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Josef Hegger, Norbert Will, Sebastian Bülte

Prestressed filigree floors for domestic construction

1 SCOPE OF THE RESEARCH PROJECT

At present, spans of floor systems for housing usually range from 3 to 6 m. Thereby, the depth of the floors averages between 14 to 20 cm. Because of the limited span a multiplicity of bearing walls is required, which does not allow future changes in the floor plan without substantial effort. A flexible floor plan can be realized through longer spans. In this case no changes of load-bearing members are required for alterations.

Within the research project [1] design rules for prestressed floor systems which combine precast filigree floors made of high-strength concrete with normal-strength site-mixed concrete were derived on the basis of theoretical and experimental investigations. The project perpetuated the investigations of [2]. The adoption of modern production systems and new building materials offers very rational methods of construction as well as a great flexibility. The research report was donated by means of the federal office for building and regional planning (file references: Z 6 - 5.4.00 - 15/II 13 - 80 01 00-15).

2 REALISATION OF THE PROJECT

2.1 General remarks

At the Institute of Structural Concrete of the Aachen University 4 long-term deflection- and 13 load-capacity tests of prestressed filigree floors with a structural concrete topping were accomplished. The experimental investigations were supplemented by numerical simulations and parameter studies. The load-carrying behaviour was evaluated for both the erection state and the final state. Based on the results a complete design concept was derived.

2.2 Experimental tests

The experimental investigations were conducted on nine prestressed filigree slabs made of high-strength concrete. Seven of these slabs were topped with normal-strength concrete. The chosen cross sections are shown in fig. 1. Besides evaluating the prestress release of all slabs, the deflection under service load as well as the fracture behaviour in the ultimate limit state were determined for four slabs (type 1). The influence of transverse cavities on the load-bearing capacity was tested on the other three slabs with structural concrete topping (type 2). The carrying behaviour of the floor system during construction was determined by load tests on two slabs without a concrete topping (type 3).

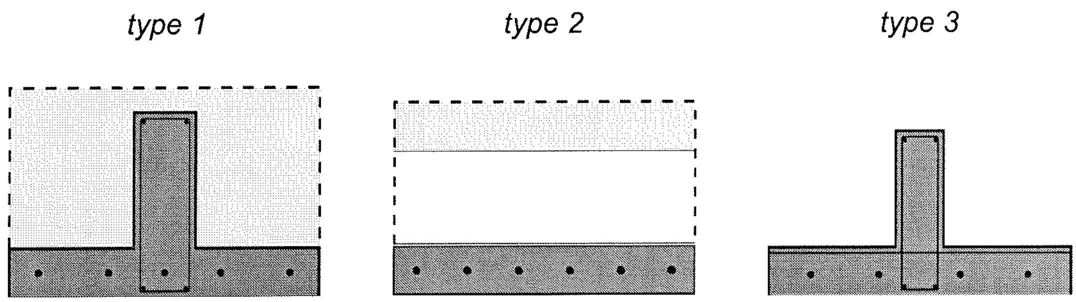


Fig 1: Cross sections

2.3 Numerical simulations

The deflection depends on a multiplicity of influence factors, such as the prestressing load, the geometry as well as the load history. Therefore, it can't be described just by test results and specific numerical simulations are required. In order to analyse the individual parameters the program QKS was developed which allows the computation of the deflection regarding the time-dependent material properties. If the number of strands with a prestressing of 1000 N/mm² is designed to resist the ultimate limit state without further reinforcement, no pronounced cracking under service loads has to be expected. The program QKS was calibrated on the basis of test results. The calculations showed a good agreement with the measured values.

Simulations of the load bearing capacity of the filigree floors with transverse cavities were also conducted. The results were used to describe the stress transfer. The simulations were done with the program LIMFES [3].

