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BENDING RESISTANCE OF COMPOSITE STEEL-CONCRETE FLOOR SYSTEM MADE OF BUILT-UP COLD-FORMED STEEL ELEMENTS

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ABSTRACT

This study explores the utilisation of built-up cold-formed steel elements combined with concrete to create efficient and lightweight composite floor systems suitable for both new constructions and reconstructions, thanks to their reduced self-weight. The main focus of this paper is to investigate different analytical approaches for determining the bending resistance of an innovative lightweight composite floor system called LWT-FLOOR. The LWT-FLOOR system is composed of spot-welded built-up cold-formed steel elements that are connected to a concrete slab. After introducing the LWT-FLOOR system, various commonly used analytical approaches for calculating its bending resistance are examined. The first approach relies on the plastic bending resistance of composite cross-sections with the full shear connection. The second approach modifies the Eurocode 4 procedure to account for the elastic behaviour of cold-formed steel cross-sections in bending, considering partial shear connection. Lastly, the third approach considers non-linear bending resistance. As anticipated, the first approach yields the highest bending resistance. However, achieving a full shear connection is not always feasible, and the results obtained from the second and third approaches, considering partial shear connection, indicate bending resistances that are still acceptable for practical applications. The findings of this research serve as a foundation for future numerical and experimental investigations to identify the optimal analytical approach for determining the bending resistance of the proposed system.

Keywords: Composite System; Built-up Cold-formed Steel; Bending Resistance; Analytical Approaches.

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1. INTRODUCTION

Humanity is at the point where it needs to completely rethink its lifestyle in order to reduce its negative environmental impact and become sustainable. In civil engineering this aspect is mostly related to reduction of raw material and energy consumption, which can be achieved through reuse and recycling of building materials and structural components during their lifecycle. For that purpose, innovations in the building industry, providing relevant solutions, are needed. Although there is no systematic approach, scientists and engineers all over the world are aware of this issue and are already tackling the problems through individually provided solutions.

Composite steel-concrete are systems that seem to provide a required sustainable solution in the construction industry. These systems are, for a fact, known to provide cost-effective multi-storey building solutions. The reason for that lies in integration of structural efficiency with the construction speed. Structural efficiency comes as a result of effective utilisation of structural materials, which is secured through omission of their disadvantages. Namely, concrete is dominantly used in compression while steel is dominantly used in tension. Furthermore, construction time can be significantly shortened when formwork installation and propping is reduced or even completely omitted. However, the field of composite steel-concrete structures is constantly evolving which is, according to Ahmed et al. [1], driven by considerations of socioeconomic and environmental consequences towards sustainability and resilience and is related to the development of innovative construction methods and new structural products.

One of the innovative solutions in composite-steel concrete systems is to apply cold-formed (CFS) elements which are known to provide high stiffness and strength, allow easier prefabrication and installation, as well as reduce transportation and handling costs [2]. The use of CFS profiles allows formation of various optimised cross-section shapes as well as formation of built-up cross-sections which through various arrangement of single CFS profiles can significantly increase resistance of built-up sections and members. Furthermore, their combination with concrete slab can then result in effective and lightweight composite floor systems.

Here, a newly proposed innovative structural system solution composed of built-up CFS corrugated web (CW) girders and concrete slabs, named LWT-FLOOR system [3,4], is presented. To fulfill the aforementioned sustainability goals the proposed system considers the use of a demountable shear connection between steel and concrete parts of the composite cross-section. As such a system is new the standards do not provide guidelines for its resistance calculation. Therefore, the aim of this paper is to present the system itself, as well as review available analytical approaches within the existing literature. The results of this research will then serve as a foundation for future numerical and experimental investigations to help identify the optimal analytical approach for bending resistance calculation of the newly proposed system.

2. LWT-FLOOR SYSTEM

The application of CFS sections in steel-concrete floor systems leads to advantages such as [5]:

- The possibility of reducing overall slab depth by using lighter sections at closer spacing.
- Ease of cross-section variation for irregular layouts.
- Freedom in cross-section design.
- Flexibility in assembling the sections and attached components in the workshop and/or on-site.
- Simplicity and availability of manufacturing technology for CFS sections.

