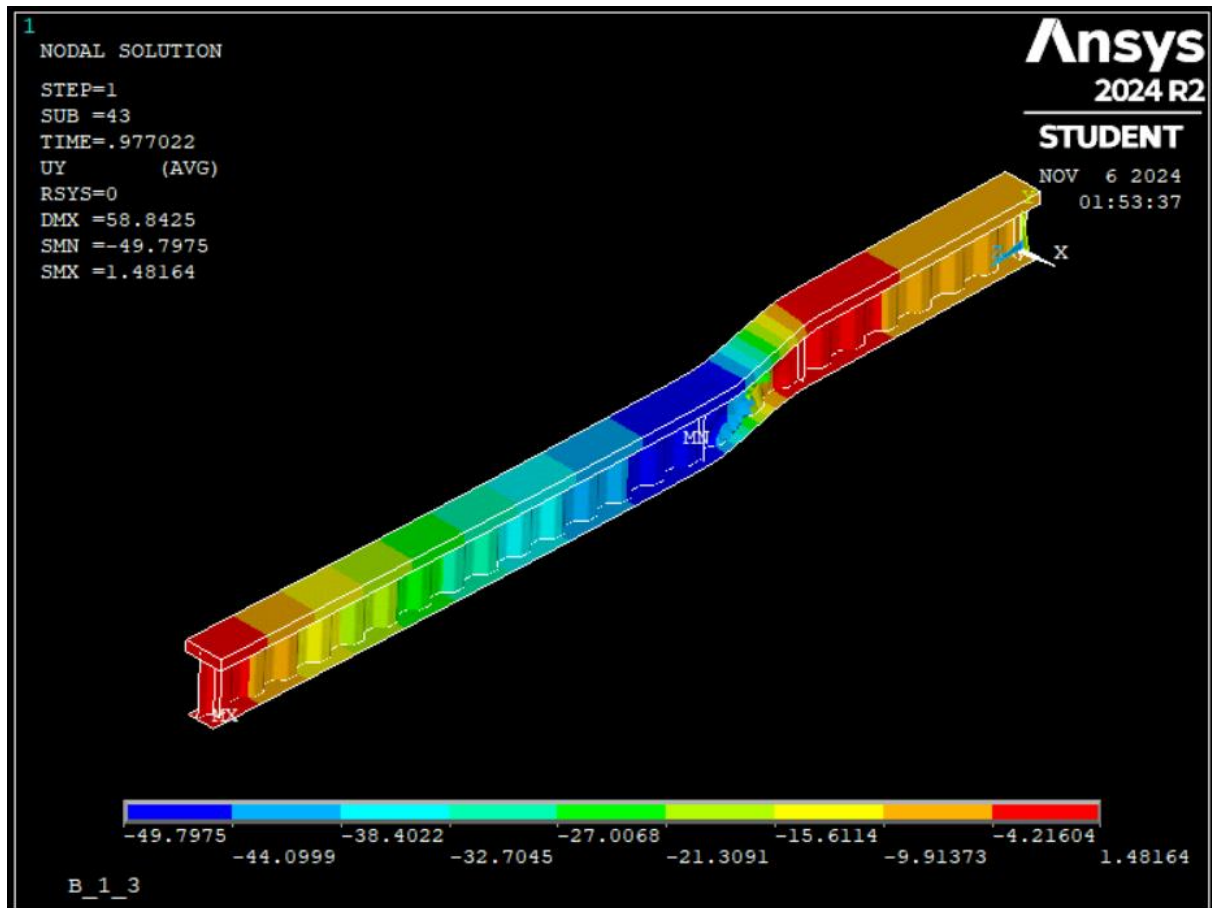
Shear failure of the beam occurred with buckling of the web after ~8.5mm of deflection and this was still maintained by the reinforced concrete top chord for a further ~55mm. When the longitudinal rebars buckled, crushing the concrete core and reaching the maximum load capacity of the beam.

6.3. LOAD CASE #3 (B-1-3)

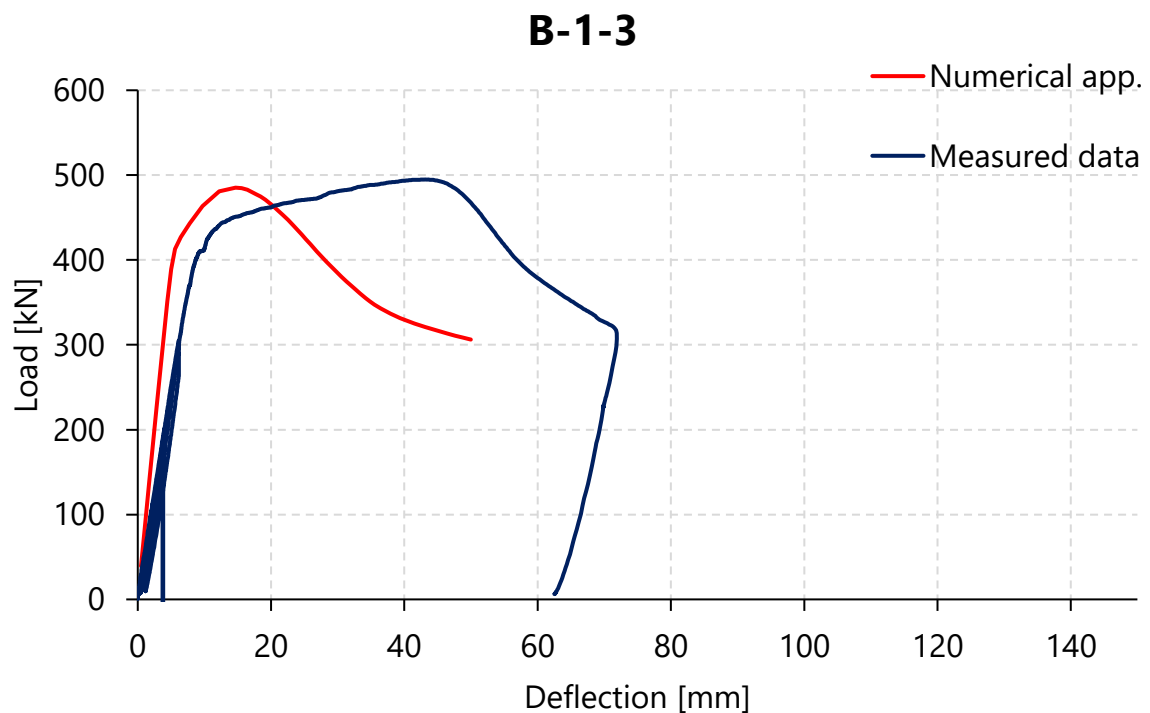
B-1-3 is the deformed girder 1-2, but studied on a 6 meters range, tested on the none-yielded part of the beam. The introduce load is 1 meter from the support. The goal of the test was also to evaluate the shear behaviour of this beam. The web tickness is 4 mm.



