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# Numerical Modelling of High Strength Fibre-Concrete's columns in Multi-Storey Building 

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#### Abstract

The complexity of architecture is continuously growing. Therefore, it is necessary to develop a new construction system with complicated topology using less human work. A fibreconcrete is very promising material to solve those problems. However, design methods of fibreconcrete load bearing structures such as columns are not well developed. Therefore, a new numerical simulation framework is proposed for analysis of fibre-concrete structures macro scale. The method can take into account non-linear post-cracking behaviour of fibre concrete. This includes local fibre orientation in thin elements, anisotropic continuum damage models on different scales, efficient meso-scale fibre orientation prediction tools and others. The proposed method is used to analyse building's columns where part of structural elements is made from fibre-concrete. The effectiveness of fibre-concrete is estimated and proposed a guideline for optimal design using fibre-concrete.


## 1. Introduction

High performance fiber reinforced concrete (HPFRC) and ultra-high performance fiber reinforced concrete (UHPFRC) is becoming more popular in construction industry. However, mechanical behaviour of HPFRC is not well understood, especially when fibers are oriented and material is additionally strengthened with traditional reinforcement bars (re-bars). Additional factors that affect properties of fiber concrete are fiber shape, aspect ratio, volume fraction and properties of concrete itself [1-3]. Meanwhile, design standard EC2 do not include information about design of fiber reinforced concrete [4].

Columns are one of the most important structural elements, especially in high rise buildings. Due to horizontal loads and non-symmetrically loaded structure, a high values of bending stress can appear in columns, that is a cause of tensile stress and cracking [6-9]. The high strength concrete allows to design a relatively thin-walled structures [10-12], therefore columns with rectangular hollow sections are analysed in present work. Thin-walled concrete columns shows a significant influence of fiber's shape, length, orientation on the load bearing capacity of the column or any other structural element [13-14]. Therefore, by analysing and adapting the state-of-art numerical modelling techniques [15-20], there are created a numerical model for the analysis and optimization of rectangular columns. Numerical model is validated with experimental results. The guidelines for optimal design of fiber reinforced high strength concrete's columns is proposed according to numerical results.

## 2. Model validation at compressive stress

The improvement in the strength of the concrete peak from the use of fibres is usually insignificant, but the use of fibres reduces the fragility of high-performance concrete and significantly improve the deformative properties of the concrete [1-2]. The developed in the new numerical simulation framework finite element model allows appreciating the improvement of deformative properties of concrete from the use of fibres by defining the stress-strain curve of the respective material. The developed 3D and 1D numerical models which considered non-linear behaviour of high-performance steel fibre reinforced concrete (HPSFRC) material are validated by the experimental data from the publication [3] for column with pinned supports at both ends, which is subjected to combined action of compression and bending with additional longitudinal reinforcement.

The validation of model is carried out for the column with length $\mathrm{L}=1300 \mathrm{~mm}$ and external sizes $125 \times 125 \mathrm{~mm}$. Column has B550 class 4 longitudinal bars with a diameter of 8 mm , located in the corners of the columns. The compressive strength of the used concrete from the test results is $67-72 \mathrm{MPa}$. The eccentricity with which the columns are loaded is $\mathrm{e}=50 \mathrm{~mm}$. The high strength concrete of C60 /75 [4] class is used for calculation. Consequently, fcm $=68 \mathrm{MPa}$ has been accepted for the compressive strength of the high-performance concrete and 4.4 MPa for mean tensile strength. Compression stressstain values for HPSFRC and high-performance concrete (HPC), which is shown in figure 1 are taken according the typical stress-strain diagrams of steel fibre concrete [3].


Figure 1. Stress-strain curve for high-performance concrete and high-performance concrete with $1.0 \%$ RC $65 / 35$ BN-type steel fibres [3].
The column 3D model is designed for one-half the length of the column to reduce the time of calculation by using symmetry conditions. Collapse scheme of the 3D design model of column specimen and typical failure mode of the column specimen from the experimental test are shown in figure 2. As it can be seen, there is similar character of the failure scheme. The first cracks are formed in the middle of the column span.


Figure 2. Failure scheme of column specimen: (a) from experiment [3]; (b) from calculation; (c) with max compression stress.

