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CO_2 –optimization design of a concrete slab structure

Abstract: In these days the environmental consciousness, the environmental footprint and minimizing the CO_2 emissions have taken a significant role in every part in the industries. The construction industry is not an exception, where the production of the concrete, steel and the reinforced concrete structures are one of the largest emitting institutions worldwide. Prioritizing the environmentalism and reducing the harmful emissions made by these materials is obligatory. By changing the geometry of the structure, the reinforcement, the concrete strength class, and also using the right design method we are able to reduce significantly the environmental footprint without reducing the load bearing capacity of the structure. This paper introduces an optimization of a traditional reinforced concrete slab. During the optimization the flat slab was converted into ribbed slab to reduce the net weight. Instead of the traditional steel stirrups synthetic macro fibres were used as shear reinforcement. Using these materials, the CO_2 emissions are much lower than with traditional steel reinforcement, because of their production, shipping and labour work have less environmental loads. In addition to the traditional analytic calculation, the structure has been optimized using advanced finite element analysis, which provides more opportunities for optimization. The developed structure is functionally equivalent to the traditional slab, however, generates significantly less CO_2 emissions.

1. Introduction

In today's world, the building sector is the third largest CO_2 emitting industry worldwide. According to data from the National Climate Data Center [1], the average surface temperature of the earth increased by 0.07 °C every decade since 1880, and the average sea level as of 2019 has risen by 21-24 cm compared to 1880 [2, 3, 4]. According to the 2019 Global Status Report for Buildings and Construction [5] by the United Nations Environment Program, building construction represents 28% of global energy-related CO_2 emissions (39% when construction industry emissions are included) [6].

With the support of the Paris Agreement under the Global Climate Action Agenda the EU and its Member States are committed to a binding target of a net domestic reduction of at least 55% in greenhouse gas emissions by 2030 compared to 1990. With the global population increasing by 2.5 billion by 2050, new buildings will have an important effect on future buildings-related energy use and emissions [6]. To support the reduction of emissions, several design methods, material usage could show significant results.

Concrete is the world's most consumed construction material. Referring to Ritchie at Our World in Data [7], the gas emission of iron and steel is more than 7.2% while the cement industry takes more than 3% of the total global greenhouse gas emission. We distinguish between operation emissions and embodied emissions. Embodied emissions in construction contains all greenhouse gas emissions arising from procuring, mining, harvesting raw materials, transporting, transforming these materials into construction products, maintaining, disposing, etc. In addition to this there are building operation emissions which appear from the energy used for heating and/or cooling, ventilation and air conditioning, lighting, hot water supply, and process-related climate-relevant Green House Gas (GHG) emissions, etc.

According to Hendriks et al. [8] the production of 1 kg cement generates 0.9 kg of CO_2 which equates to about 3.24 billion tons of CO_2/year. This data only accounts for the production, the embodied

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emissions would account for a far more significant amount of CO_2. Previous research indicated that the construction, operation, and demolition stages are responsible for approximately 13%, 85% and 2% of CO_2 emissions [9, 10]. For the cement manufacture over 90% of the energy is required from fossil fuels and the remaining 10% of the energy is obtained via electricity [11]. The cement industry is one of the highest consumers of fossil fuel energy, approximately consuming 12-15% of total industrial energy use, with an estimated 1.75 ± 0.1 MJ of energy required to produce 1 kg of cement [12,13].

The production of materials such as non-metallic minerals, cement, iron, steel etc. and material transportation contributes 82-96% of the total CO_2 emissions through the construction period. Production of raw materials means 80-93% of total emissions which from the production of non-metallic minerals, cement, iron, steel takes 44%.

In addition, the global carbon footprint of plastics has doubled since 1995, in 2015 reaching 4.5% of global GHG emissions [14]. From 2015 to 2017 approximately 6300 Mt of plastic waste had been generated, around 9% of which was used up for recycling, 12% was incinerated and 79% was accumulated in landfills or the natural environment [15].

Fig. 1. Cement manufacturing accounts for 8% of the world's carbon dioxide emissions [16]

From this data, the waste of plastics could be reduced if the use of recycled plastics would be vindicated in the construction field such as synthetic reinforcement. Taking a bigger importance and attention is for the plastic to be reused in a more ethical way, which would reduce the plastics that are accumulated in landfills or the natural environment.