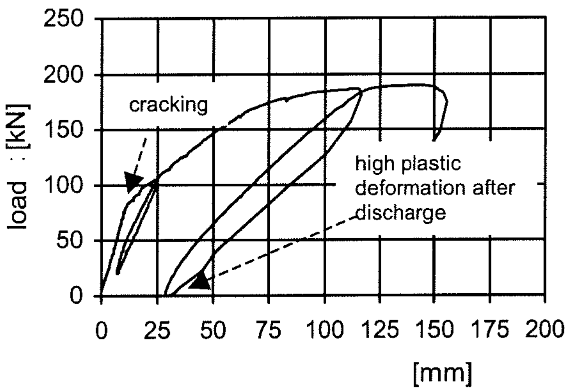
3 TEST RESULTS

3.1 General remarks

Without any additional reinforcement, filigree floors made of high-strength concrete in the tensile zone and normal-strength concrete in the compression zone fulfill the requirements of the load-carrying behaviour for the serviceability limit state (SLS) as well as ultimate limit state (ULS). The derived design procedures extend the existing design guidelines, confirm the suitability of the presented construction method and allow long-span floor schemes. The results will resumed below.

3.2 Flexural Capacity in Final State

As the rectangular filigree slabs in [2], the slabs with an upper web (fig. 1; Type 1) showed a load-bearing behaviour comparable to monolithic prestressed concrete slabs. The requirements for the alligatoring under service load and the ductility in the ultimate limit state are fulfilled by the combination of high-strength concrete in the prestressed tensile zone and predominantly normal strength concrete in the compression zone (fig. 2).



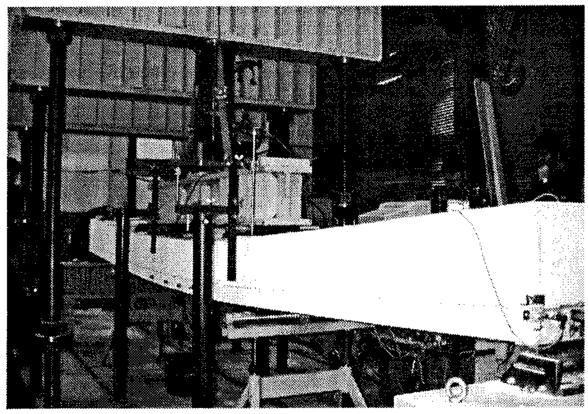


Fig. 2: Load-deflection diagram and loaded filigree slab with structural topping

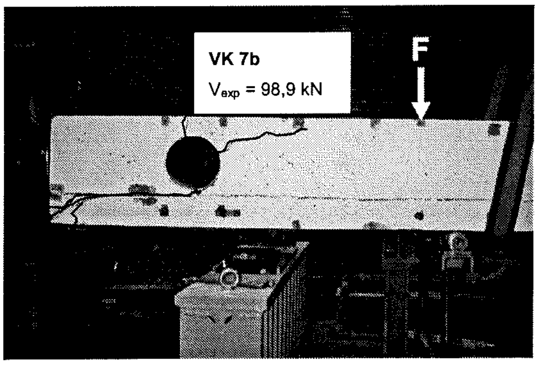
The contact zone between the precast slab and the in-situ concrete topping was untreated and without stirrups. The measured shear capacity of the joint exceeded the calculated capacity according to DIN 1045-1 [4] substantially. The test showed that the interface is not critical for the design of regular slabs, although the calculation according to DIN 1045-1 predict a premature failure at this point. Based on the load-bearing tests the presented floor system can be assumed monolithic.

3.3 Shear Capacity in Final State

The measured load-bearing capacity of the slabs with transverse cavities (type 2) was significantly lower than the calculated load based on DIN 1045-1. This reduction can be explained by the effects of the openings. At the moment of the shear crack initiation, the failure of the slabs occured suddenly. Due to the natural stress transfer, the shear stress increased close to the cavities (fig. 3). Thereby, the shear stress depended on the diameter of the opening. Since the maximum stress arose only within a small area and since this stress could be transferred to less strained areas, a median stress is sufficient for the specification of the increased shear stress. It is suggested to consider the increase of the stress close to the openings by the ratio of the size of the openings to the slab depth. This procedure follows the recommendations of [5]. The magnification factor $1/\alpha_r$ can be determined as follows:

$$\alpha_r = \frac{h}{(h + d_{cav})} \tag{1}$$

where: h slab depth
d_{cav} diameter of the cavity



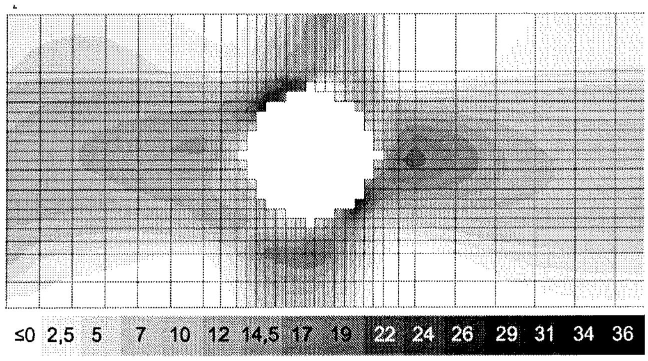


Fig. 3: Crack pattern of test VK 7b and stress distribution [N/mm²] close to the cavity when shear crack initiates

In other shear tests without cavities ([2],[5]) it was observed that the load-bearing capacity is approximately 20 % higher than the load, when the shear crack initiate. This results from the activation of a truss load-bearing system. Slabs with cavities in the structural concrete topping don't show this increased shear capacity, since the trusses cause a tensile load in the contact zone of the precast and the in-situ concrete. Tension in the interface leads to an immediate failure of the slab. Thus, the design capacity has to be limited to 80 % of the load-carrying capacity of monolithic slabs. The load-bearing capacity averages:

$$V_{calc,red} = 0{,}8 \cdot \alpha_r \cdot V_{calc} \qquad (2)$$

where: V_{calc} load-carrying capacity based on DIN 1045-1 [4]
α_r reduction factor for cavities

Hereby, the load-bearing capacity can be predicted easily. Fig. 4 shows the good agreement between the experimental failure load V_{exp} and the calculated one V_{calc} with the ratio V_{calc}/V_{exp} being always greater than 1. The reliability of the calculated capacity equals the one of prestressed filigree floors without cavities [2] and prestressed monolithic concrete beams [6].

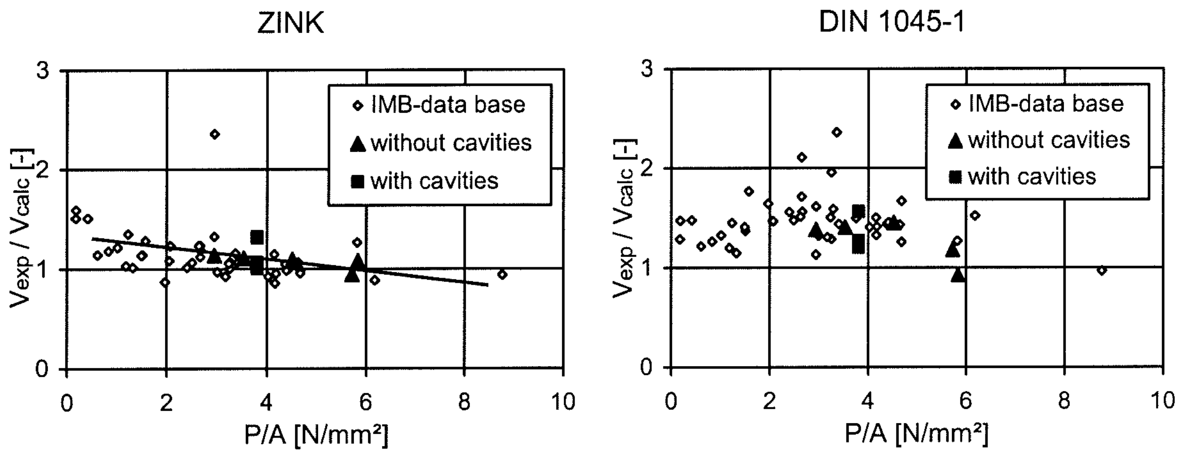


Fig. 4: Comparison between experimental load-bearing capacity V_{exp} and the reduced value V_{calc} (eq. 2) based on the procedures of Zink [7]and DIN 1045-1 [4]

In order to avoid immoderate superposition of stress peaks between cavaties a minimum clearance a greater than h has to be regarded.