6.7. Figure: B-1-3 initial setup and deformed CSW



6.8. Figure: Vertical displacement of B-1-3 according to my FE approximation



6.9. Figure: Load-Deflection graph below the load of B-1-3

The beam used in the experiment behaves elastically up to 409 kN, followed by a plastic hardening zone, which gives a total load bearing capacity of 507 kN. The former results were calculated by FE to be 413 kN and 485 kN. This is a difference of 1% and 4% respectively, which I found acceptable.

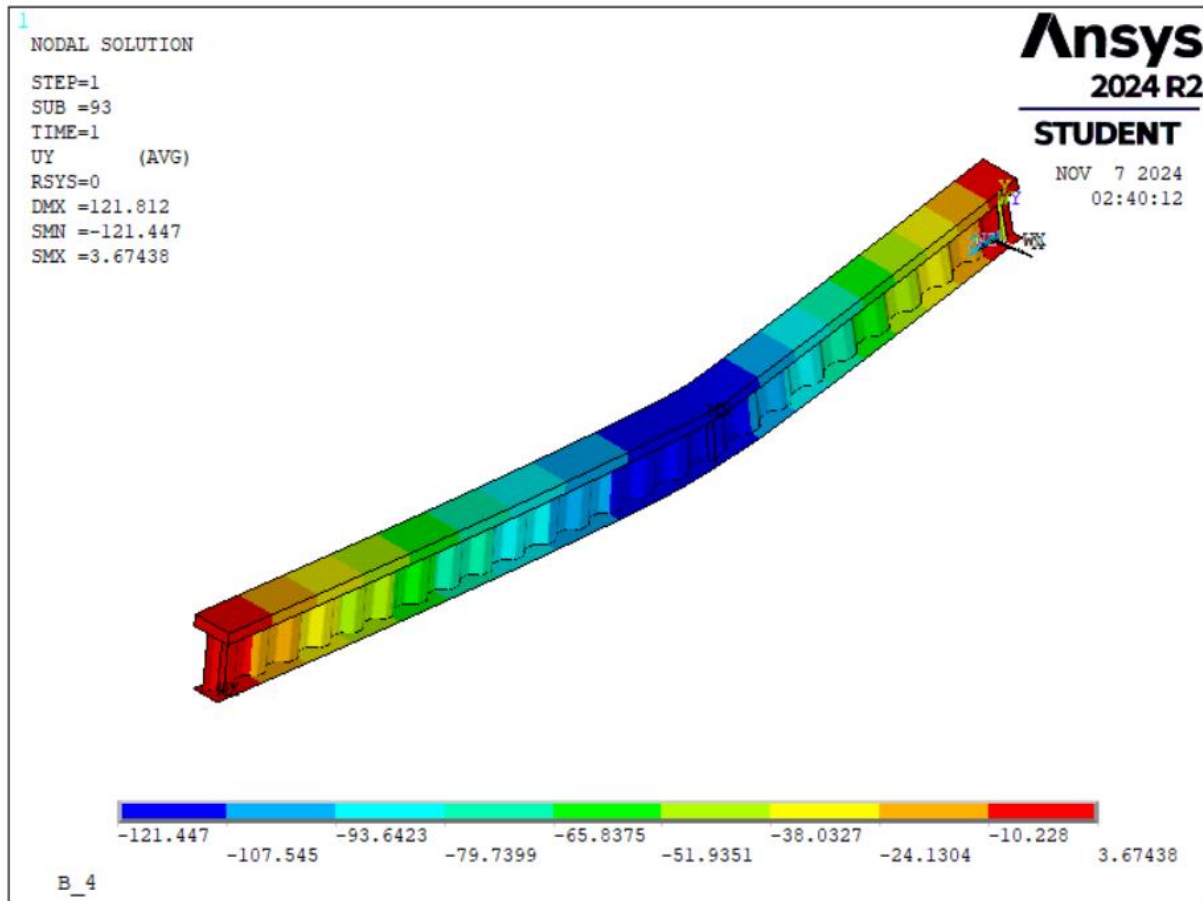
Shear failure of the beam occurred with buckling of the web after ~8.5mm of deflection and this was still maintained by the reinforced concrete top chord for a further ~45mm. When the longitudinal rebars buckled, crushing the concrete core and reaching the maximum load capacity of the beam.