Load-displacement curves of column specimen subjected to the compression with flexure, according to laboratory tests from publication, and developed 3D and 1D numerical model calculations are summarized in figure 3.


Figure 3. Load-displacement curves for column specimen.
According to the results obtained, the crushing axial load obtained from the laboratory test was 176 kN , while the calculated by 3D and 1D non-linear numerical models crushing axial load was 172.5 kN and 163.5 , respectively. The difference between design and experimentally determined axial load for 3 D model is $2.0 \%$, for 1 D model $-7.1 \%$. The behaviour of the calculation models is like the actual operation of the column specimen and developed models are safe for prediction of the behaviour of
high-performance steel fibre reinforced concrete material. Crushing axial load of the high-performance concrete column according to the experimental data is 159 kN , while the calculated by 1D numerical model it was 151.5 kN . The difference between design and experimentally determined axial load for 1D model is $5.0 \%$.

## 3. Optimal design of HPSFRC

The optimal design is described for column structure, which is shown in figure 4, subjected to the compression and bending. The columns cross-sections are selected as a box-type to provide the decreased consumption of non-renewable materials and high stiffness of the element in the both planes. The box-type cross-section enables the integration of engineering communication inside of columns. Column cross-section is with constant $500 \times 500 \mathrm{~mm}$ external dimensions and variable wall thickness.


Figure 4. Details of column structure: (a) column 3D numerical model with meshing; (b) design scheme of the column.

Columns with the height $(\mathrm{H})$ from 3.5 to 4.5 m are chosen as a region of interest for the optimization. Material consumption of the high-performance fibre reinforced concrete and steel of the longitudinal reinforcement are regulated by optimization input variables $t$ - wall thickness of the box-type column cross-section with limits from 5 to 12 mm and $\mathrm{A}_{\mathrm{s}}-$ area of the longitudinal steel reinforcement in column cross-section with limits from 0 to $1.5 \%$ of concrete area of the column cross-section.

To explore the relationships between several variables, such as column height and material consumption of the steel of the longitudinal reinforcement and high-performance concrete, and response variable - load-bearing capacity of the column, response surface methodology is used.

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### 3.1. Experimental design

There are 3 input variables with 3 levels of each factor, so the total number of experiments is 27 . The experimental design that defines which experiments should be carried out in the experimental region is based on Box-Behnken design.

Each of the experiment is calculated to determine load-bearing capacity of the column as function $F, M=f\left(H, t, A_{s}\right)$. There is a relationship between axial force F and bending moment $\mathrm{M}: M=F \cdot e$, where $e$ - eccentricity, which is equal to the distance from the centre of the column cross-section to the centre of the box-type cross-section wall of the column. This relationship is based on the simplified design scheme of the multi-storey building, where one of the ends of the column is fixed, and loads are transferred from each storey with eccentricity by beams.

Simulations of analysed process based on experimental design are made to get necessary statistic data set of column load-bearing capacity, which makes regression analyses possible.

### 3.2. Calculation of the experiment

For the calculation of each experiment is used column 3D and 1D numerical models which considered non-linear behaviour of high-performance steel fibre reinforced concrete material. The behaviour of non-linear material is characterized by degradation of Young's modulus cause of cracking. Damage variable is used for the prediction of behaviour of high-performance steel fibre reinforced concrete [5].

Calculations with the 3D numerical model are complex and time-consuming, therefore verification of the 1 D numerical model of the column with fixed support at one end, which is subjected to the compression and flexure has been done by comparison of the results of calculations with both models.

Calculations of 3 experiments with 3D and 1D model have been made. Verification of the 1D model is made by comparison of crushing axial load values or load value at displacement value H / 250, depending on which value is reached first.