3.4 Load-bearing Capacity during Erection

The load-bearing tests on the precast filigree slabs without structural concrete topping (fig. 1, type 3) showed a crack initiation in the lower flange of the ⊥-section close to the support before reaching the flexural capacity. This cracking results from the superposition of the shear stress between web and flange and of the radial tensile stress caused by the prestress release.

The anchorage of the tensile stress of strands has to be proven along the anchorage zone as shown in fig. 5. Due to the cracking within the anchorage zone only half of the uncracked bond strength may be considered [4]. Longitudinal cracks are to be excepted.

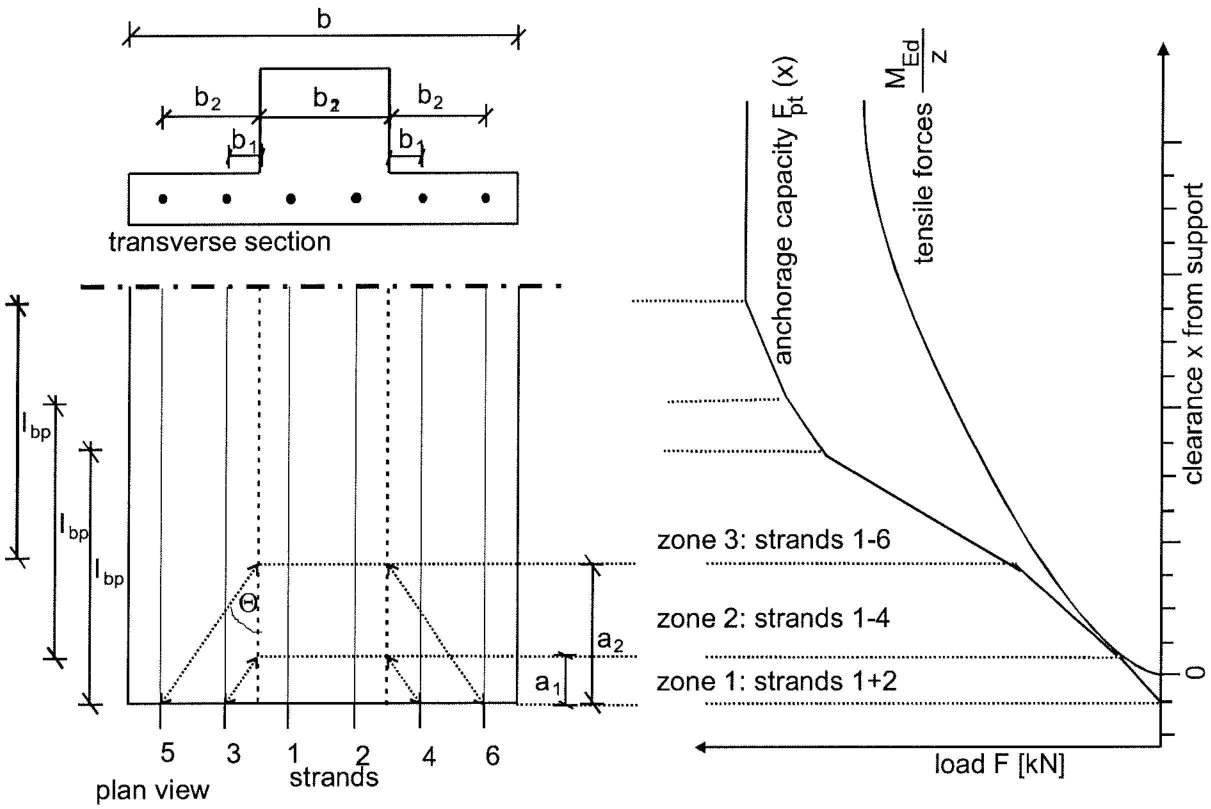


Fig. 5: Diagrammatic procedure to prove the anchorage of the tensile stress of a strand

At the support only the strands below the web can be taken into account for the anchorage of the tensile forces (strands no. 1 and no. 2). Starting from distance a_1 the strands no. 3 and no. 4 are attached to the web and contribute to anchorage. The distance a_1 depends on the the clearance b_1 of the strands to the web and on the strut angle θ which was determined by the tests to be 35°. The contribution the other strands to the anchorage is accordant:

$$a_i = \frac{b_i}{\tan 35°} = 1{,}43 \cdot b_i$$

In order to prevent a premature anchorage failure due to shear cracking, transverse reinforcement is required to connect the flange to the web. The transverse reinforcement has to be designed according to DIN 1045-1 [4].