6.4. LOAD CASE #7 (B-4)

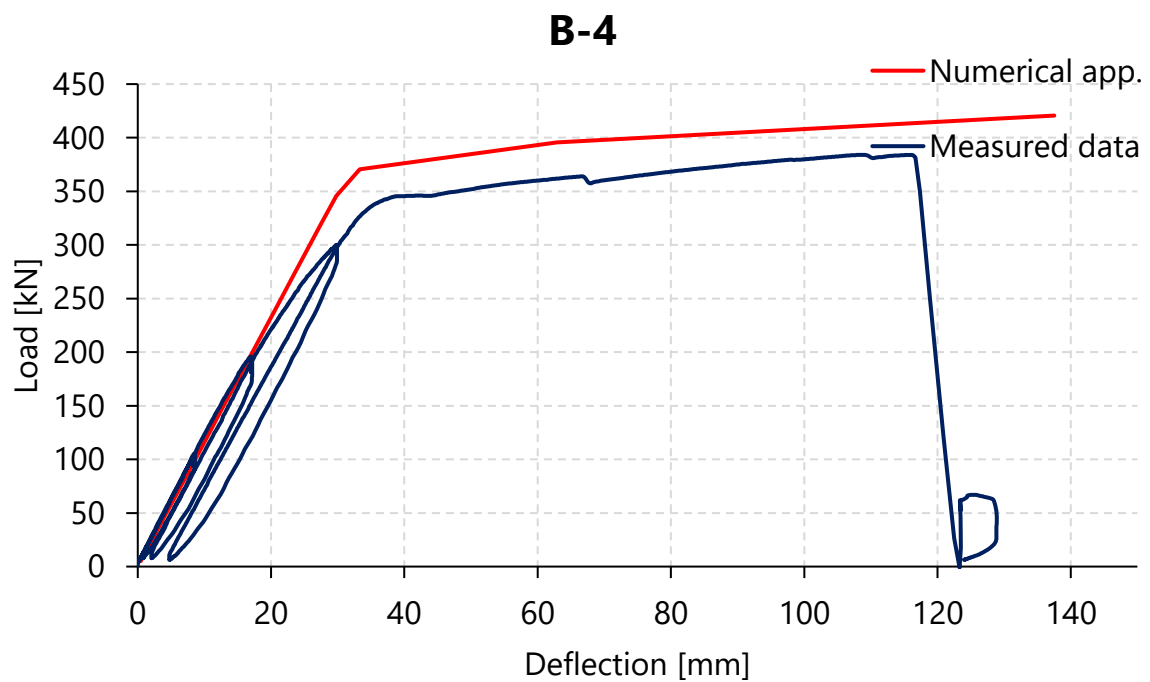
B-4 is an 8 meters beam, tested to evaluate the shear impact since the introduce load is 3 meter from the support. Its shows a sudden column type failure of the concrete slab. There are no deformation of the steel web. The web thickness is 4 mm. The strain gauges placed on the steel flange show the yielding of the element. This yielding creates a loss of stiffness leading the beam to enter a second phase of large displacement. Moreover, the gauges reach their ultimate nominal value of 6000 $\mu\text{m/m}$, meaning that the deformation of the flange after its yielding is quite large.



6.10. Figure: B-4 undeformed shape and failure mode



6.11. Figure: Vertical displacement of B-4 according to my FE approximation



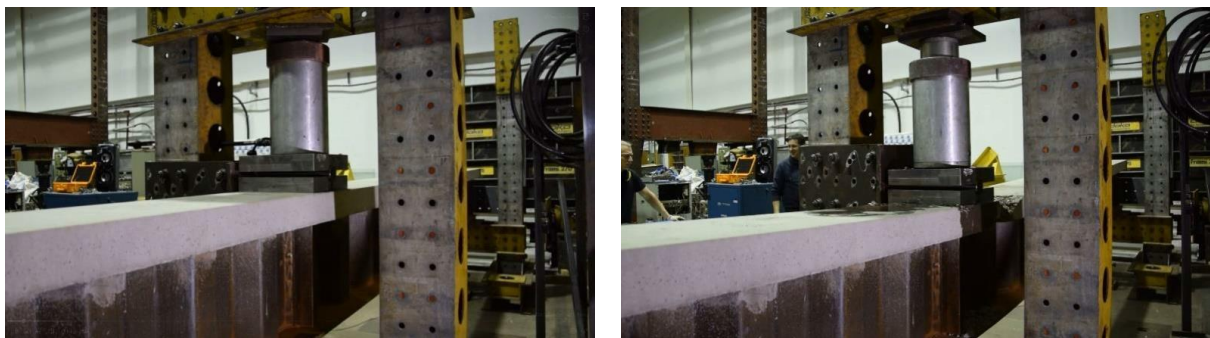
6.12. Figure: Load-Deflection graph below the load of B-4

The beam used in the experiment behaves elastically up to 328 kN, followed by a plastic hardening zone, which gives a total load bearing capacity of 396 kN. The former results were calculated by FE to be 365 kN and 453 kN. This is a difference of 11% and 14% respectively, which I found acceptable.