To further increase the capacity of CFS - concrete composite systems, several CFS elements can be combined to form a built-up section. CW beams represent a relatively new structural system that was developed for various applications, i.e. the mainframes of single-storey steel buildings, secondary beams of multi-storey buildings, etc. CW beams use thin webs (1.5 mm to 3 mm) which significantly reduces their weight compared to hot-rolled profiles or welded I-sections. The CW also increases the beam's local and lateral-torsional buckling stability and reduces possibility of web crippling, which may result in a more effective design. In static terms, the CW beam can be compared with a lattice girder where bending moments and applied forces are transferred through the flanges, while the shear forces are transferred only through the infill of the lattice girder. Therefore, the flanges provide the flexural strength of the girder while the infill, in this case CW, provides the girder's shear capacity. Furthermore, the use of CW reduces need for stiffeners and allows use of thinner webs which leads to estimated cost savings of 10-30 % compared to conventionally fabricated sections and more than 30 % compared to standard hot-rolled beams[6,7]. Further enhancement of CW beams can be achieved through usage of CFS profiles, instead of flat hot-rolled plates, to form a flange of built-up section. This has been demonstrated through investigation of built-up CFS CW beams where cross-section parts were connected using screws [6,8,9], or spot welding (SW) and cold metal transfer techniques [10–12], where spot welding technique turned out to produce built-up beams with greater strength and stiffness compared to the ones using self-drilling screws as means of connection. Additionally, the SW technique significantly increases the speed of production when compared to traditional arc welding technique and enables automation of the production process. Numerous full-scale experimental as well as numerical research have been conducted on such beam specimens which used built-up CFS steel section with the application of the SW connection technique with different variations [10–15], which proved that such a solution has excellent behaviour.

Based on these findings a new solution was proposed which will maximise the values of each building component and used materials. The solution proposes new composite floor system termed LWT-FLOOR and it has been investigate within the LWT_FLOOR research project carried out at the University of Zagreb, Faculty of Civil Engineering, Croatia. The LWT-FLOOR project integrates state-of-the-art knowledge in new, fast, and productive SW technology and innovative cold-formed steel-concrete composite solutions proposing a new construction method as a combination of built-up cold-formed steel members and cast-in-place concrete slab. Additionally, particular focus within the project, besides already mentioned SW connections, will be given to innovative types of shear connections between steel and concrete part of the cross-section which will allow disassembling and the potential reuse or recycling at the end of the product design life. Such a solution presents cost-effective and sustainable floor system which can result with multiple benefits such as a high degree of prefabrication, reusability/recyclability, and high strength capacity.

Regarding sustainability philosophy the proposed LWT-FLOOR system considers many aspects, and falls into the category of adaptive building technology, following environmental and climate change requirements. It uses steel which is highly recyclable, but also cold-formed products that are known to use a high percentage of recyclable steel [15]. Furthermore, due to its demountable shear connection, the system has the potential for future reuse and recycling of materials and components. SW technology proved its economic effectiveness as well as sustainable performance in terms of consumption of welding materials, energy, workers' health and safety issues, elimination of inert shielding gases and particle emissions into the environment. The system possesses durability with regard to climate change effects and in an aggressive environment due to its high protection from corrosion and the fact that all components are galvanised. The system components are easy to manipulate, transport and erect because of the reduced weight and safe connection technology. This can contribute to the safety of workers in the shops and building sites, cleaner activity on working sites, reduced energy consumption and lower emissions of greenhouse gases. In the case of inner-city projects, construction speed and lack of on-site storage require a high level of prefabrication, which the proposed system can provide [14].

Mentioned benefits of the system will be investigated within the LWT-FLOOR project through experimental and numerical research with the support of probabilistic methods and lifecycle analyses by means of a holistic approach combining Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP) analyses.

3. ANALYTICAL RESEARCH

3.1. Configuration of analysed LWT-FLOOR system

The analysed LWT FLOOR system, Fig. 1, consists of four cold-formed C120 steel sections whose thickness is 2.5 mm, lipped channel flange of 47 mm width, lip length of 21 mm and inner radius of 3 mm. System also consist of shear plates made of steel sheets with a thickness of 1.0 mm and various heights (400 mm, 500 mm, 600 mm), the corrugated web with various thicknesses (0.8 mm, 1.0 mm, 1.5 mm) and heights (400 mm, 500 mm, 600 mm) and metal sheets that serve as formwork for a concrete slab with an overall thickness of 120 mm and effective width of 1500 mm made of concrete class C25/30. All configurations are analysed for beam span of 6 m.

