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Results of experimental studies of civil defense structures self-weight pre-stressed continuous three-span steel-reinforced concrete slabs

The paper presents results of experimentally tested samples of resource-saving non-cut three-span steel-reinforced concrete slabs that can be used for the construction of floors of civil defense structures. The specimens differed in the lengths of the concreting grips, which made it possible to study the development of deflections in the extreme and middle spans due to changes in the stiffness of the section at the supports. The length of the concreting section is determined by the points of zero bending moments. During the first stage of concreting, the middle span of the steel structure bends downward due to the concrete mix's own weight, which causes the extreme sections that are not under load to bend upward. After the concrete of the first stage reaches the design strength, the concrete of the outer spans is concreted. At the same time, the extreme spans bend downward due to the weight of the concrete mixture, forcing the middle span, which already has a steel-reinforced concrete section, to bend upward.

Keywords: experiment, continuous scheme, steel-reinforced concrete slabs, resource-saving manufacturing technology.

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Introduction

The problem of ensuring a uniform stress distribution in continuous steel–concrete composite slabs remains a pressing issue in modern construction. The application of prestressing technologies helps to reduce stress irregularities; however, the question of effectively inducing prestressing in experimental specimens of such slabs has not yet been sufficiently investigated. The study of this process is essential for improving the service performance of structures and enhancing their load-bearing capacity.

Review of the research sources and publications

Steel–concrete composite structures are employed both in new construction and in the reconstruction of buildings and facilities, achieved through the effective combination of predominantly bar-type steel elements and cast-in-place concrete [1]. Multi-span flexural steel–concrete composite structures, in which the cast-in-place reinforced concrete slab serves as the compressed planar component of the cross-section and the profiled steel member functions as the tensioned bar component, have proven their effectiveness due to the high constructability of their installation and their substantial load-bearing capacity in both civil and industrial construction [2; 3]. In addition, the cast-in-place reinforced concrete slab provides a rigid diaphragm for the floor system and enables the subdivision of the building's internal space into multiple storeys [4].

Quite often, in order to improve the constructability of such floor systems and reduce construction time, the cast-in-place reinforced concrete slab is arranged on permanent formwork made of profiled sheeting [5]. When composite action between the cast-in-place slab and the profiled steel sheeting of the permanent formwork is ensured through the ribbed surface of the decking, such slabs become comparable to reinforced concrete floor systems in terms of material consumption, since the profiled sheeting functions not only as formwork but also as external reinforcement of the slab [6].

The aforementioned advantages of steel–concrete composite slabs—namely, increased load-bearing capacity and high installation efficiency—enable their application in the construction of floor systems for protective structures of civil defense facilities [7]. However, the wide implementation of this structural solution in practice is limited by the absence of ribbed profiled sheeting on the building materials market, due to the more complex rolling technology required, as well as by the low fire and corrosion resistance of the exposed bottom surface of the profiled steel sheeting.

With equal support spacing of continuous steel–concrete composite slabs, the profiled sheeting of the middle spans, when employed as the working reinforcement of the slab, is subjected to lower stresses than that of the end spans [8]. To achieve uniform utilization of the load-bearing capacity of the profiled sheeting in continuous steel–concrete composite slabs, it is advisable to design the end spans longer than the central span [9].

Definition of unsolved aspects of the problem In modern construction, it is essential to ensure the stability and durability of structures. One approach to enhancing these performance indicators is the identification of effective technologies and methods for inducing prestressing in structural elements, which contribute to achieving an optimal stress distribution within them [10]. The study of this process enables the identification of best practices applied in construction and their subsequent implementation in production.

Problem statement

The aim of this study is to present the results of experimental investigations of resource-efficient, continuous three-span prestressed steel–concrete composite slab specimens, which may be utilized in the construction of floor systems for protective structures of civil defense facilities.