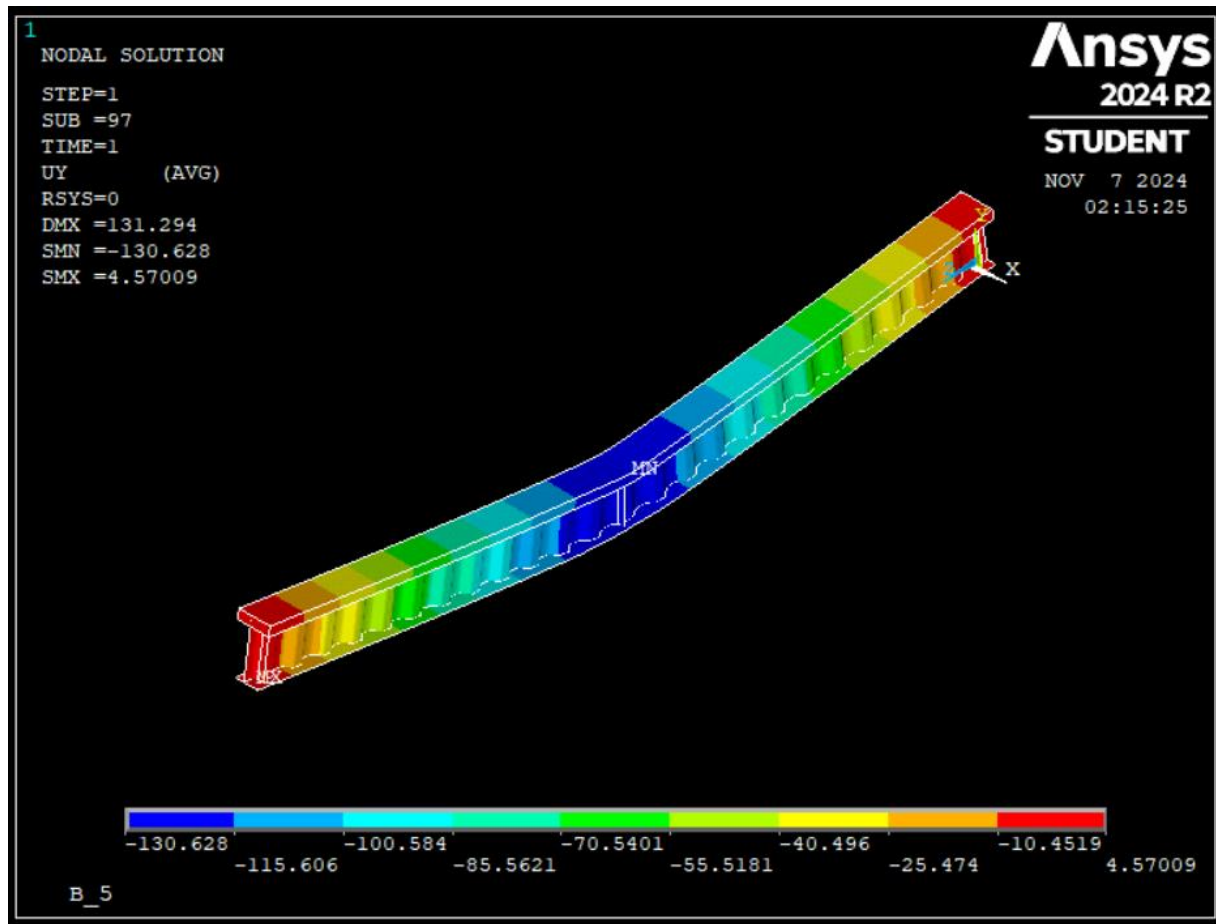
Shear failure of the beam occurred with buckling of the web after ~36mm of deflection and this was still maintained by the reinforced concrete top chord for a further ~120mm. When the longitudinal rebars buckled, crushing the concrete core and reaching the maximum load capacity of the beam.

6.5. LOAD CASE #8 (B-5)

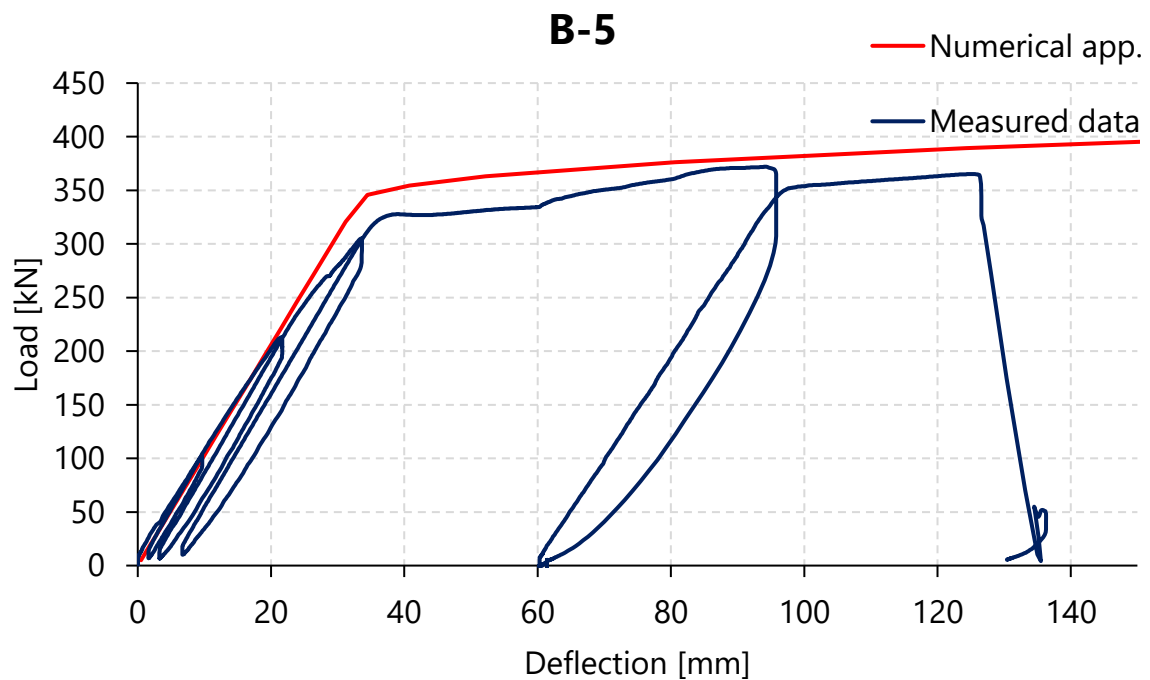
B-5 is an 8 meters beam, tested to evaluate the shear impact since the introduce load is 3 meter from the support. Its shows a sudden column type failure of the concrete slab. There are no deformation of the steel web. The web thickness is 4 mm. In the loading procedure, the hydraulic press leaked during the elastic stage. That is why there is two loading cycle on Fig. 6.13. The strain gauges placed on the steel flange show the yielding of the element. This yielding creates a loss of stiffness leading the beam to enter a second phase of large displacement. Moreover, the gauges reach their ultimate nominal value of 6000 $\mu\text{m/m}$, meaning that the deformation of the flange after its yielding is quite large.