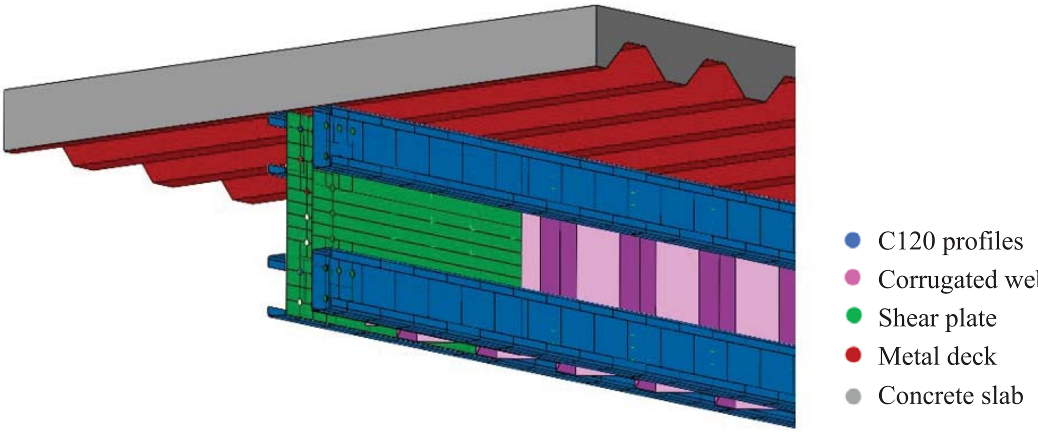
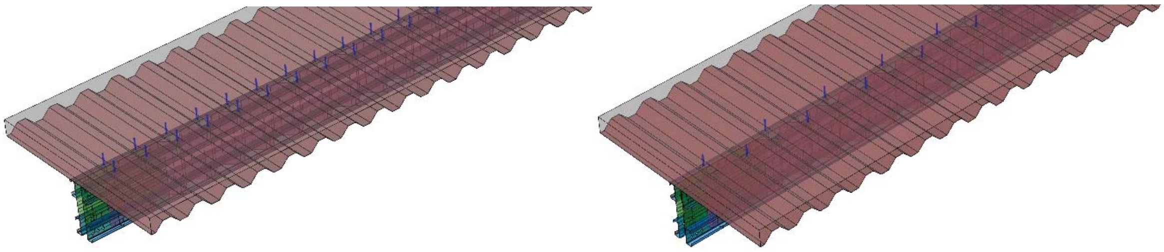


Fig. 1. Configuration of analysed LWT-FLOOR system

In order to achieve different degrees of shear connection, shear connectors are positioned in pairs or in a staggered position, as shown in Fig. 2.



a) in pairs b) staggered position

Fig. 2. Longitudinal arrangement of shear connectors

3.2. Analytical Bending Resistance of LWT-FLOOR system

Three analytical approaches were considered for the calculation of the bending resistance of the proposed composite LWT-FLOOR beam.

The first approach considers plastic stress distribution for calculation of the bending resistance. This, necessary requires finding of the plastic neutral axis position which can be done using Eq. (1) as:

$$x_{pl,k} = 4 \cdot A_a \cdot f_{yk}/(b_{eff} \cdot 0.85 \cdot f_{ck}) \tag{1}$$

where A_a is the cross-section area of the CFS C profiles, f_{yk} is the characteristic steel yield strength, f_{ck} is the characteristic value of concrete cylinder compressive strength at 28 days, and b_{eff} is the width of the reinforced concrete slab calculated according to EN 1994-1-1 [16].

Fig. 3 presents the distribution of normal stresses on the cross-section of the analysed LWT-FLOOR system, assuming the position of the plastic neutral axis (n.a.) within the concrete slab. Additionally, it is also assumed that the full degree of shear connection is achieved. In such a case, the whole steel section is under normal tensile stresses, which allows utilisation of its plastic resistance, even though the CFS sections are very slender and usually fall within the third or fourth cross-sections class.

