3D Nonlinear Analysis of Precast Prestressed Hollow Core Slab

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Abstract. In the last decades, precast prestressed hollow core slabs have gradually increased their market presence in many countries due to their excellent structural performance at room temperature. Longitudinal voids through the one way prestressed (or reinforced) concrete slabs are very important for reducing the self-weight of the structure. Although the hollow core precast concrete is larger than on-site placement concrete, when viewing the entire construction of the building as a whole, the use of hollow core precast concrete contributes more to the reduction of energy consumption than half-precast concrete or on-site placement concrete. Precast prestressed hollow core slabs are among the more advanced structural floor systems for all kinds of buildings. This paper presents results from nonlinear numerical analysis of precast prestressed concrete hollow core slabs with dimensions 13,9m length, 1,19m width and 300mm thickness. Material properties obtained from testing of concrete (concrete cube strength) and 7 wire pre-stressing strands (yield strength) were used as input data for nonlinear simulation. The ATENA 3D FEM software was used for nonlinear numerical modelling. The results from numerical are compared with data from experimental investigation of a total 7 hollow core slabs.

1. Introduction

The advantages of hollow core slabs are not only in the production technology, which is nearly fully automated, but also in others features such as optimum used of materials, slenderness of construction, environmental friendliness etc. Compared to common concrete floors, hollow core floors can save 50% concrete and 30% steel for the same performance. The use of hollow core precast concrete slab reduces the amounts of generated harmful substances and waste. In addition, the use of precast concrete members can contribute to the mitigation of energy consumption, and noise and vibration generated at the construction site resulting from reduction of transportation frequency [1].

The design of precast member according to Eurocode 2: Design of concrete structures, Part 1 General rules and rules for buildings [2]. is based firstly on the requirements of serviceability limit state, since the fundamental aim of prestressed concrete is to limit tensile stresses, and hence flexural cracking. For members with bonded tendons (such as prestressed hollow core slabs), the criterion for the limit state of crack width depends on exposure class of the member. The criterion for all exposure class (except X0 and XC1) and under quasi-permanent load combination is decompression. This means that all the tendons lie at least 25 mm within the compression zone. This requirement is related...
with protection of tendons against reinforcement corrosion, because the surface layers of the tendons are higher quality and in case of reinforcement corrosion, these layers degrade as first [3].

2. Experimental measurements
For numerical modelling, data obtained from previous experimental investigation of 13.9m long hollow core slabs were used. The purpose of this research was to investigate behaviour of hollow core slabs under static loading, development of the cracks and on not least also their deflection. The dimensions of testing members were 13900mm length, 1190mm width and 300mm height. The cross-section of the studied specimens is shown in Figure 1. Each specimen was prestressed with thirteen prestressing tendons, eleven on the bottom face and two on the top face of the member. Prestressing tendons were composed of 7-wire, low-relaxation strands with a 12.5 mm diameter and a cross-sectional area of 91.25 mm².

2.1. Materials
2.1.1. Mechanical properties of hardened concrete
The control samples (cubes and prisms) were cast to determine the concrete compressive strength and the modulus of elasticity of the concrete. These tests were performed on 28 days after concreting. The test samples have been manufactured and tests have been made according to the set of standards STN EN 12390. According to standard STN EN12390-1 [4], the compressive strength at 28-day was defined by applying (150x150x150 mm cube specimens) and experiment by the digital loading device. The modulus of elasticity test was conducted on prism of 100 mm×400 mm. The test results are shown in Table 1.

Table 1. Mechanical properties of concrete

| Sample number | Compressive cube strength [MPa] | Modulus of elasticity [GPa] |
|---------------|---------------------------------|----------------------------|
| P1            | 52.0                            | 38.5                       |
| P2            | 58.0                            | 41.1                       |
| P3            | 53.0                            | 38.9                       |
| P4            | 50.0                            | 37.5                       |
| P5            | 47.5                            | 36.3                       |
| P6            | 49.3                            | 37.2                       |
| P7            | 50.7                            | 37.9                       |
| Average       | 51.5                            | 38.2                       |
2.1.2. Mechanical properties of prestressing tendons

Two numbers of 12.5 mm seven wire strand (6x4 mm + 1 x 4.5 mm) low-relaxation tendons were tested. At one end were the strands anchored and stressed from the other end. The average ultimate tensile strength and modulus of elasticity was found to be 1873 MPa and 212.28 GPa respectively. The results for both tendons are shown in Table 2.

| Sample number | Tensile strength [MPa] | Modulus of elasticity [GPa] |
|---------------|------------------------|-----------------------------|
| 1             | 1851                   | 206.40                      |
| 2             | 1894                   | 218.17                      |
| Average       | 1873                   | 212.28                      |

The tested slabs were supported at two points (flanges of I-shape beams) and loaded with fourth increasing concentrated forces F supplied by hydraulic jacks. Force was ascending symmetrically on both face from axis of symmetry. Testing arrangement is shown on Figure 2 and Figure 3. The load was applied in increments of 5 kN.

![Figure 2. Testing arrangement – 3D visualisation](image)

![Figure 3. Testing arrangement](image)

3. Numerical modelling

The finite elements analysis included modelling of the prestressed hollow core slabs tested. The ATENA 3D FEM software was used for nonlinear numerical modelling. Due to symmetrical shape and for decrease of numerical solution, only 1/2 of specimens were modelled (Figure 4). Prestressing of tendons is applied in an unloaded stage. Tendons are ten fully bonded, and loading is applied.
Therefore, under loading the deformation of the cables is fully compatible with that of concrete. The geometry of the slab is shown in Figure 3 and the cross section of the slab is shown on the bottom side of the figure. The loading points and support are equipped with steel plates and frames to avoid stress concentration and local failures.