6.14. Figure: B-5 undeformed shape and failure mode



6.15. Figure: Vertical displacement of B-5 according to my FE approximation



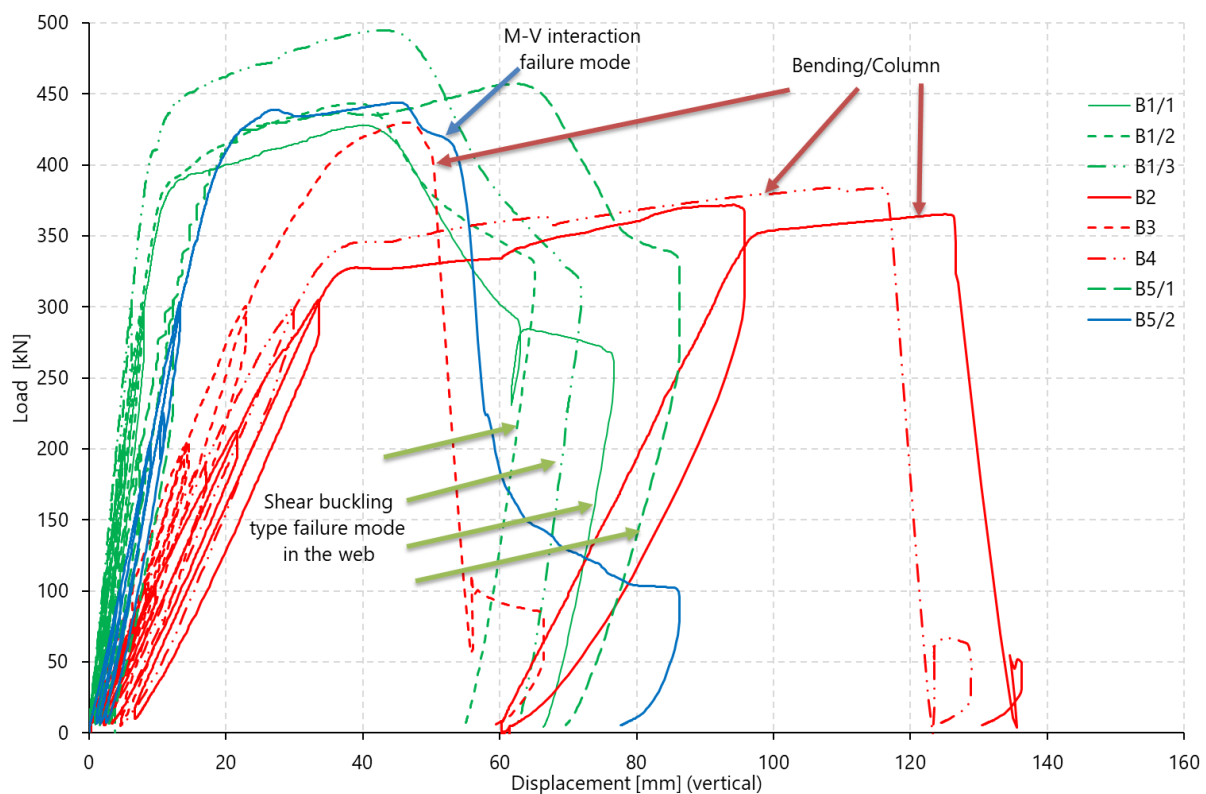
6.16. Figure: Load-Deflection graph below the load of B-5

The beam used in the experiment behaves elastically up to 324 kN, followed by a plastic hardening zone, which gives a total load bearing capacity of 384 kN. The former results were calculated by FE to be 341 kN and 390 kN. This is a difference of 5% and 2% respectively, which I found acceptable.

Shear failure of the beam occurred with buckling of the web after ~36mm of deflection and this was still maintained by the reinforced concrete top chord for a further ~130mm. When the longitudinal rebars buckled, crushing the concrete core and reaching the maximum load capacity of the beam.

6.6. CONCLUSION

In B-1 and B-2-1 there was shear failure in the web and patch loading type failure of the web around the support. The B-3, B-4, B-5 had bending type failure, the reinforced concrete top flange crushed with the buckling of rebars.



6.17. Figure: Measured Displacement-Load diagrams (failure modes indicated)

The table below summarises the load carrying capacity of the beams and the corresponding failure mode:

6.6.1. Table: Failure types and load capacity of each girder

Girder No.	Load [kN]	Failure	Load [kN]
B-1-1	352.91	shear buckling of the web	428.11

B-1-2	363.20	patch loading failure of the web around the support	443.47
B-1-3	400.94	shear buckling of the web	494.76
B-2-1	391.45	shear buckling of the web	457.00
B-2-2	386.75	Column type failure of the concrete flange	444.00
B-3	329.81	Column type failure of the concrete flange	429.96
B-4	348.19	Column type failure of the concrete flange	383.90
B-5	352.91	Column type failure of the concrete flange	372.00

The table below shows the deviations of the presented FE models from the measured results and their accuracy:

6.6.2. Table: Comparison of FE results and measured load capacity

Girder No.