Basic material and results

The tests of the steel–concrete composite slab specimens were conducted when the concrete had reached an age of more than 28 days. The loading was applied using small-sized artificial weights—hollow ceramic bricks. To determine the weight of the bricks, a selective weighing procedure was performed: 5 bricks were weighed from each batch of 50 used bricks (i.e., approximately 1/10 of the total number of bricks employed for loading).

As a result of the experimental investigations of the two continuous steel–concrete composite slab specimens, data from measuring instruments installed according to the developed schemes provided information on both the slab deflections (deformability) and longitudinal strains in the extreme fibers. These measurements allowed for the subsequent determination of stresses and load-bearing capacity.

The experimental studies of the slab specimens were extended over time due to the period required for the cast-in-place reinforced concrete slab to attain its design strength. As previously noted, the fabrication—specifically, the concreting—of the slab specimens was performed in two stages. Therefore, the overall fabrication period for the steel–concrete composite slab specimens was approximately two months. During each concreting stage, deflections at characteristic cross-sections of the specimen, caused by the weight of the freshly placed concrete, were measured. An analysis was conducted on the variation of deflections along the length of the specimen and on the strains of the normal cross-section throughout the two fabrication stages and during the application of the service load.

Figure 1 shows the general view of one of the specimens under loading with small-sized weights (hollow ceramic bricks).

Figure 1 – General view of specimen 1.7–2.3–1.7 SCCS 0.53 × 6.0 under maximum loading.

Figure 2 presents the graphs of the maximum deflections along the length of the steel–concrete composite slab specimens at the end of the two concreting stages and under the maximum applied service load with small-sized weights. For a clearer analysis of the influence of concreting segment width on the deformability of the three-span slabs, the general view of each tested specimen is shown above the corresponding deflection graph.

Figure 3 illustrates the development of deflections in the end and central spans at each stage of the steel–concrete composite slab fabrication and during loading with bricks. As can be seen from the deflection variation graphs in Figure 3, extending the first concreting stage beyond the intermediate supports results in more limited adjustment of internal forces during the second concreting stage, compared to concreting the first stage up to the intermediate supports. Consequently, the load-bearing capacity of the continuous slab is exhausted in the central span, where, at the moment of failure, deflections are 32% greater than in the end spans.