To reduce greenhouse gases, it should be considered from the very beginning to undertake sustainable design which focuses much more on the materials we use for construction. In general terms, sustainable construction means building with renewable and recyclable resources and materials, or through new technologies and design methods that reduce the amount of material required.

The primary focus of this study is minimizing carbon emissions and reducing resources by optimizing the geometry and the materials of a slab structure.

2. Design of a reinforced concrete slab

The current practice in structural design of slab structures is to calculate flat slabs. These structures definitely have their benefits, like increasing the speed of the construction, optimizing the labour and the material of the formwork. Also, their design method is simple, it does not need any advanced tools like finite element analysis. However, in the majority of the structure, the concrete does not add any additional capacity to the structure, it simply adds weight. To meet with the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) requirements usually these structures contain large amounts of steel reinforcement.

Also, some of these steel bars are not utilised, their inclusion is only because of tradition (e.g. top reinforcement in the sagging zone, etc.).

In case of precast elements, the steel reinforcement is optimized, but they have their own limit: only designated spans can be made. Also, lifting and installing these elements in most cases is time consuming and requires special equipment.

Ribbed or waffle slab is a slab system which consists of a series of parallel reinforced concrete T beams framing into reinforced concrete girders. The slab is the flange of the beam, and the extended part is the web. The extended part is known as ribs. This design and construction concept is useful in long-span construction of floors where the self-weight becomes excessive compared to the applied dead and imposed loads, thereby resulting in an uneconomic method of construction. They also provide a very good form of construction where slab vibration is an issue, such as laboratories and hospitals. The ribbed slabs decrease the weight of the floor and thus can increase the allowable live load. In some cases, it is also possible to decrease the size of the foundation of the building, because the self-weight of the slabs will be significantly lower. However, manufacturing and spacing the stirrups in the beams are time consuming and needs extra labour work.

As mentioned, along with the material and labour costs of slab elements, the CO_2 emissions should also be considered in the design phase. Taking into consideration the effect of carbon footprint, the ribbed slab looks more economic, however the labour work of manufacturing the stirrups needs to be re-thought.

The critical failure mode in flat slab structures is normally the bending, or the punching shear if the slab is supported by columns. The shear reinforcement at supports is not necessary, due to the shear capacity of the relatively high cross section of the slab. If the geometry of the flat slab is changed to ribbed, the slab part will be thinner, which leads to smaller shear capacity. According to the Eurocode 2 [17] in case of T section beams the shear capacity should be calculated only from the web of the beam, the contribution of the flanges should be neglected. However, this should not cause any design problem, because according to the traditional design in the beam parts some stirrups should be used.

In the following chapter a new way will be presented for ribbed slabs, which shows benefits in cost, labour, and CO_2 emissions as well.

3. Specific example

The main disadvantage, next to all the benefits of using a ribbed slab, is the increased labour work required to manufacture, transport, and install the reinforcement of the slab beams. This part of the construction can be reduced if fibre reinforcement is added to the structure, replacing the steel stirrups. Over the past decades several publications showed that fibre reinforcement can increase the shear capacity [18, 19, 20] by providing post-cracking tensile resistance across inclined cracks, resulting in higher aggregate interlock forces in a manner similar to that observed for beams with normal stirrup shear reinforcement. Several studies have been performed in the past about using steel fibres in RC beams as shear reinforcement. However, keeping in mind the decrease of CO_2 emission, the changing from steel fibre to macro synthetic fibre should be investigated. Previous studies showed [21, 22, 23], that macro synthetic fibres can enhance the shear strength of reinforced concrete beams significantly.

In this article a parametric study was carried out for a simple supported slab element. The main goals were the following:

- investigate the behaviour and failure mode of a ribbed slab with decreasing the thickness of the slab;
- define the shear capacity of the slab with synthetic fibre reinforcement;
- compare analytical and numerical calculation methods;
- define the CO_2 emission for the models.

The geometry of the calculated structure can be seen in fig. 2.