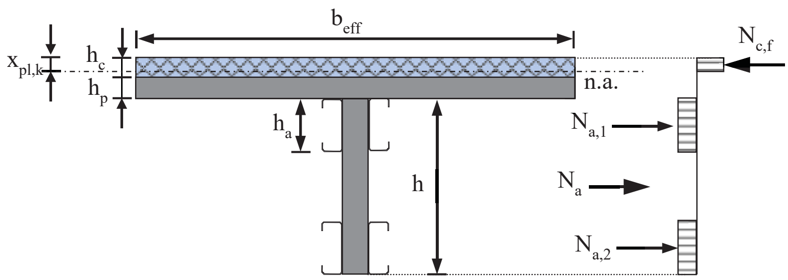


Fig. 3. Normal stress distribution in the LWT-FLOOR composite beam cross-section

The plastic bending resistance of cross-section with plastic n.a. in slab and full shear connection assumption can be calculated using Eq. (2) as:

$$M_{pl,Rk} = 2 \cdot A_a \cdot f_{yk} \cdot (\frac{h_a}{2} + h_c + h_p -/2) + 2 \cdot A_a \cdot f_{yk} \cdot (h - h_a/2 + h_c + h_p - x_{pl,k}/2) \quad (2)$$

where h_a is the height of the CFS C section, h_c is the height of the concrete slab, and h_p is the height of the profiled sheet rib positioned perpendicularly to the composite beam direction.

In case when the bolts are arranged in such a way that a full shear connection between steel and concrete parts cannot be achieved another analytical approach can be used. This approach calculates bending resistance using the approach for partial shear connection given in Eurocode 4 [16] with a slight modification. Here the plastic bending resistance of the pure structural steel section, $M_{pl,a,Rk}$, is replaced with the elastic bending resistance of the pure structural steel section, $M_{el,a,Rk}$, as given in Eq. (3) [3,17].

$$M_{Rk} = M_{el,a,Rk} + (M_{pl,Rk} - M_{el,a,Rk}) \cdot \eta \quad (3)$$

where $M_{pl,Rk}$ is the bending resistance of the composite beam in the case of full shear connection (calculated using Eq. (2)), and η is the degree of shear connection calculated using Eq. (4) as follows:

$$\eta = n/n_{full} \quad (4)$$

where n is the number of shear connectors used and n_{full} is number of shear connectors required to achieve a full shear connection. The actual number of used shear connectors, n, in case when ribs of profiled sheeting are perpendicular to the beam direction, depends on the available number of metal sheet ribs within the shear length and on the number of the shear connectors within each rib. In this matter, this paper considers two cases, i.e. two shear connectors per rib or one shear connector per rib, in which case they are then placed in a staggered arrangement along the length of the beam. For shear connectors, this paper assumes the application of 12 mm diameter bolts with a grade quality of 8.8. This falls outside the scope of the European standard [16] where the equations for calculating the resistance of headed shear studs are valid for tensile strengths up to 500 N/mm^2 and for the diameters between 16 mm and 25 mm. However, for this research it was assumed that these equations are also valid when using bolts as shear studs.

Additionally, according to EN 1994-1-1 [16], to calculate the resistance of the shear connector when the ribs of the metal sheet are positioned perpendicular to the direction of the composite beam, it is necessary to apply the reduction coefficient, k_t using Eq. (5)

$$k_t = 0.7/\mathrm{sqrt}(n_r) \cdot b_0/h_p \cdot (h_{sc}/h_p - 1) \leq k_{t,max} \quad (5)$$

where b_0 is a mean width of a concrete rib, h_{sc} is the overall nominal height of a connector, n_r is the number of stud connectors in one rib, h_p is the overall depth of the profiled steel sheet (excluding embossments), and $k_{t,max}$ is the limit value of the reduction coefficient provided in [16]. Minimum overall height of the shear connector per EN 1994-1-1 [16] is required to be:

$$h_{sc} = h_p + 2 \cdot d \tag{6}$$

where d is the diameter of the shear connector.