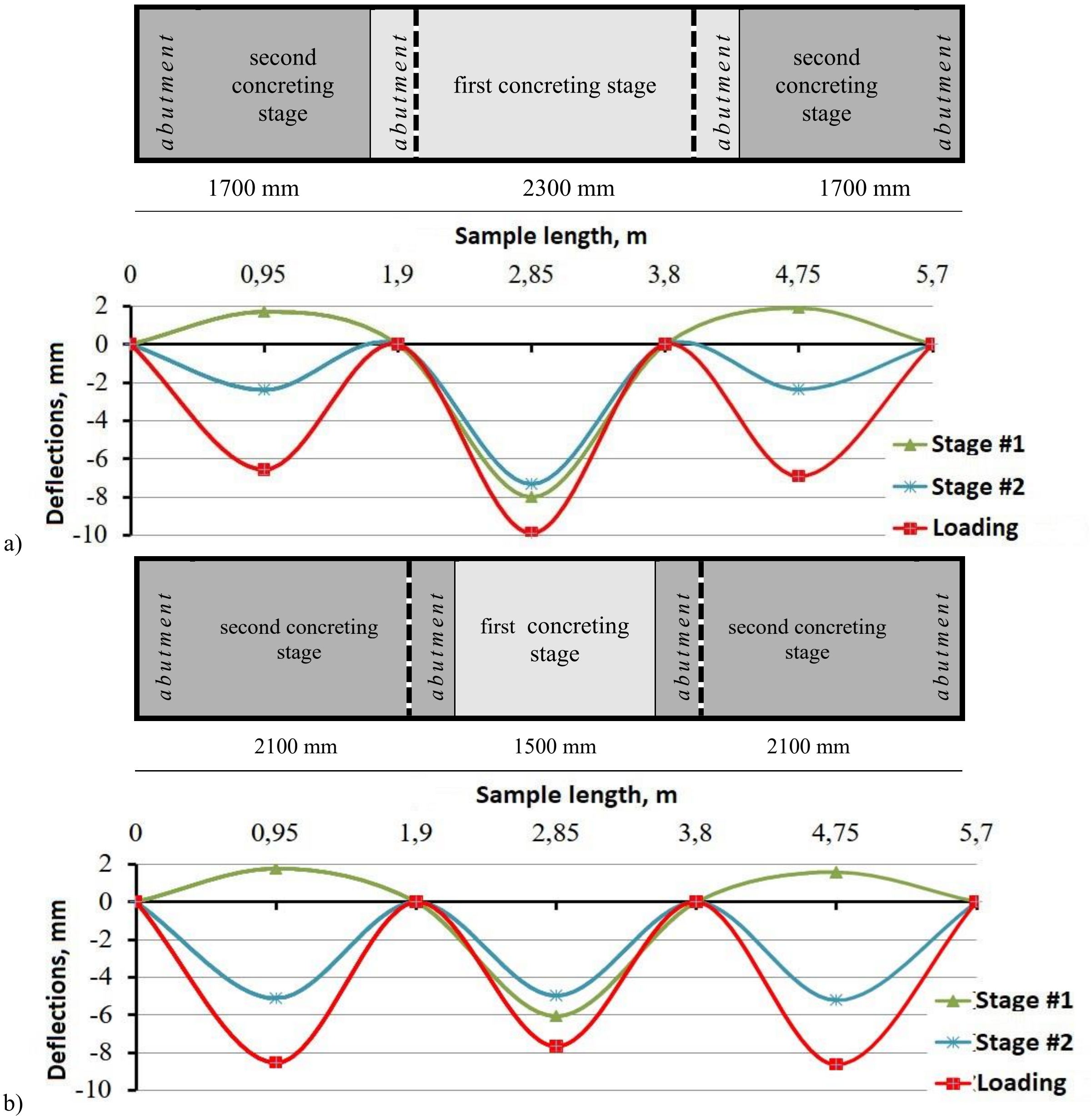

Figure 2 – Deflection variations of the specimens at the end of the two concreting stages and under maximum applied load: (a) 1.7–2.3–1.7 SCCS (0.53 × 6.0 m); (b) 2.1–1.5–2.1 SCCS (0.53 × 6.0 m).

For the slab with the first segment concreted up to the intermediate supports (specimen 2.1–1.5–2.1 SCCS 0.53 × 6.0), uniform deformability was achieved across all three spans (Figure 3b), with a deflection difference of 11.2% under maximum load.

Figure 4 shows the development of relative strains in the bottom and top fibers of the cross-sections at the midpoints of the left, central, and right spans of the steel–concrete composite slab specimens as the load in the cross-sections increased. The vertical axis represents the total load applied to the specimen, including its self-weight.

From the strain distribution diagrams in Figure 4, it can be observed that, for the specimen with a steel–concrete cross-section at the intermediate supports, at the moment of the second concreting stage and under the maximum applied load, the strain in the most tensioned fiber of the central span is 12.5% higher than in the end spans. For the second specimen, the corresponding strain difference in these cross-sections is 4.1%. Moreover, the maximum tensile strains in the second specimen under the same load level decreased by 44.7%.

Thus, based on the measurements of relative strains, the same conclusion regarding the segment lengths in two-stage concreting can be drawn as from the analysis of deflection development: for the investigated continuous three-span slabs with an external reinforcement ratio of 1.7% provided by the profiled sheeting, the first concreting stage should not extend to the intermediate supports by 1/10 of the span. Under this condition, the exhaustion of load-bearing capacity and deformability across all three spans occurs at the same level of applied external load. In this case, the steel decking in the span areas acts solely in tension, and the load-bearing capacity is governed by the attainment of yield stresses in the bottom flanges of the profiled sheeting.