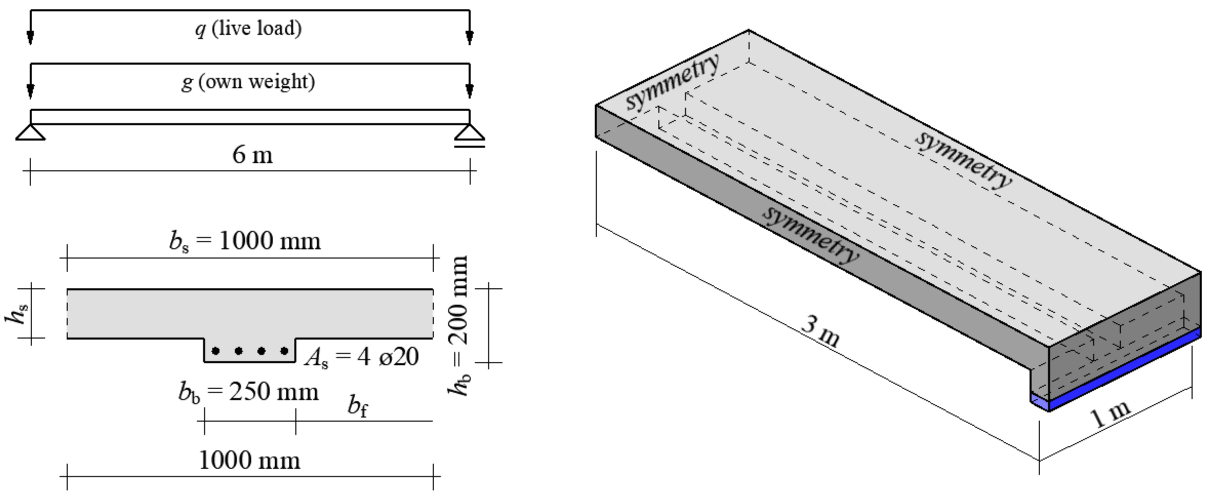

Fig. 2. Geometry of the structure

The effective span of the slab was 6.0 m, while the modelled width was 1.0 m. The shape of the rib was a rectangle with a width of 250 mm and height of 200 mm. The thickness of the slab was the variable parameter from 40 mm to 200 mm. The slab had only 4 Ø20 main reinforcement in the rib, the rest of the element was unreinforced.

By decreasing the thickness of the slab, the effective surface close to the supports became smaller, thus the failure mode changed from bending failure to shear failure. To this point the additional live load increased due to the decreasing of the self-weight. The calculations were made with using the Eurocode 2 [17] formulas. The material parameters were calculated with their design values, but the loads were applied with characteristic values.

For calculating the shear capacity of the structure, the shear for the rib part, the shear for the slab part and the shear for flange part were calculated separately, with the following formula according to fig. 3.:

$$V_{cb} = 0.12\left(1+\sqrt{\frac{200}{d_b}}\right)\left(100\frac{A_f}{b_b\,d_b}f_{ck}\right)^{\frac{1}{3}} b_b\,d_b \tag{1}$$

$$V_{cs} = 0.12\left(1+\sqrt{\frac{200}{d_s}}\right)\left(100\frac{A_f}{b_s\,d_s}f_{ck}\right)^{\frac{1}{3}} b_s\,d_s \tag{2}$$

$$V_{cf} = 0.12\left(1+\sqrt{\frac{200}{d_f}}\right)\left(100\frac{A_f}{b_f\,d_f}f_{ck}\right)^{\frac{1}{3}} b_f\,d_f \tag{3}$$

where:

V_{cb}	shear capacity of the beam part
V_{cs}	shear capacity of the slab part
V_{cf}	shear capacity of the flange part
A_f	area of fully anchored steel reinforcement
b_b	width of the rib
b_s	width of the slab
b_f	width of the flange
d_b	effective depth of the beam
d_s	effective depth of the slab
d_f	effective depth of the flange
f_{ck}	characteristic value of the concrete's compressive strength