Finally, the shear connector resistance is calculated as:

$$P_{t,Rk} = k_t \cdot P_{Rk} \tag{7}$$

where P_{Rk} is the design value of the shear resistance of a single connector calculated using equations from EN 1994-1-1 [16].

The bending resistances calculated using these two described approaches are provided in Table 1. Bending resistances, as well as degrees of shear connection, are calculated for two bolt arrangements and regarding the application of reduction coefficient, k_t.

Table 1. Composite beam bending resistances and shear connection degrees.

Bolt Arrangement	Steel beam height [mm]	Bending Resistance [kNm]		Degree of Shear Connection [–]	
		without k_t	with k_t	without k_t	with k_t
Pairs	400	257.7	181.7	1.0	0.64
Staggered		210.0	142.7	0.78	0.46
Pairs	500	299.7	215.3	1.0	0.64
Staggered		246.8	171.9	0.78	0.46
Pairs	600	341.7	249.2	1.0	0.64
Staggered		283.7	201.6	0.78	0.46

The third approach considered in this work calculates the non-linear bending resistance in accordance with [16,18,19]. Here, the elastic bending resistance of the cross-section is calculated by converting the concrete slab into a steel cross-section. Then, the ideal area of the concrete slab cross-section, as well as the distance between the centre of gravity of the composite section and the concrete slab, must be found. The ideal area of the concrete slab is calculated by considering the thickness of the concrete slab above the centre of gravity of the composite section. Finally, it is possible to calculate the ideal second moment of area of the concrete slab and of the entire composite section. The stiffness of the composite cross-section is calculated using Eq. (8):

$$EI_L = E_a \cdot I_{i,L} \tag{8}$$

where E_a is the modulus of the elasticity of steel and $I_{i,L}$ is the ideal second moment of the area of the composite section.

Furthermore, non-linear bending resistance can be considered for propped and unpropped systems. In a propped system, with regard to the maximum allowable stresses within steel and concrete sections, two equations can be formulated to calculate the bending resistance. Eq (9) is limited by stressed in steel, while Eq. (10) by stresses within the concrete slab.

$$\sigma_a = E_a \cdot M_{Rk,1}/EI_L \cdot z_a \tag{9}$$

$$\sigma_c = E_c \cdot M_{Rk,2}/EI_L \cdot z_c \tag{10}$$

where $M_{Rk,1}$ is the maximum allowable moment that will cause stress in the steel section and $M_{Rk,2}$ is the maximum allowable moment that will cause stress in the concrete section. The equation providing lower result from the two should then be considered in further calculations. To choose the right expression for elastic bending resistance, as demonstrated in [16], the value of a normal compressive force in a concrete slab which corresponds with $M_{el,Rk}$, obtained as minimum M_{Rk} value from Eqns. (9) and (10), can be calculated as:

$$N_{c,el} = k \cdot M_{el,Rk}/I_{i,L} \cdot (A_{c,L} \cdot z_{ic,L}) \tag{11}$$

$$z_{ic,L} = A_a \cdot a_a/A_{i,L} \tag{12}$$

where k is the minimum value between k_a, k_s, and k_c. These values are obtained from the cross-section outermost fiber that has reached its limit in terms of stress. $A_{c,L}$ is the area of the ideal composite cross-section, $z_{ic,L}$ is the distance between the centers of gravity of the composite cross-section and the center of gravity of the concrete slab; a_a is the distance between the center of gravity of the cross-section steel part and the center of gravity of the composite cross-section.

On the other hand, an unpropped system needs to be considered during the construction phases. In the first phase (pure streel section), the weight of the steel beam and concrete must be considered, and then the characteristic bending moment of the calculated load is used to calculate the stresses on the steel section. In the second phase (composite steel-concrete section), the stress in the steel section is assumed to be limited by the yield stress; this allows the remaining stress to be taken into account. Two bending moments were calculated using the equations that calculate stresses in the steel (Eq. (13)) and in the concrete (Eq. (14)); the minimum value is associated with $M_{Rk,II}$.

$$\sigma_{a,II} = E_a \cdot M_{Rk,II,a}/EI_L \cdot z_a \tag{13}$$

$$\sigma_c = E_c \cdot M_{Rk,II,c}/EI_L \cdot z_c \tag{14}$$

The elastic bending resistance of the composite section is then calculated as follows:

$$M_{el} = M_{Rk,I} + k \cdot M_{Rk,II} \tag{15}$$

where $M_{Rk,I}$ is the design bending resistance from first phase, and k is the lowest factor when a stress limit is reached.