According to the results obtained, the load value at displacement value $\mathrm{H} / 250$ calculated by 3D and 1 D non-linear numerical models for column with $\mathrm{H}=3.5 \mathrm{~m}, \mathrm{t}=5 \mathrm{~mm}$ and $\mathrm{As}=0.2 \%$ is 353.9 kN and 342 kN , respectively; for column with $\mathrm{H}=4 \mathrm{~m}, \mathrm{t}=12 \mathrm{~mm}$ and $\mathrm{As}=0 \%$ is 703.0 kN and 682 kN and for column with $\mathrm{H}=4.5 \mathrm{~m}, \mathrm{t}=8.5 \mathrm{~mm}$ and $\mathrm{As}=1.5 \%$ is 823.8 kN and 805 kN . The difference between axial load values determined by 3 D and 1 D models is between $2.3 \%$ and $3.4 \%$. Results of verification showed, that used 1D finite element model describes with sufficient accuracy the behaviour of the 3D numerical model of the column subjected to the compression and flexure, therefore 1 D model is used for the further calculations of the experiments.

Results of calculations of 27 experiments shown, that dependence of load-bearing capacity value from column height in region of interest for column heights $3.5-4.5 \mathrm{~m}$ is negligible, because of load bearing capacity is equal with load value at displacement value $\mathrm{H} / 250$, which is $14-18 \mathrm{~mm}$, in most of the experiments.

The comparison of the load-bearing capacity of the column with height equal to 4 m from highperformance steel fibre concrete (HPSFRC) and from high-performance concrete (HPC) with stressstrain values according to figure 1 is summarized in table 1 .

Table 1. Comparison of load-bearing capacity of the 4 m height column from HPSFRC and HPC.

| $\mathrm{t}, \mathrm{mm}$ | 5 | 8.5 | 12 | 5 | 8.5 | 12 | 5 | 8.5 | 12 |
| :--- | :--- | :--- | :--- | :--- | :--- | :--- | :--- | :--- | :--- |
| As, $\%$ | 0 | 0 | 0 | 0.2 | 0.2 | 0.2 | 1.5 | 1.5 | 1.5 |
| $\mathbf{F}_{\text {HPSFRC }}, \mathbf{k N}$ | 320 | 512 | 682 | 340 | 547 | 735 | 525 | 847 | 1145 |
| $\mathbf{F}_{\text {HPC }}, \mathbf{k N}$ | 309 | 492.5 | 656 | 320.5 | 512.5 | 685 | 445 | 740 | 1025 |
| $\mathbf{\Delta}, \mathbf{\%}$ | 3.4 | 3.8 | 3.8 | 5.7 | 6.3 | 6.8 | 15.2 | 12.6 | 10.5 |

Graphs of the dependence of column with height 4 m load-bearing capacity on the wall thickness of the box-type column cross-section $(\mathrm{t})$ with different $\mathrm{A}_{s}$ values are summarized in the Figure 5.


Figure 5. The dependence of column load-bearing capacity ( F ) on the wall thickness ( t ) with different area of the longitudinal steel reinforcement in column cross-section as a percentage of the concrete area $\left(\mathrm{A}_{\mathrm{s}}\right)$.
Graphs of the dependence of column with height 4 m load-bearing capacity on the area of the longitudinal steel reinforcement in column cross-section as a percentage of the concrete area $\left(\mathrm{A}_{\mathrm{s}}\right)$ with different $t$ values are summarized in the Figure 6.


Figure 6. The dependence of column load-bearing capacity ( F ) on the amount of reinforcement $\left(\mathrm{A}_{s}\right)$ with different wall thickness of the box-type column cross-section ( t ).

## 4. Conclusions

Possibility to increase load-bearing capacity of the high-performance concrete columns subjected to compression and flexure by adding steel fibres was checked and confirmed by calculations. The use of high-performance steel fibre concrete as column material is especially effective for columns with
additional longitudinal reinforcement where load-bearing capacity is up to $15 \%$ bigger in comparison with high-performance concrete columns with the same cross-section.

The difference in the ultimate load between the calculated and experimental results is $2.0 \%$ for 3D model and $7.1 \%$ for 1D model. The linear behaviour of the 3D model is identical to the experimentally obtained of the column, the failure mode of the column specimen and the plastic behaviour of the material is similar in both cases, which leads to the conclusion that the developed constitutive model of material behaviour with sufficient precision describes the behaviour of the column subjected to the compression and flexure.

The developed optimization method shows, that difference of the column load-bearing capacity for column height between 3.5 and 4.5 m is small. The obtained results of experiment calculations allow predicting the load-bearing capacity of column and regulating material consumption of the highperformance fibre reinforced concrete and steel for additional longitudinal reinforcement.

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