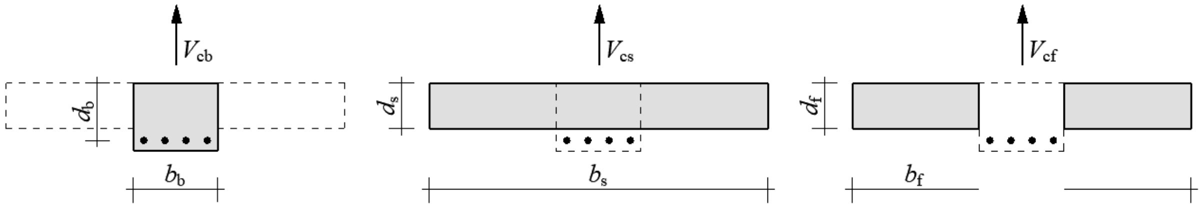

Fig. 3. Definition of shear capacity

In case of T shaped beams, the Eurocode formula eliminate the shear capacity of the slab part, it only calculates with the beams. However, this method can lead to a serious underestimation of shear capacity, especially if the slab thickness is close to the beam height. To get a more proper value the shear capacity in the parametric design was also calculated with equation 4. However, this model looks reasonable and was studied before [23] but it must be validated and developed in future research.

$$V_{\mathrm{c}} = V_{\mathrm{cb}} + V_{\mathrm{cf}} \tag{4}$$

From the moment and shear capacity calculation and from the self-weight of the structure (dead load) the maximum live load was calculated. Therefore, this live load takes into consideration the decreasing of the self-weight due to thinning the slab.

As there is a point where the shear failure is the critical failure mode, the structure was calculated also with fibre reinforcement. The effect of the fibres was added according to formulas (5)–(7). This formula is the recommendation of the MC2010 [18] for steel fibre reinforced concrete, however according to studies [21], the formula represents well the synthetic fibre reinforcement as well. In case of fibre reinforced concrete, the shear capacity was also calculated for beam, slab and ribbed slab structures like for plain concrete.

$$V_{\mathrm{cb.FRC}} = 0.12\left(1+\sqrt{\frac{200}{d_{\mathrm{b}}}}\right)\left(100\frac{A_{\mathrm{f}}}{b_{\mathrm{b}}\,d_{\mathrm{b}}}\left(1+7.5\frac{f_{\mathrm{Ftuk}}}{f_{\mathrm{ctk}}}\right)f_{\mathrm{ck}}\right)^{\frac{1}{3}} b_{\mathrm{b}}\,d_{\mathrm{b}} \tag{5}$$

$$V_{\mathrm{cs.FRC}} = 0.12\left(1+\sqrt{\frac{200}{d_{\mathrm{s}}}}\right)\left(100\frac{A_{\mathrm{f}}}{b_{\mathrm{s}}\,d_{\mathrm{s}}}\left(1+7.5\frac{f_{\mathrm{Ftuk}}}{f_{\mathrm{ctk}}}\right)f_{\mathrm{ck}}\right)^{\frac{1}{3}} b_{\mathrm{s}}\,d_{\mathrm{s}} \tag{6}$$

$$V_{\mathrm{cf.FRC}} = 0.12\left(1+\sqrt{\frac{200}{d_{\mathrm{f}}}}\right)\left(100\frac{A_{\mathrm{f}}}{b_{\mathrm{f}}\,d_{\mathrm{f}}}\left(1+7.5\frac{f_{\mathrm{Ftuk}}}{f_{\mathrm{ctk}}}\right)f_{\mathrm{ck}}\right)^{\frac{1}{3}} b_{\mathrm{f}}\,d_{\mathrm{f}} \tag{7}$$

$$V_{\mathrm{c.FRC}} = V_{\mathrm{cb.FRC}} + V_{\mathrm{cf.FRC}} \tag{8}$$

where:

$V_{\mathrm{cb.FRC}}$	shear capacity of the fibre reinforced concrete beam
$V_{\mathrm{cs.FRC}}$	shear capacity of the fibre reinforced concrete slab
$V_{\mathrm{cf.FRC}}$	shear capacity of the fibre reinforced concrete flange
$V_{\mathrm{c.FRC}}$	shear capacity of the fibre reinforced concrete ribbed slab
f_{ctk}	characteristic value of the concrete's tensile strength
f_{Ftuk}	residual flexural strength value of fibre reinforced concrete

The residual flexural strength value (f_{Ftuk}) was calculated using the Rigid model according to MC2010 Chapter 5.6.4 [18]. The results of the analytical calculations can be seen in fig. 4.