Following the conditions given in EN-1994-1-1 [16], the characteristic value of the bending resistance of a composite section is calculated using Eqns. (16) and (17).

for $N_c \leq N_{c,el}$

$$M_{Rk} = M_{a,Rk} + (M_{l,Rke} - M_{a,Rk}) \cdot N_c/N_{c,el} \tag{16}$$

for $N_{c,el} \leq N_c \leq N_{c,f}$;

$$M_{Rk} = M_{el,Rk} + (M_{pl,Rk} - M_{el,Rk}) \cdot ((N_c - N_{c,el})/(N_{c,f} - N_{c,el})) \tag{17}$$

The bending resistances calculated using nonlinear approach for unpropped and propped construction are provided in Table 2. It is important to note that these bending resistances are obtained using the degrees of shear connection calculated in Table 1. i.e., values of 0.46 and 0.64 obtained for one and two bolts per rib, respectively.

Table 2. Composite beam bending resistances and shear connection degrees.

Bolt Arrangement	Steel beam height [mm]	Bending Resistance [kNm]	
		Unpropped	Propped
Pairs	400	176.39	161.74
Staggered		134.71	112.55
Pairs	500	208.46	194.42
Staggered		161.69	139.72
Pairs	600	240.83	227.21
Staggered		189.12	167.27

Finally, a simplified relationship between the bending resistance, M, and the normal compressive force within the concrete slab, N_c, is provided in Fig. 4. The diagrams provided in Fig. 4. show differences between results when propped (orange line) and unpropped (green line) construction procedure is used, and are defined using approach provided in [16].

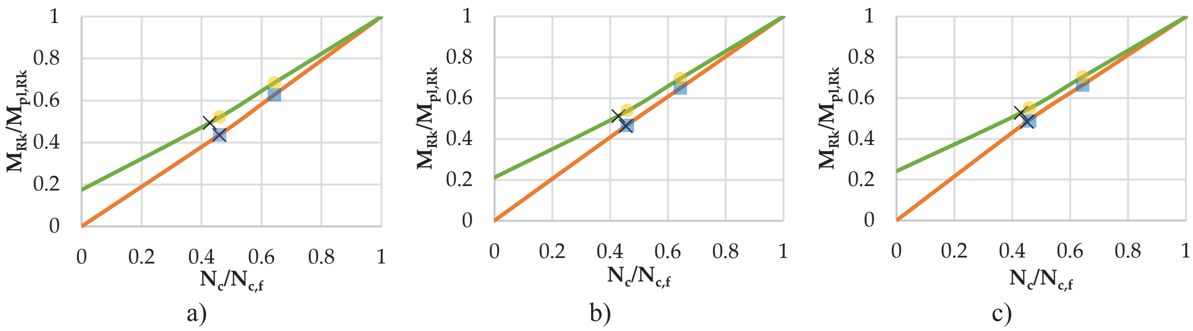


Fig. 4. Relationship between degree of shear connection and bending resistance of composite LWT-FLOOR system using various steel beam heights: a) 400 mm; b) 500 mm; c) 600 mm

4. CONCLUSIONS

This paper analyses three analytical approaches for calculating the bending resistance of the newly proposed composite LWT-FLOOR system comprised of a built-up CFS beam and concrete slab within profiled steel sheeting. Applied analytical approaches are taken from European Standard even though the analysed system falls out of the scope of the mentioned standard. As CFS elements are very slender elements, the analytical approaches are based on the elastic distribution of stresses. However, in cases when the built-up CFS beam is completely under tension stresses, the plastic stress distribution can be assumed, which allows the utilisation of composite section plastic resistance. The results of this paper will serve as a foundation for further numerical and experimental research to identify the optimal analytical approach for determining the bending resistance of the newly proposed system.

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