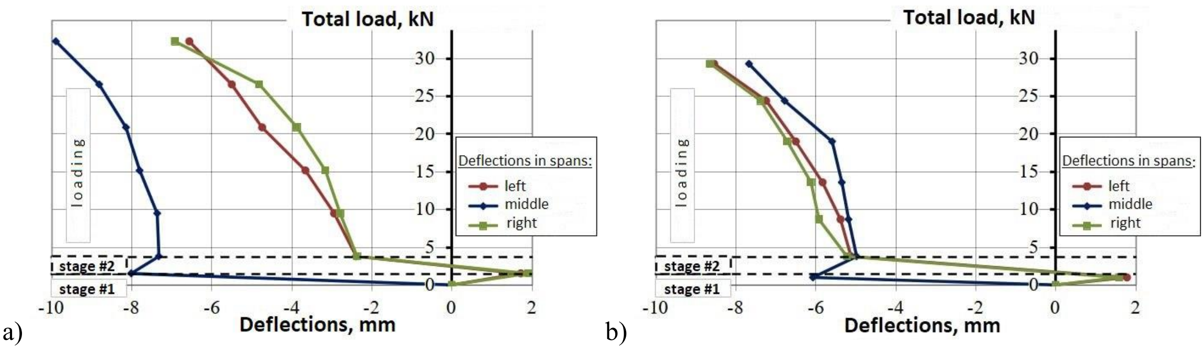

Figure 3 – Deflection development in the spans of steel–concrete composite slab specimens during each fabrication stage and under applied loading: (a) 1.7–2.3–1.7 SCCS (0.53 × 6.0 m); (b) 2.1–1.5–2.1 SCCS (0.53 × 6.0 m).

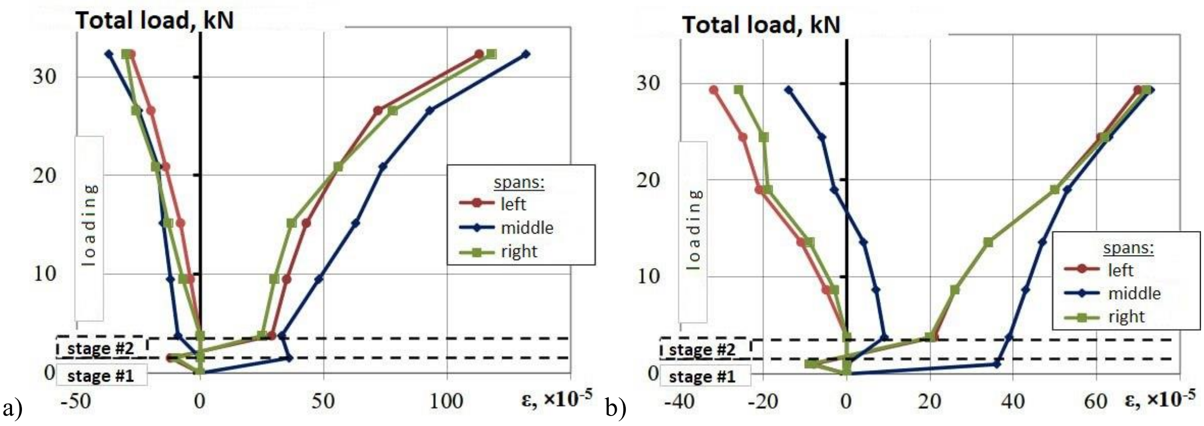

Figure 4 – Increments of relative strains in the bottom and top fibers of the cross-sections of the steel-concrete composite slab specimens: a) 1.7–2.3–1.7 SCCS (0.53 × 6.0 m); b) 2.1–1.5–2.1 SCCS (0.53 × 6.0 m)

Failure of the steel–concrete composite slabs occurred suddenly due to the formation of cracks in the top concrete zone at the intermediate supports (on the support face closer to the central span; see Figure 5), in the region of maximum bending moment within the tensile zone of the top fibers of the cross-section. No relative slip between the concrete slab and the steel decking was observed.