End of initial stiffness

Ultimate failure

	Measured [kN]	FE model [kN]	Accuracy [%]	Measured [kN]	FE model [kN]	Accuracy [%]
B-1-1	375	370	1%	440	414	6%
B-1-2	387	395	2%	455	467	2%
B-1-3	409	413	1%	507	485	4%
<i>B-2-1</i>	<i>389</i>	<i>Nan</i>	<i>-</i>	<i>469</i>	<i>Nan</i>	<i>-</i>
<i>B-2-2</i>	<i>399</i>	<i>Nan</i>	<i>-</i>	<i>456</i>	<i>Nan</i>	<i>-</i>
<i>B-3</i>	<i>377</i>	<i>Nan</i>	<i>-</i>	<i>442</i>	<i>Nan</i>	<i>-</i>
B-4	328	365	11%	396	453	14%
B-5	324	341	5%	384	390	2%

7. BENDING-SHEAR INTERACTION

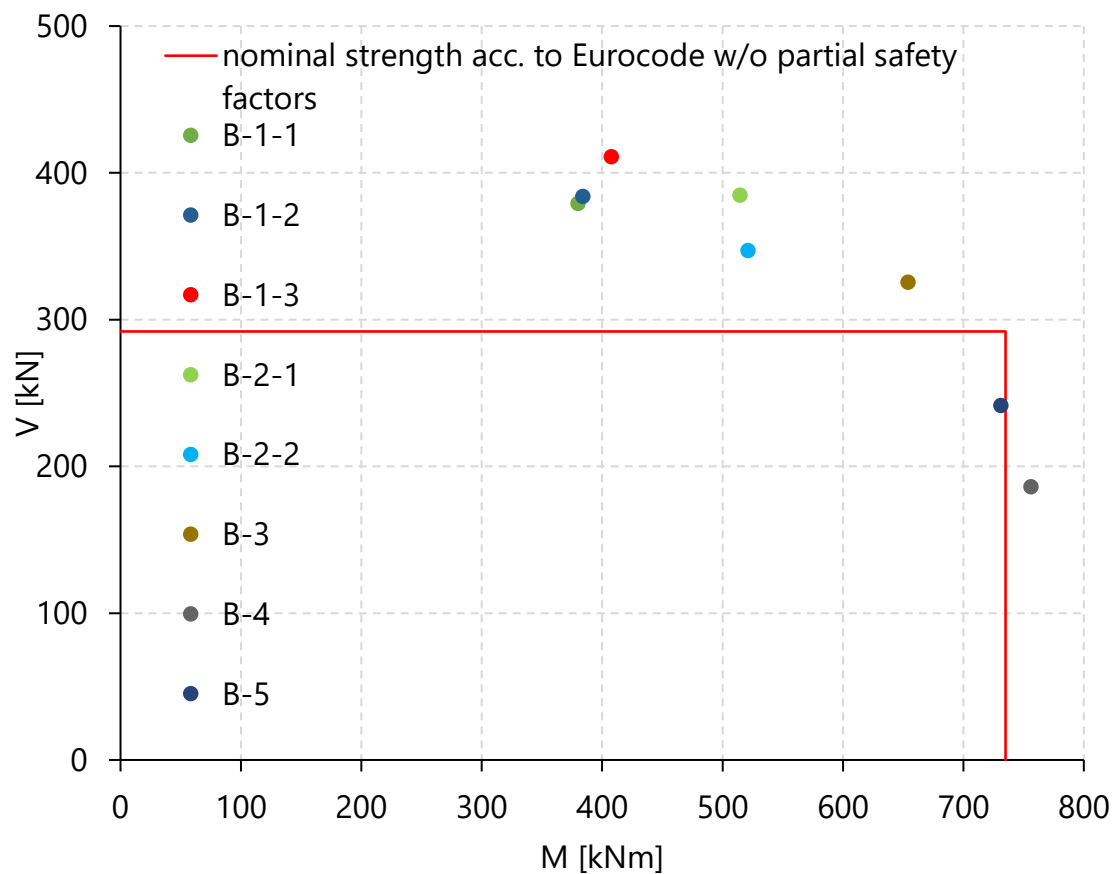
In this section I will present the main part of my research, where I will compare the M-V (bending shear) interaction behavior of beams with the Eurocode design rules.

First, I determined the moment and shear resistances for each beam by hand calculation. Here I also took into account the self-weight and, in the case of specimens