3.5 Deflection

The use of high-strength concrete and the strong prestressing forces allow slim floor constructions without exceeding the permissable deflection. Common design procedures were developed (e.g. [8]) on behalf of monolithic construction units. Hereby, the deflection due to creep and shrinkage is not sufficiently considered. Precast slabs with structural concrete topping are imposed to an additional stress which results from the different time-dependent material behaviour of the two concrete types. The precast unit of high-strength concrete retards the contraction due to shrinkage of the concrete topping. Unopposed creep and shrinkage can be described by an equivalent axial force Q_{eq}. Caused by the precast unit this axial force leads to an additional bending moment of the overall cross section (fig. 6).

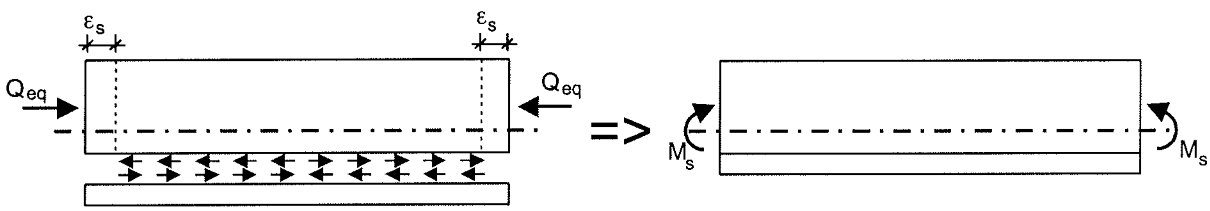


Fig. 6: Loading of the filigree slab due to shrinkage of the concrete topping

Accordant to the investigations on rectangular filigree slabs without web in [2], the deflection behaviour of 4 ⊥-shaped precast filigree slabs made of high-strength concrete with a structural normal-strength concrete were accomplished. The significant difference to the rectangular sections was the eccentric pretensioning which leads to a negative deflection of the precast unit before the structural topping is added.

In order to evaluate the results of the parameter studies of both the final deflection and deflection increase after loading, the effective flexural rigidity EI_{eff} of the section was introduced. This rigidity arises from the deflection of a single span after 20 years:

$$EI_{eff}(t = 20a) = \frac{M_{quasi-permanent} \cdot l^2}{9{,}6 \cdot f(20a)} \tag{3}$$

Thus, the specific flexural rigidity α can be expressed by the ratio of the effective rigidity EI_{eff} and the nominal rigidity EI_0 of the cross section.

$$\alpha = \frac{EI_{eff}}{EI_0} \tag{4}$$

The relation $(1/\alpha - 1)$ expresses the increase of the deflection over 20 years compared to the elastic deflection of an uncracked monolithic single span with adequate dimensions. An increased value α means a reduced time-dependent loss of the flexural rigidity.

The influence of each single factor was described by a function based on statistic analysis of the simulation results. Regarding load and geometry of the member these functions were defined separately for the final state and deflection increase after loading. The functions for the different characteristics were multiplied to allow the computation of the specific rigidity α of any slab in consideration of all factors. After calculating the specific flexural rigidity α the required depth of the slab can be read from nomographs to prevent an excessive deflection. There are different nomographs for the two types of the precast fliligree slabs (with or without web) and for various quasi-permanent loads. The diagram for a filigree floor with web for a quasi-permanent load of 0,45 kN/m² and a load increase for partitions of 1,5 kN/m².