Figure 5 – Crack formation at the central support during the failure of the steel–concrete composite slabs

The proposed and experimentally investigated two-stage concreting technology for continuous cast-in-place steel–concrete floor slabs was implemented during the construction of a three-story superstructure atop a three-story public building in Poltava. Figure 6 shows the general view prior to casting the cast-in-place reinforced concrete slab of the steel beam framework, with the profiled permanent formwork in place and flexible anchors made of reinforcing bars welded in position. Figure 7 shows the overall arrangement of the supports — secondary steel beams — and the continuous cast-in-place reinforced concrete floor slab.

The application of this cast-in-place slab using the two-stage concreting technology allowed a reduction in steel consumption of up to 5% per square meter of the floor slab.

Figure 6 – General view prior to casting the cast-in-place reinforced concrete slab of the steel beam framework with the profiled permanent formwork in place (a) and flexible reinforcing bar anchors welded in position (b)

Figure 7 – General view of the support arrangement – the secondary steel beams – of the continuous cast-in-place reinforced concrete floor slab from below

Conclusions

Pre-stressing of bent steel–concrete composite structures, taking into account material nonlinearity, was studied using continuous three-span floor slabs as an example. The effectiveness of the two-stage fabrication technology for slabs prestressed by their self-weight, as well as the method for inducing initial camber, was experimentally demonstrated on two continuous three-span steel–concrete composite slab specimens, 6 m in length, cast on permanent profiled-sheet formwork. It was established that, for the investigated slabs with an external reinforcement ratio of 1.7%, the concreting of the central span during the first stage should not extend to the intermediate supports by 1/10 of the span. Under this condition, the exhaustion of load-bearing capacity and deformability in all three spans occurs at the same level of applied external load.

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Результати експериментальних досліджень попередньо напружених власною вагою нерозрізних трипролітних сталезалізобетонних плит споруд цивільного захисту

Проблема забезпечення рівномірного розподілу напружень у нерозрізних сталезалізобетонних плитах є актуальною для сучасного будівництва. Використання технологій попереднього напруження дозволяє зменшити нерівномірний розподіл напружень у перерізах таких плит. У статті представлені результати експериментальних випробувань зразків ресурсозберігаючих нерозрізних трипрогонових сталезалізобетонних плит, які можуть бути використані для будівництва перекрить споруд цивільного захисту. Зразки відрізнялися довжинами бетонувальних ділянок, що дозволило дослідити розвиток прогинів у крайніх та середніх прольотах через зміну жорсткості перерізу на опорах. Довжина бетонуючої ділянки визначається точками нульових згинальних моментів. Під час першого етапу бетонування середній проліт сталевої конструкції згинається вниз під дією власної ваги бетонної суміші, що призводить до згину крайніх ділянок, які не перебувають під навантаженням, вгору. Після досягнення бетоном першого етапу проектної міцності бетонують бетон зовнішніх прольотів. Водночас крайні прольоти під дією ваги бетонної суміші згинаються вниз, змушуючи середній проліт, який вже має сталезалізобетонний переріз, згинатися вгору. Експериментально доведено ефективність двостадійної технології виготовлення попередньо напружених від власної ваги й технології створення, за допомогою яких створюють попередні вигини, двох зразків нерозрізних трипролітних сталезалізобетонних плит довжиною 6 м, виконаних по незнімній опалубці із профільованого настилу. Встановлено, що для досліджених плит із коефіцієнтом зовнішнього армування 1,7%, бетонування середнього прольоту в першому етапі слід не доводити до середніх опор на 1/10 прольоту. За такої умови вичерпування несучої здатності та деформативності в трьох прольотах буде при однаковому рівні зовнішнього навантаження.

Ключові слова: експеримент, нерозрізна схема, сталезалізобетонні плити, ресурсоощадна технологія виготовлення.

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