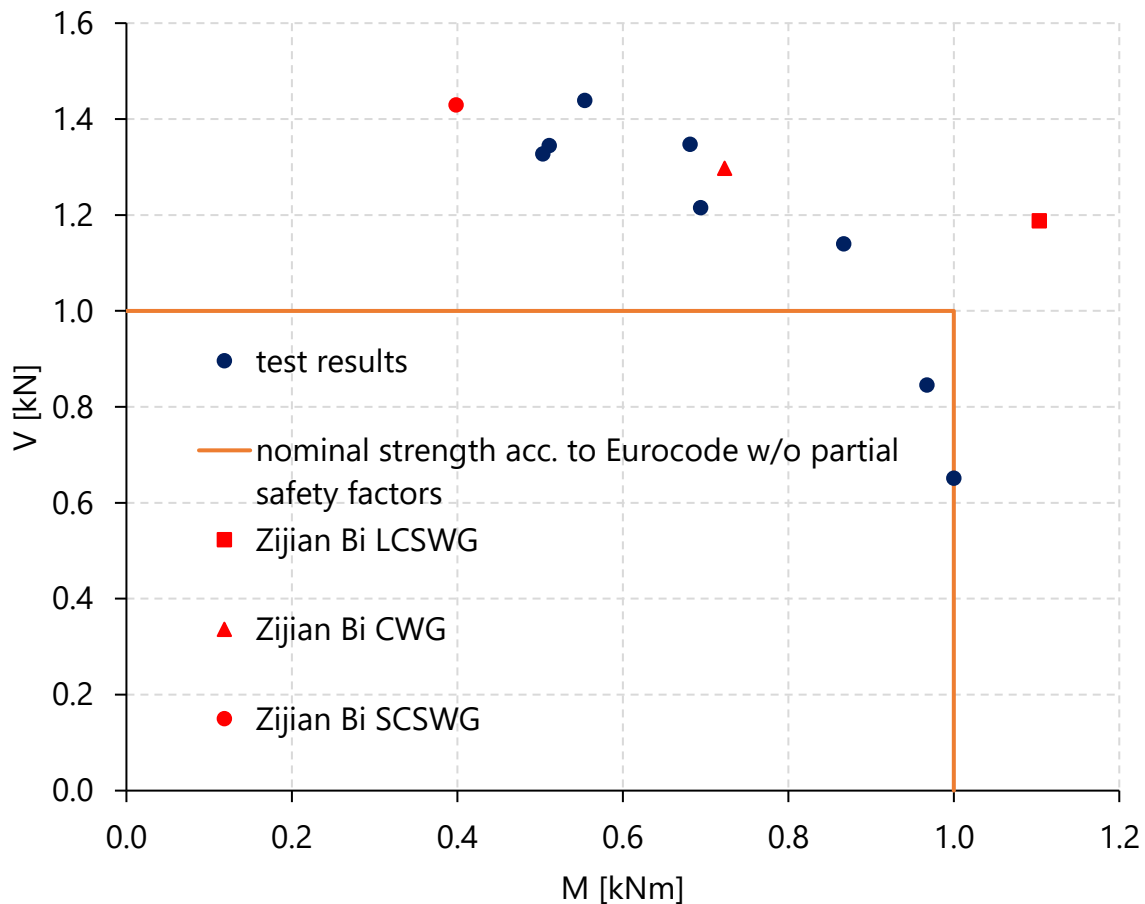
used more than once, the internal forces due to the overhang. These could have been negligible, their effect on the internal forces is about 1-2%.

Girder no.	Span [m]	Load position [m]	Cantilever length [m]	M_{test} [kNm]	V_{test} [kN]
B-1-1	8	1	0	379.8	379.1
B-1-2	7	1	1	384.0	384.0
B-1-3	5.67	1	2.33	407.6	411.0
B-2-1	8	1.333	0	514.3	384.9
B-2-2	6.67	1.5	1.33	521.0	347.1
B-3	8	2	0	653.9	325.5
B-4	8	3	0	731.1	241.4
B-5	8	4	0	756.0	186.0

Then I used Eurocode to determine the shear resistance of the cross-section. The results summarised in the previous table are plotted in the M-V interaction graph below. The load carrying capacities from the Eurocode is indicated by the red lines. (It is straight because the standard does not take into account the effect of the interaction of the bending moment and shear on the load carrying capacity, as it is described in Section 2.1.)



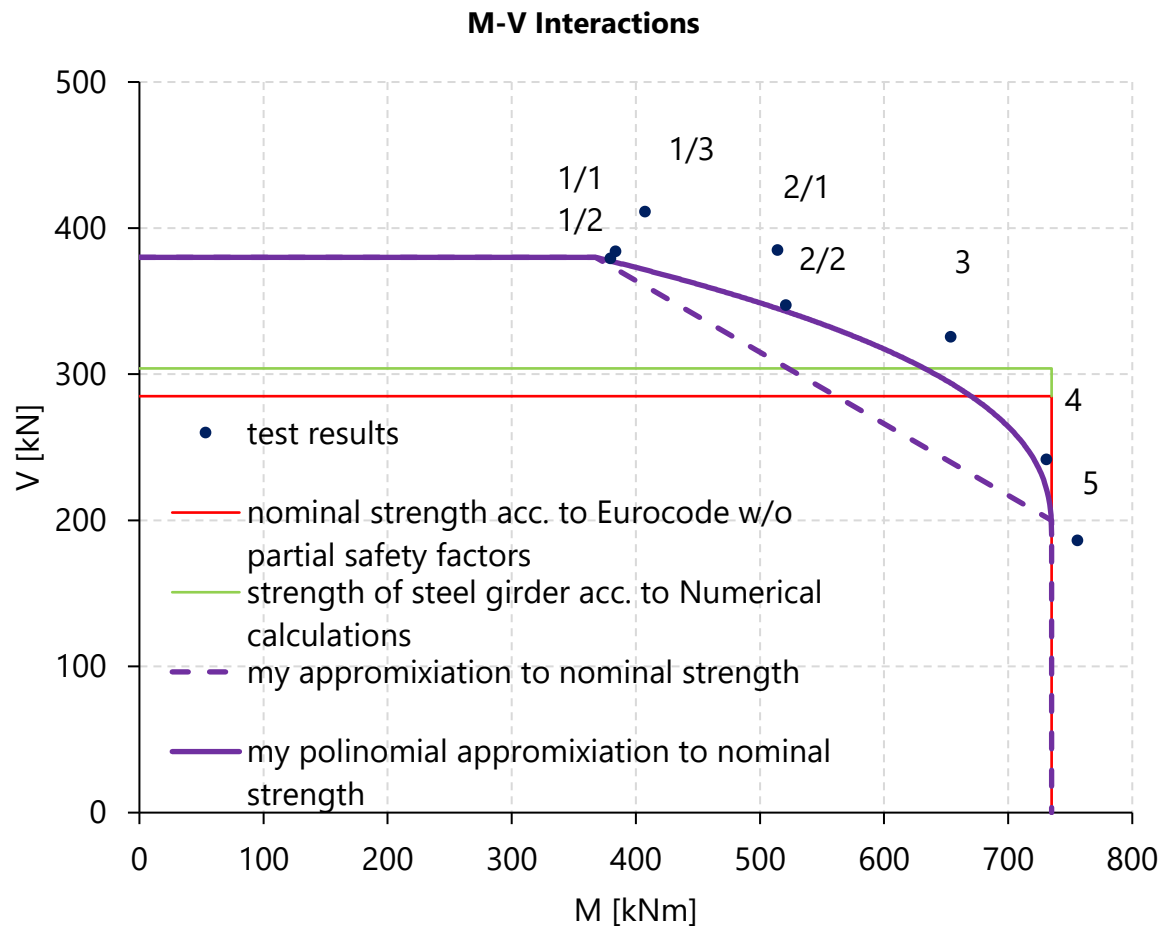
7.1. Figure: M-V interaction graph of test specimens