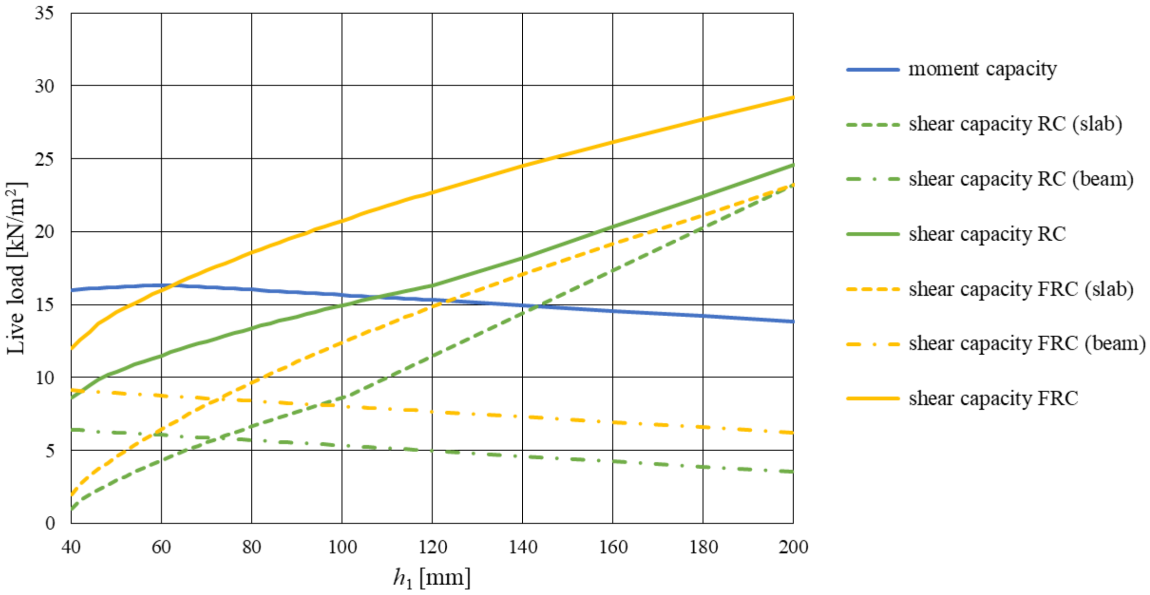

Fig. 4. Analytical results

As it can be seen the moment failure is the ruling failure mode from 200 mm to 100 mm slab thickness in case of RC slabs. Below 100 mm slab thickness the shear is the critical failure mode. Taking into consideration the effect of the fibre reinforcement, the shear capacity is increasing and the critical point where the failure mode changes moves to 60 mm thickness. The calculation mode does not take into consideration the fibres effect in the moment capacity, however according to studies it has also an effect on it, which could lead to an even higher result.

4. Finite element model

To check the results and the behaviour of the slab with different calculation methods as well, finite element models were prepared to investigate the behaviour of the ribbed slab with different thicknesses. The numerical modelling of the slabs were done with ATENA finite element software [24]. The finite element models of the structures can be seen in fig. 5.