3.1. Concrete
Atena 3D Nonlinear Cementitious 2 material model for the concrete was used for modeling of the concrete part of specimen. Fracture-plastic model combines constitutive models for tensile (fracturing) and compressive (plastic) behavior. The fracture model is based on the classical orthotropic smeared crack formulation and crack band model. It employs Rankine failure criterion, exponential softening, and it can be used as rotated or fixed crack model. The hardening/softening plasticity model is based on Menétry-Willam failure surface. The model uses return mapping algorithm for the integration of constitutive equations. Special attention is given to the development of an algorithm for the combination of the two models. The combined algorithm is based on a recursive substitution, and it allows for the two models to be developed and formulated separately. The algorithm can handle cases when failure surfaces of both models are active, but also when physical changes such as crack closure occur. The model can be used to simulate concrete cracking, crushing under high confinement, and crack closure due to crushing in other material directions [6]. Concrete is assumed with average material parameters listed in Table 1.

Tetrahedral 3D Solid Elements was chosen for FE mesh of concrete part of hollow core slab. The element has fourth (CCIsoTetra<xxxx>) or ten (CCIsoTetra<xxxxxxxxxx>) nodes with three degrees of freedom at each node (translations in the nodal x, y, and z directions) – see Figure 5. These are isoparametric elements integrated by Gauss integration [6]. This means that for each element there are eight integration points.

Figure 4. Finite element model and cross section for prestressed hollow core slab

Figure 5. Geometry of CCIsoTetra elements.
3.2. Prestressing tendons
Prestressing reinforcement is modelled as elastic perfectly plastic material. Tendons are assumed with average material parameters listed in Table 2. For prestressing reinforcement, the multilinear Stress-Strain laws was chosen with data from stress-strain testing of tendons material.

Brick 3D Solid Elements was chosen for FE mesh of tendons. These elements have eight (CCIsoBrick<xxxxxxxxx>) or twenty (CCIsoBrick<xxxxxxxxxxxxxxx>) nodes with three degrees of freedom at each node (translations in the nodal x, y, and z directions) – see Figure 6. Simirarily to the Tetrahedral, too Brick elements are integrated by Gauss integration.

![Figure 6. Geometry of CCIsoBrick elements.](image)

3.3. Loading frame and support plate
The loading frame and support steel plates are assumed to remain elastic, with Young’s modulus 210 GPa and Poisson’s ratio 0.3. These parts of nonlinear analysis are modelled using Atena 3D Elastic Isotropic material model. Brick 3D Solid Elements was chosen for FE mesh.

3.4. Loads and supports
Bottom supports are prescribed as the load case LC1 in all steps of the analysis. The first loading step corresponds to prestressing. In this step, prestressing force of 0.171 MN is applied to all thirteen reinforcement lines (LC3). Then, vertical loading is applied by prescribing vertical displacement at the upper loading plates in constant increments of 1 mm. For numerical solution, the Newton-Raphson method was employed. The overall response is recorded at four monitoring points: loading at the top of the loading frame, the reaction at the point of the support and deflections at the location, where loading of the sample is applied.

4. Discussion
Comparison of the load versus vertical displacement from the test (specimen no. S3, S4 and S6) and the finite element analysis are shown in Figure 7. As can be seen, the finite element analyses were able to capture the deflection behaviour a reasonably good agreement. Results of comparison between experimental work and numerical analysis in deflection value are in the range 6.5% to 9.6% (6.5% for specimen S3, 9.6% for specimen S4 and 8.5% for specimen S6). The deflection obtained from numerical analysis are higher than results from experimental measurements.

As shown in Figure 7, the maximum vertical displacement from the experimental test at the same load level of F=90 kN is 281.7 mm for S3, 298.8 mm for S4, and 295.5 mm for S6 respectively. In the numerical analysis, at the load F =90N the reached deflection of hollow core slab is 241 mm. Maximal loading F=93.2 kN of specimen was reached at deflection 260.3 mm before creep of reinforcement. The numerical calculation was stopped at step 48 at vertical displacement 290.8 mm, when one of used
materials of specimen (concrete or tendons) were corrupted. Deflection contours obtained from finite element analysis are shown in Figure 8.

![Figure 7. Comparison of experimental and numerical results.](image)

![Figure 8. Vertical displacement of slab from the analysis after maximum load.](image)

The software is capable to indicating the crack propagation. First crack of with width more than 0.1mm was recorded at loading step 15 (Figure 9). The initial cracking of the slab in the FE model corresponds to a load of F=66.3kN as shown in Figure 8. This first crack occurs in the constant moment region and is a flexural crack. According to results from experimental investigation, first crack with w=0.05mm width was found at a load F=61.4kN on specimen S3. For the specimen S4 and S6, the first crack with a width w=0.05mm was found at a load F=50.4kN. Vertical displacement of
the tested slab at the time of first crack formation was 61.2mm for specimen S3, 42mm for specimen S4 and 39.6mm for specimen S6 respectively.

**Figure 9.** Crack pattern from the analysis of slab after loading step 15 – bottom side view

**Figure 10.** Crack pattern from the analysis of slab after maximum load.

5. **Conclusions**
This paper presents results from the first part of the project that deals with finite element analyses of hollow core units. The results from the analyses are compared with those from the corresponding tests. Nonlinear finite element analysis of hollow core units, subjected to loading, were carried out. Results from numerical solutions were compared to results obtained from experimental investigation. By comparing numerical and experimental results, regarding the behavior of hollow core slab under loading, a reasonably good agreement was obtained. Regarding to deflection of hollow core slab, the results obtained from numerical analysis were higher than results from experimental investigation results circa 6.5% for specimen S3, 9.6% for specimen S4 and 8.5% for specimen S6.

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