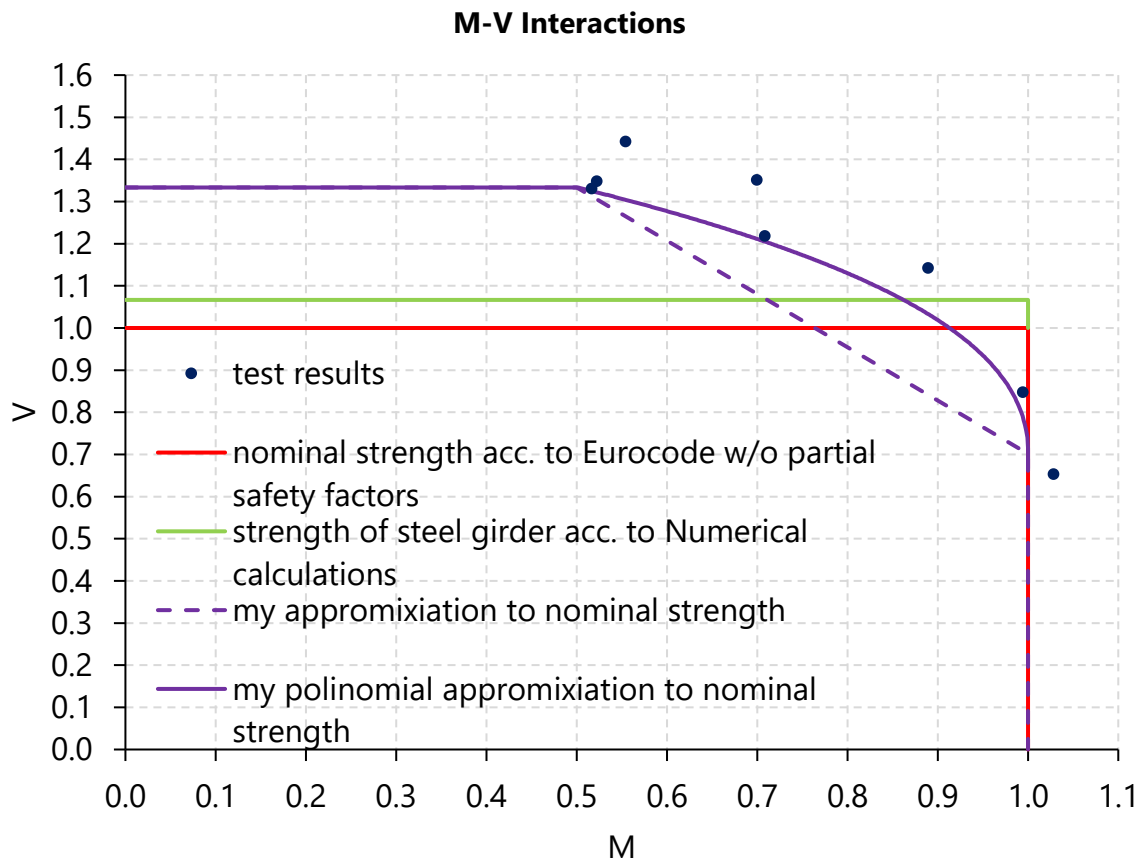
7.2. Figure: normalised M-V interaction graph with chinese test results

It clearly shows which beams exceed the load capacity specified in the standard. Where shear failure occurs there is a significant increase in the measured capacities.

In order to identify further points on the M-V interaction diagram, I searched for experimental results of other research colleagues in this interpretation range. I found three similar measurements by Zijan Bi from China (red markers) and added them to my own Eurocode normalized load carrying capacity diagram.



7.3. Figure: M-V interaction graph with my app. to the nominal strength



7.4. Figure: Normalised M-V interaction graph with my app. to the nominal strength

Based on the collected experimental test results a preliminary bending-shear interaction curve is developed presented by the purple line in the above figure. The interaction curve is found in the form of:

$$M/M_R + 0,5 \cdot (2 \cdot V/V_R - 1)^{2.5} \leq 1.0 \quad (31)$$

where: $M_R = 735 \text{ kNm}$, $V_R = 380 \text{ kN}$.

All the experimental test results are above the preliminary interaction curve. In order to determine a more general interaction curve, an extended parametric study is required. Which was not done within the scope of this study.

Nevertheless, it can be clearly seen from the normalised plot (Figure 6.6.2) that the contribution of the concrete flanges to the integrated shear resistance is around 15-40%.

8. CONCLUSIONS

The RC flange contribution to the shear resistance is notable, ~20-30% is observed in the tests. My plan is to extend the results by an extended numerical parametric study. This requires a more accurate calibration of the FE model in the future.

9. ACKNOWLEDGEMENT

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