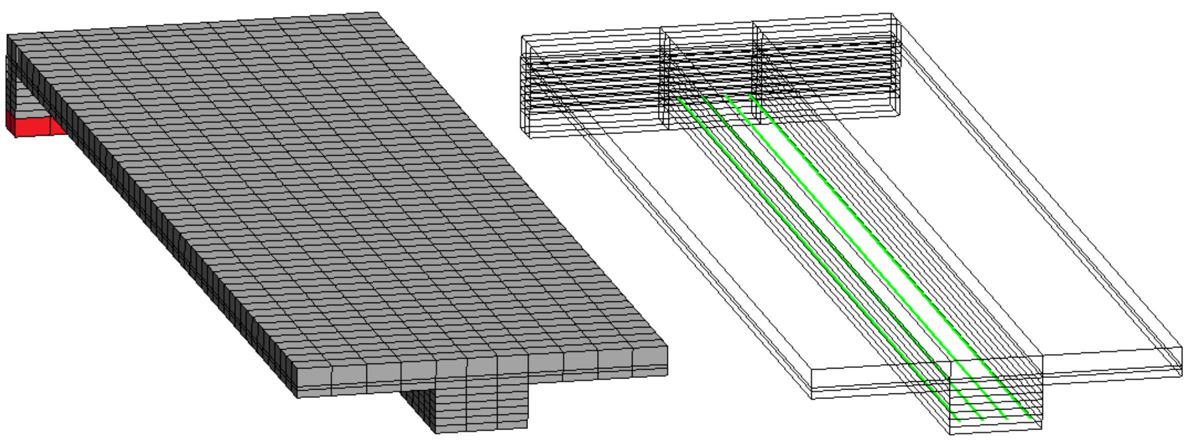

Fig. 5. Slab calculation with ATENA, red: support, grey: PC/FRC, green: steel bars

To ensure that the design model is in connection with the analytical, the same geometry and support conditions were modelled. A one beam and half-half slab was modelled using bi-axial symmetry, to simulate the analytical model while decrease the running time.

The concrete was modelled using an advanced material model using combined failure surfaces. With this material model the different behaviour (elastic-plastic or brittle, compressive and tensile strength, fracture energy) of concrete in tension and compression can be modelled. There are many such models available in the literature, the most used are: Von-Mises and Rankine; Drucker- Prager and Rankine; and Menétrey-William and Rankine (Rankine cube is at the tension side). However, it is important to note that these models only define the peak strength of the material, not the post-cracking response. Numerous other models can be used to approximate the post-cracking capacity of FRC. The model presented in the ITAtech guideline [25] was used here (fig. 6.).

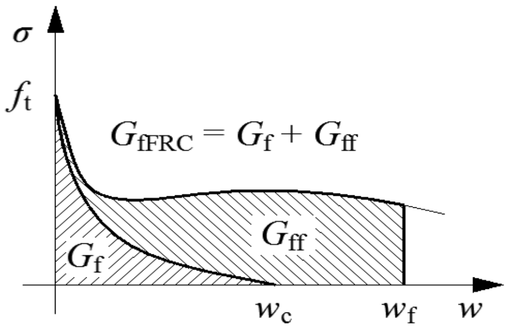

Fig. 6. Fracture energy of the FRC [26]

When stresses exceed the tensile strength of the concrete it will crack. There will be residual stress at the crack surface that depends on the crack width opening distance. This stress is associated with an energy, called fracture energy (G_f). This energy is influenced by the aggregate type (round or crushed), size, and its bond to cement mortar. Fibres increase this fracture energy (G_{ff}), thereby making the concrete a more ductile material. This approach is called the modified fracture energy method [26]. The most important criterion for the selection of the FRC material model is to be able to model this increased fracture energy (G_{fFRC}) and select a value that is appropriate to the FRC used for a design (fig. 6). For our models the additional fracture energy was modelled with a constant residual strength, f_{fdu}, as can be seen in fig. 7.

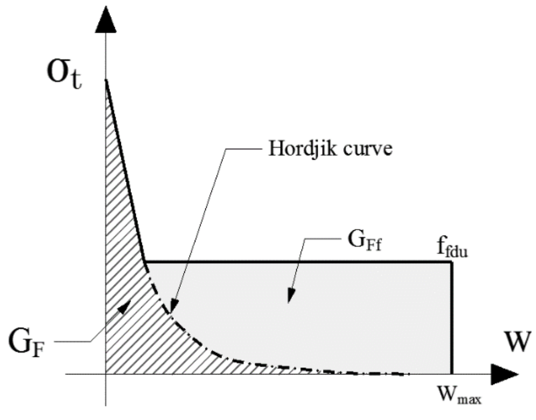

Fig. 7. Used tensile function for numerical calculation

The concrete was modelled as a three-dimensional (3D) brick element with a material model consisting of a combined fracture-plastic failure surface [27]. Tension is handled herein by a fracture model, based on the classical orthotropic smeared crack formulation and the crack band approach. It employs the Rankine cube failure criterion, and it can be used as a rotated or a fixed crack model. The plasticity model for concrete in compression uses the William-Menétrey failure surface [28]. Changing aggregate interlock is considered by a reduction of the shear modulus with growing strain, along the crack plane, according to the law derived by Kolmar [29].