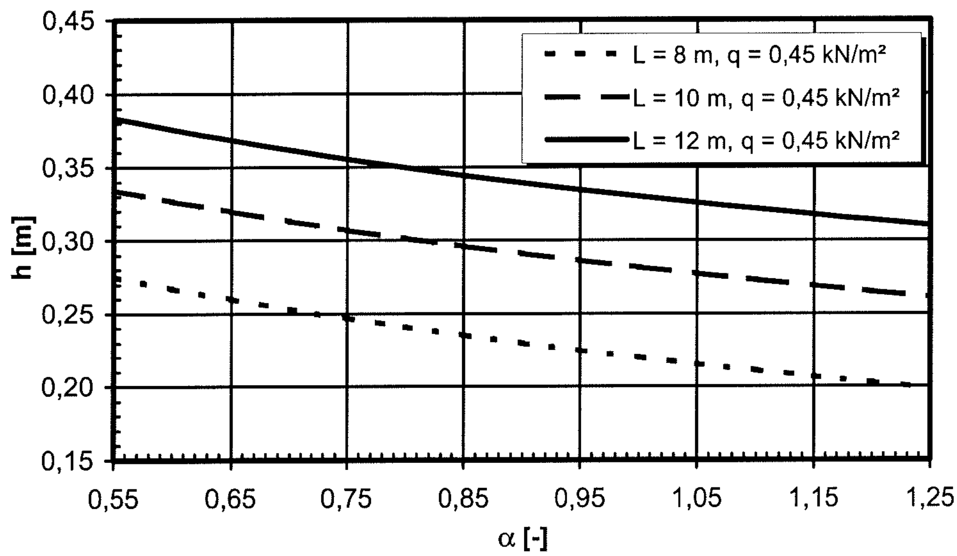


Fig. 7: α/h - nomograph to determine the required depth of the overall cross section

The nomographs were derived from the equation of a simple span deflection and the allowable total deflection according to DIN 1045-1:

$$f = (g+q)\cdot\frac{5}{384}\cdot l^4\cdot\frac{1}{\alpha\cdot EI_0} \le \frac{l}{250} \tag{5}$$

For the deflection increase after loading analogous nomographs were developed for an allowed deflection of $l/500$. The results of the investigation are given in [1].

3.6 Design concept

General information

The investigations showed that the design of prestressed filigree floors made of high-strength precast concrete with a normal-strength concrete topping can be accomplished according to DIN 1045-1 [4] and DIN 1045-4 [8]. Only the limitation of the deflection has to be conducted with a new approach.

Dimensioning

Due to economic reasons the precast unit of the filigree floor should be 8 cm deep and should be made of a concrete grade C 60/75. The normal-strength concrete of the structural topping should be C 20/25 respectively C 30/37. The use of a C 30/37 reduces the required slenderness by 10 to 15 % because of its higher rigidity and less creep and shrinkage.

In a first step, the mutual dependence between the prestress load and the required depth, calls for an assumption of the needed number of strands. In order to avoid an excessive deflection during the erection, a cracking has to be prevented at this state. Therefore, rectangular precast units of a filigree floor should be supported approximately every 3.0 m until the concrete topping is hardened. These supports are not required if the precast unit has a web of at least 30 cm per meter.

Design

The investigations showed that the deflection is critical to the design. Therefore, at first the required depth of the floor has to be determined. This can be done with the recommended approach of the effective flexural rigidity EI_{eff}.

The load-bearing capacity can be verified according to monolithic slabs. The design rules of DIN 1045-1 have to be kept. The concrete strength of the topping is crucial to the design. In order to prevent stress corrosion and to secure a ductile failure, it is recommended to limit the prestressing to σ_{pm0} = 1000 N/mm². In this case no further reinforcement is required. This regulation corresponds to different authority approvals.

As a result of the basic investigations of rectangular precast units in [2] an overhang of 3 cm is sufficient to prevent an anchorage failure for both the erection and the final state. A further proof is not required for this type of filigree floor. This is also true for the final state of filigree floors made of precast units with web. However, the required overhang for the erection state has to be proven explicitly for this type of filigree floor.

The design rules of DIN 1045-1 regarding the interface between precast and in-situ concrete are too conservative. The investigations showed that even an untreated interface without stirrups is unlikely to fail. The observed failures were analogous to monolithic slabs. However, further research is needed to increase the design capacity of the interface.

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