The concrete has a stress-strain relationship according to Eurocode 2 [17]. The crack width was calculated from the stress-crack width diagram, determined by means of inverse analysis, with the help of the characteristic length, which is a function of the size of the element and the angle of the crack within the element. This method is the only one that could realistically represent the cracks in the quasibrittle material. This is the main advantage of this advanced material model. The steel bars were modelled with discrete 1D elements.

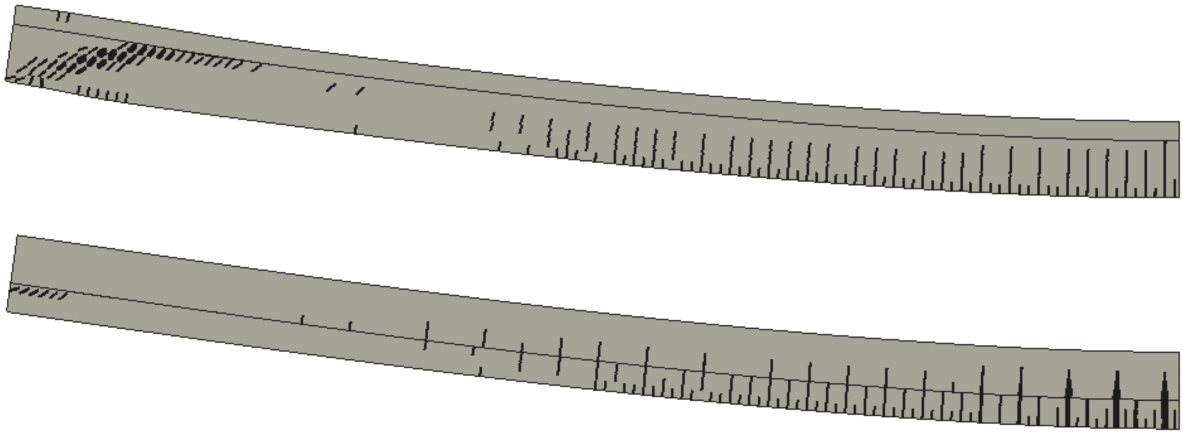

Fig. 8. Typical shear (top) and bending (bottom) failure in finite element model

The results of the numerical models can be seen in fig. 9. In case of plain concrete, the capacity line of the Finite Element models follows the characteristic of the analytical model. It can be seen that the moment failure is the ruling failure mode, while the difference between analytical and numerical mode is less than 10%. The crack propagation in shear and bending failure can be seen in fig. 8.

From the graph of fig. 9. it can be observed that the shear capacity of the structure calculated by finite element method is close to the results of analytical model calculated with formula (4) and (8), where the beam and the flange part was combined.

The failure mode in the FEA changes also at 100 mm slab thickness, up to this point the same increasing effect can be seen in the maximum applied load as in case of the analytical model.

Adding fibre to the numerical model increase also the maximum applied load in all thicknesses. It can be seen that the effect of the fibres is significant also in those thicknesses where the ruling failure mode is bending. Also, it can be seen that the analytical model underestimated the shear capacity for the fibre reinforced concrete section, in FEA the failure was for bending in all thicknesses.

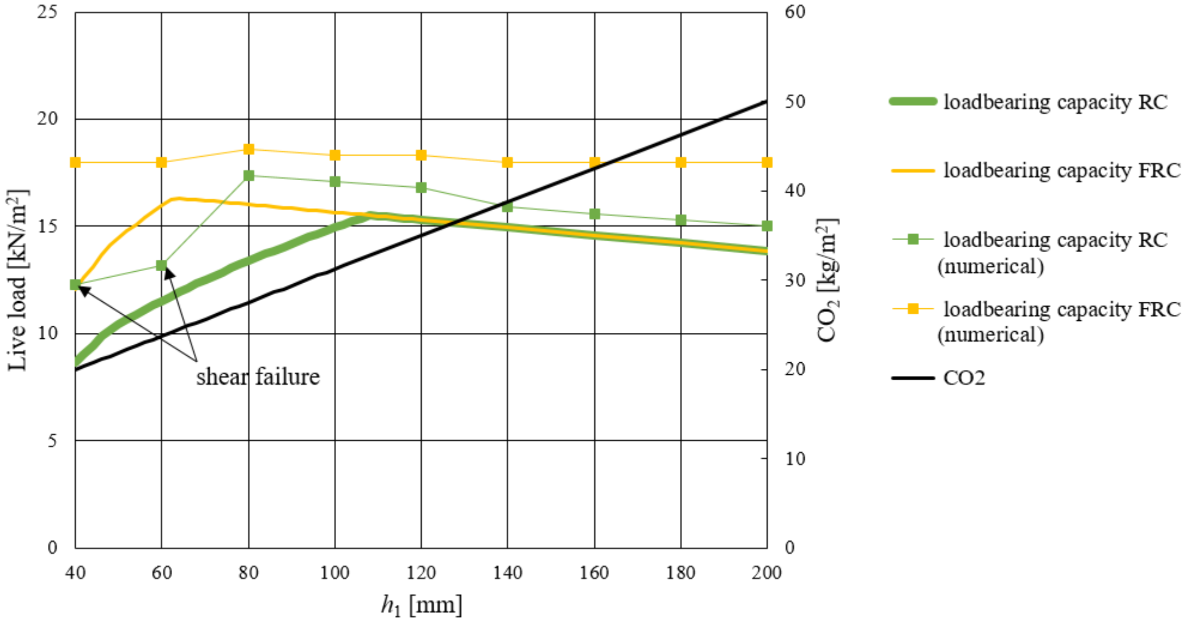

Fig. 9. Results of numerical model and CO_2 emission

5. Conclusion and future research

According to [30] the CO_2 emission of producing 1 m^3 average concrete is ~250 CO_2 kg/m^3. Calculating the loss of weight at different thicknesses the CO_2 emission decrease linearly. It also can be seen, that with plain concrete the optimum slab thickness is ~100 mm, with using fibre reinforcement the ribbed slab structure has the same performance with 40 mm slab thickness as the original flat slab structure, while the CO_2 was decreased with ~30 kg/m^2. This reduction in CO_2 emission is significant even for a 150 m^2 residential house, where the total reduction is 450 kg CO_2 emission, this value is equal to 1800 km driven by an average gasoline-powered passenger vehicle [31]. It also can be seen from the results, that with using an advance method, e.g. the finite element method for the calculation, the results are less conservative than with the traditional analytical formulas.

Due to the high CO_2 emissions of reinforced concrete structures, the optimization of a structure can be the most effective in addition to the geometrical modifications of the structures by the proper choice of the used reinforcing materials and the application of advanced calculation methods. In this paper, a practical example of this was presented, in which a 200 mm thick plate structure was optimized by changing its geometry, using synthetic macro fibres, and using advanced finite element software (ATENA). Based on the presented optimization, it can be seen that even with a simple structure, up to ~50% CO_2 savings can be achieved. During the optimization the beam height was constant 200 mm, however, by increasing the height of the rib and the distance between the ribs, further optimization can be achieved. Along with using synthetic macro fibre reinforcement, additional CO_2 emission reductions could be achieved by using FRP bars to replace the longitudinal steel reinforcement.

The calculation does not consider the additional labour work and CO_2 emissions caused by installing the formwork. The study does not deal with SLS either, i.e., with the issue of deflections and oscillations. Even with this simple structure, there are a number of additional optimization possibilities that should be addressed in the future. The optimization of one structural element can affect the others as well, so it is worth considering the full optimization of a complete building, as well as the resulting total CO_2 savings.

Significant CO_2 savings can only be achieved by applying a combination of the presented changes: i.e., (1) geometric optimization, (2) used material optimization, and (3) optimization from more accurate calculation with an advanced calculation method. In order for this optimization to be truly usable in everyday life, it is important for architect designers to be able to compromise, to use and disseminate new, unproven geometries, and for new materials to be standardized and used responsibly. Extensive knowledge, safe use, and experience with new advanced finite element software. There is still a lot of work to be done for the engineers at each of these points, but we are definitely on the right way.

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